GEOTECHNICAL INVESTIGATION
THE RISE
Cupertino, California

Prepared For:

Vallco Property Owner, LLC
Menlo Park, California

Prepared By:

Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, California 95113

Jenna Fontaine
Senior Staff Engineer

REGISTERED PROFESSIONAL ENGINEER
JOHN GOUCHON
NO. 2282
Exp. 6/30/25
GEOTECHNICAL
STATE OF CALIFORNIA

John Gouchon

John Gouchon, GE #2282
Principal/Vice President

4 December 2023
770633101

LANGAN

TABLE OF CONTENTS

ATTACHMENTS

FIGURES

APPENDICES

GEOTECHNICAL INVESTIGATION
THE RISE
Cupertino, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation by Langan for the proposed project called The Rise at 10123 N. Wolfe Road in Cupertino, California. The approximate location of the project is shown on Figure 1.

The site is north of the intersection of N. Wolfe Road and Stevens Creek Boulevard and encompasses approximately 30 acres. It is bound by Stevens Creek Boulevard to the south, Perimeter Road and residential housing to the west, Perimeter Road, and Interstate 280 to the north and commercial buildings to the east, as shown on Figure 2. N. Wolfe Road runs north-south through the site.

Currently, the site is occupied by remnants of the Vallco Shopping Center. The shopping center included a two-level shopping center building, multi-level parking structures, surface parking lots, a pedestrian bridge spanning N. Wolfe Road, a vehicle tunnel crossing below N. Wolfe Road, and several stand-alone buildings. The portion of the shopping center west of N. Wolfe Road has been razed. We understand that the portion of the shopping center east of N. Wolfe Road will be razed in the future during a separate phase.

Based on design development drawings [Kohn Pederson Fox Associates (KPF) 2023], the proposed residential, retail, and office buildings will be constructed in four phases and will be laid out in a grid forming 15 blocks, as shown on Figures 2 and 3, respectively. Phases 1 through 3 are west of N. Wolfe Road and Phase 4 is east of N. Wolfe Road, as shown on Figure 2. Blocks 1 through 10 and 13 will be located west of N. Wolfe Road and Blocks 11, 12, 14, and 15 will be located east of N. Wolfe, as shown on Figure 3. A brief description of the proposed development is presented in Table 1.

TABLE 1
Summary of Proposed Development

Phase	Block	Building Number	Basement Levels	Podium Levels	Tower Levels	Stories	Approximate Roof Height (feet)	Preliminary Dead Plus Live Foundation Bearing Pressures (psf)
1	1	1A	One[1]	2	4	6	85	1,450
		1B			4	6	85	1,450
	2	2A		3	5	8	85	2,000
		2B			5	8	85	2,000
	5	5A	–	3	5	8	85	1,550
		5B	–		5	8	85	1,550
2	3	3	–	2	5	7	85	1,300
	4	4	One[2]	4	16	20	230	5,050
	6	6	–	–	3	3	35	350
	7	7	–	3	4	7	85	1,500
	8	8A	–	4	4	8	85	1,750
		8B	–		14	18	205	4,450
3	9	9	–	3	5	8	85	1,550
	10	10A	–	3	5	8	85	1,550
		10B	–		5	8	85	1,550
	13	13A	One[2]	5	7	12	155	3,700
		13B			6	11	140	3,450
4	11	11A	–	3	5	8	85	1,550
		11B	–		5	8	85	1,550
	12	12A	–	3	5	8	85	1,550
		12B	–		12	15	170	3,800
	14	14	Three[3]	–	11	11	170	3,950
	15	15A		6	6	12	195	4,300
		15B			7	13	210	4,550
		15C			9	15	225	5,100

Notes:
1. According to correspondence with KPF on 8 November 2023, we understand that generally the basement heights will be 13 feet plus a 4-foot-thick mat foundation. For Blocks 1 and 2 specifically, we anticipate the excavation for the structures will be approximately 16 to 20 feet, which includes localized excavations for the elevator pits.
2. For Blocks 4 and 13, we anticipate a 13-foot basement height and a 4-foot-thick mat foundation, for a total excavation depth of 17 feet.
3. For Blocks 14 and 15, we anticipate each basement level will be 13 feet in height in addition to a 4-foot-thick mat foundation, for a total excavation depth of 43 feet.

According to DCI, the project structural engineer, we understand that, with the exception of the townhomes (shown as Block 6 on Figure 3) and any ancillary structures, the structures for the project are planned to be supported on mat foundations.

Based on a topographic survey of the project site (Sandis, 2016), the existing ground surface elevations range from Elevation 176.4 feet[1] at the north side of the project to Elevation 198.4 feet at the southwestern portion of the project.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposals dated 10 August 2016 and 2 June 2023 and our budget increase requests dated 1 November 2019 and 17 August 2020. We reviewed available subsurface information for the site and vicinity from our files and further explored subsurface conditions at the site by drilling borings and advancing cone penetrometer tests (CPTs). We conducted laboratory tests on samples recovered from the borings and used the results from our field exploration to perform engineering analyses and develop conclusions and recommendations regarding:

- anticipated subsurface conditions including groundwater levels;
- 2019 California Building Code (CBC) site classification, mapped values SS and S1, modification factors Fa and Fv and SMS and SM1;
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, fault rupture;
- appropriate foundation type(s) including shallow foundations and alternatives for deep foundations, as necessary;
- design parameters for the recommended foundation type(s), including vertical and lateral capacities and associated estimated settlements;
- lateral earth pressures for temporary and permanent shoring;
- lateral earth pressures for permanent basement walls;
- subgrade preparation for slab-on-grade floors and exterior slabs and flatwork, including sidewalks;
- site preparation, grading, and excavation, including criteria for fill quality and compaction;
- corrosivity, including a corrosion evaluation report;
- construction considerations.

[1] All elevations reference North American Vertical Datum of 1988 (NAVD88).

3.0 FIELD EXPLORATION AND LABORATORY TESTING

We began our investigation by reviewing previous geotechnical investigations performed at or in the vicinity of the site. To further investigate subsurface conditions at the site, we drilled five test borings, and performed five CPTs.

Prior to performing the field exploration, we:

- obtained a soil boring/monitoring well permit from the Santa Clara Valley Water District (SCVWD);
- notified Underground Service Alert;
- checked the boring locations for underground utilities using a private utility locator.

Details of the field exploration activities and laboratory testing are described in the remainder of this section.

3.1 Previous Investigation

We reviewed existing subsurface information from a report titled "Preliminary Geotechnical Investigation, The Hills at Vallco, Cupertino, California," dated 19 November 2015, by TRC.

We used the information provided on the boring logs from the above referenced report to supplement the information developed from our exploration of the site. The approximate locations of the previously drilled borings by TRC are shown on Figures 2 and 3. Logs of borings and the associated laboratory test results presented in the TRC report are presented in Appendix A.

3.2 Borings

Our field exploration included drilling five borings. The borings, designated as B-1 through B-5, were drilled at the site at the approximate locations shown on Figures 2 and 3. Borings B-1 and B-2 were drilled using truck mounted rotary wash drilling equipment from 6 through 8 September 2016 by Pitcher Drilling Company. The borings were drilled to depths of 101.5 and 141 feet bgs. Borings B-3 to B-5 were drilled using truck mounted hollow stem auger drilling equipment on 13 and 14 September 2016 by Exploration Geoservices. The borings were drilled to depths of 50 to 100 feet bgs.

During drilling, our field engineer logged the borings and obtained representative samples of soil encountered for visual classification and laboratory testing.

Logs of the borings are presented in Appendix B on Figures B-1 through B-5. The soil encountered in the borings was classified in accordance with the Classification Chart, presented on Figure B-6.

Samples were obtained using the following split-barrel sampler types.

- Sprague & Henwood (S&H) sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches.
- Standard Penetration Test (SPT) sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners.

The sampler types were chosen on the basis of soil type and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soils. The SPT sampler was used to evaluate the relative density of granular soils.

For the rotary wash borings (Borings B-1 and B-2), the SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.1, respectively, to account for sample type and hammer energy and are shown on the boring logs.

For the hollow stem auger borings (Borings B-3 to B-5), the SPT and S&H samplers were driven with a 140-pound, downhole, wireline safety hammer falling 30 inches. The blow counts required to drive the S&H and SPT samples were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sample type and hammer energy and are shown on the boring logs. Boring B-4 was drilled with two different drilling rigs due to equipment issues. The conversion factors to account for sample type and hammer energy were similar between both drilling rigs and hammers.

The SPT and S&H samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration or less if the blow count approached 50 blows. The driving of sampler was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration.

The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

NorCal Geophysical was retained to perform in-situ downhole suspension logging to measure the shear wave velocity of the subsurface materials within boring B-1. The details of the suspension logging methodology, procedures, and the results are presented in Appendix C.

Upon completion of drilling or suspension logging, the borings were backfilled with grout consisting of cement, bentonite, and water in accordance with the requirements of SCVWD. The borings were completed at the ground surface with cold patch asphalt. The soil cuttings and drilling fluid were placed in 55-gallon drums stored temporarily at the site, tested, and have been transported off-site for proper disposal.

3.3 Laboratory Testing

The soil samples recovered from the field exploration program were re-examined in the office for soil classification, and representative samples were selected for laboratory testing. The laboratory testing program was designed to evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, plasticity (Atterberg Limits), gradation, shear strength, and compressibility, where appropriate. Results of the laboratory testing are included on the boring logs and in Appendix D on Figures D-1 through D-15.

3.4 Cone Penetration Test

To supplement the soil boring data, five CPTs, designated as CPT-1 through CPT-5, were performed on 29 and 30 September 2016 by Gregg Drilling and Testing (Gregg) at the approximate locations shown on Figures 2 and 3. The CPTs were advanced to depths of approximately 75 feet bgs.

The CPTs were performed by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe, with a projected area of 15 square centimeters, into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges or load cells within the cone continuously measured the cone tip resistance and frictional resistance during the entire depth of each probing. Accumulated data was processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance and friction ratio by depth, as well as interpreted SPT N-Values, friction angle, soil strength parameters, and interpreted soil classification, are presented in Appendix E on Figures E-1 through E-5. Soil types were estimated using the classification chart shown on Figure E-6.

After completion, the CPTs were backfilled with cement-bentonite grout in accordance SCVWD requirements. The CPTs were completed at the ground surface with cold patch asphalt.

3.5 Soil Corrosivity Testing

To evaluate the corrosivity of the soil near the foundation subgrade, we performed corrosivity tests on samples obtained at depths of 18½ feet, 26 feet and 63½ feet. The corrosivity of the soil samples was evaluated by CERCO Analytical using the following ASTM Test Methods:

- Redox – ASTM D1498
- pH – ASTM D4972
- Resistivity (100% Saturation) – ASTM G57
- Sulfide – ASTM D4658M
- Chloride – ASTM D4327
- Sulfate – ASTM D4327

The laboratory corrosion test results and a brief corrosivity evaluation by JDH Corrosion are presented in Appendix F.

4.0 SITE AND SUBSURFACE CONDITIONS

The existing site and subsurface conditions observed and encountered at the site, respectively, are discussed in this section.

4.1 Site Conditions

Previously the site was a shopping mall development that included a two-level shopping center located on the east and west sides of N. Wolfe Road, multi-level parking structures, surface parking lots, a pedestrian bridge spanning N. Wolfe Road, a vehicular tunnel crossing below N. Wolfe Road, and several stand-alone buildings. However, the portion of the mall west of N. Wolfe Road has been razed. Based on a topographic survey of the project site (Sandis, 2011), the range of existing ground surface elevations is:

- West of N. Wolfe Road: Ground surface elevations range from Elevation 178.1 feet at the northern portion of the parcel to 198.4 feet at the southwest corner of the parcel;
- East of N. Wolfe Road: Ground surface elevations range from Elevation 176.4 feet at the northwest corner of the parcel to Elevation 198.0 feet at the eastern portion of the parcel.

4.2 Subsurface Conditions

Where asphalt pavement was encountered, the section consists of 1½ to 6 inches of asphalt concrete (AC) over 3 to 10 inches of aggregate base (AB). In general, the project site is underlain by alluvial deposits consisting of stiff to hard clays and sandy clays and medium dense to very dense sand and gravel. TRC (as Lowney Associates) encountered 1½ and 4½ feet of clay fill in borings LB-6 and LB-8, respectively. The surficial clayey soil has moderate to high expansion potential[2]; where tested, the upper clay layers have plasticity indices of 25 and 39. Where tested, laboratory test results of the undrained shear strength of relatively undisturbed samples of the alluvium ranges from 1,220 to 4,750 pounds per square foot (psf). An undrained shear strength of 640 psf was recorded during testing of a disturbed sample collected from boring B-1 at a depth of 75½ feet bgs. In addition, the consolidation laboratory test results indicate the alluvium is overconsolidated[3] and has compression ratios ranging from 0.1 to 0.12.

Idealized subsurface profiles, Figures 4 and 5, illustrate the general subsurface conditions at the site.

Based on our review of published maps (California Division of Mines and Geology, 2002), historic high groundwater in the project vicinity is deeper than 50 feet bgs. Based on previous geotechnical investigations at or nearby the project site, (Langan Treadwell Rollo, 2014 and TRC, 2015), groundwater was encountered at depths of approximately 65 to 75 feet bgs. During our current investigation, the groundwater levels were measured at depths of approximately 48 and 96 bgs (corresponding to Elevations 146 to 86 feet) at Borings B-1 and B-4, respectively. However, this depth was measured during drilling and may not represent a stabilized ground water level. Groundwater levels may fluctuate due to seasonal rainfall.

Pore-pressure dissipation tests[4] (PPDTs) were attempted at CPT-1 through CPT-5 at depths of approximately 62 feet to 75 feet bgs; groundwater was not encountered at those depths. Groundwater depth and elevation data from the current and prior investigations are summarized in Table 2.

[2] Highly expansive soil undergoes large volume changes with changes in moisture content.

[3] An overconsolidated clay has experienced a pressure greater than its current load.

[4] PPDTs are conducted at various depths to measure hydrostatic water pressures and to determine the approximate depth of the groundwater level. The variation of pore pressure with time is measured behind the tip of the cone and recorded.

TABLE 2
Summary of Groundwater Depth and Elevation Data

Consultant	Location	Year of Exploration	Ground Surface Elevation (ft)	Exploration Depth (ft)	Groundwater Depth (ft)	Groundwater Elevation (ft)
Langan	B-1	2016	194.2	141	48	146.2
	B-2	2016	197.6	101.5	-	
	B-3	2016	196.1	50	-	
	B-4	2016	182.4	100	96	86.4
	B-5	2016	179.8	50	-	
	CPT-1	2016	195.4	75.3	-	
	CPT-2	2016	194.2	75.3	-	
	CPT-3	2016	194.0	75.5	-	
	CPT-4	2016	176.4	75.3	-	
	CPT-5	2016	189.2	75.5	-	
TRC (as Lowney Associates)	EB-9	2004	184.2	84.5	68	116.2

Notes:
1. Groundwater level obscured by drilling method in Boring B 2.
2. Groundwater not encountered in Borings B 3, B 5, and CPT 1 to CPT 5.
3. TRC (as Lowney Associates or Lowney Kaldveer Associates) borings that did not encounter groundwater are not included.

Downhole suspension logging was performed in Boring B-1. Shear wave velocities ranged from about 790 to 2,498 feet per second in the alluvial deposits. A plot of shear wave velocity with depth is presented in Appendix C.

5.0 REGIONAL SEISMICITY

The project site is in a seismically active region. Numerous earthquakes have been recorded in the region in the past, and moderate to large earthquakes should be anticipated during the service life of the proposed development. The Monte Vista - Shannon, San Andreas, and Calaveras faults are the major faults closest to the site. These and other faults of the region are shown on Figure 6. For each of these faults, as well as other active faults within about 100 kilometers (km) of the site, the distance from the site and estimated mean Moment magnitude[5] [2014 Working Group on California Earthquake Probabilities (WGCEP) (2015) and Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report

[5] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

2013-1165] are summarized in Table 3. The mean Moment magnitude presented on Table 3 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

TABLE 3
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista - Shannon	4.4	Southwest	7.0
San Andreas 1906 event	10	Southwest	8.1
Pilarcitos	13	West	6.7
Butano	16	Southwest	6.7
Total Hayward-Rodgers Creek Healdsburg	19	Northeast	7.6
Sargent	22	Southeast	6.8
Total Calaveras	22	East	7.5
Mission (connected)	23	Northeast	6.1
Total San Gregorio	32	West	7.6
Greenville	46	East	7.1
Monterey Bay-Tularcitos	46	South	7.2
Mount Diablo Thrust	48	Northeast	6.6
Franklin	58	North	6.7
Contra Costa (Lafayette)	59	North	6.1
Contra Costa (Larkey)	60	North	6.0
Clayton	61	North	6.4
Great Valley 07 (Orestimba)	62	Northeast	6.8
Ortigalita (North)	63	East	6.6
Concord	64	North	6.4
Contra Costa Shear Zone (connector)	65	North	6.6
Quien Sabe	71	Southeast	6.4
San Andreas (Creeping Section)	75	Southeast	7.3
Contra Costa (Dillon Point)	77	North	6.1
Great Valley 05 Pittsburg - Kirby Hills	80	North	6.3
Green Valley	80	North	6.8
Ortigalita (South)	81	East	6.9
Contra Costa (Vallejo)	89	North	5.6
Contra Costa (Lake Chabot)	90	North	5.6
Great Valley 09 (Laguna Seca)	94	East	6.6
West Napa	95	North	6.8

Note:
1. The table above is a summary and does not include all the fault segmentation, alternate traces and low activity faults included in the UCERF3 model.

Figure 6 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 7) occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989 in the Santa Cruz Mountains with a M_w of 6.9; the epicenter of the earthquake was approximately 34 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2).

The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 95 km northeast of the site, with a M_W of 6.0.

The 2016 U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (Aagaard et al. 2016). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 4.

TABLE 4
Estimates of 30-Year Probability (2014 to 2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
Calaveras	26
N. San Andreas	22
Hunting Creek/ Berryessa/ Green Valley/ Concord/ Mt. Diablo/ Greenville	16
San Gregorio	6

6.0 GEOLOGIC HAZARDS

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[6], lateral spreading[7], and seismic densification[8]. Each of these conditions has been evaluated based on our literature review, field investigation, and analyses, and is discussed in this section.

6.1 Liquefaction and Associated Hazards

When saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of shear strength as a result of a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The site is not within a zone designated for liquefaction, as identified by the California Geologic Survey (CGS) in a map titled, *State of California Seismic Hazard Zones, Cupertino Quadrangle*, prepared by the California Geologic Survey, dated September 23, 2002 (CGS 2002a).

---

[6] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[7] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[8] Seismic densification (also referred to as Differential Compaction) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

Saturated loose sand was not encountered in the borings and CPTs drilled at the site. The high groundwater level observed at the site is approximately 48 feet bgs, corresponding to Elevation 146.2 feet. Blow count data indicates the cohesionless soil below the groundwater table is dense to very dense. Therefore, we conclude the potential for liquefaction and liquefaction-induced failures including lateral spreading is nil.

6.2 Seismic Densification

Seismic densification (also referred to as cyclic densification and differential compaction) can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. Up to five feet of medium dense clayey sand and silty sand was encountered in B-1 and B-2 above the groundwater table. This layer could densify in a major earthquake. Using the Tokimatsu and Seed (1984) method for evaluating seismically-induced settlement in dry sand, we estimate settlement will be less than ½ inch. The soil above the groundwater table encountered in the other borings is either very clayey or has sufficient density to resist seismic densification; therefore, we conclude the potential for seismic densification to occur is low at these locations.

6.3 Fault Rupture

Historically, ground surface ruptures closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Additionally, the site is not within an area mapped has having the fault rupture potential (County of Santa Clara, 2015). Therefore, we conclude the risk of fault offset through the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude that the risk of surficial ground deformation from faulting at the site is low.

7.0 DISCUSSION AND CONCLUSIONS

We conclude the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the project plans and implemented during construction. Average excavation depths of 19 to 34 feet bgs will be required to achieve the floor slab and foundation subgrades for the proposed buildings.

The primary geotechnical issues for this project include:

- presence of moderately to highly expansive clay at the ground surface

- selection of an appropriate foundation system to support the building loads and accommodate estimated static and seismic settlements;
- support for proposed excavations and adjacent structures during construction
- providing a stable subgrade and adequate working surface at the base of the excavation.

Our conclusions regarding these and other geotechnical issues are discussed in the remainder of this section.

7.1 Expansive Soil Considerations

The existing near-surface soil has moderate to high expansion potential. Moisture fluctuations in near-surface expansive soil could cause the soil to shrink or swell resulting in movement and potential damage to improvements that overlie them. Potential causes of moisture fluctuations include drying during construction, and subsequent wetting from rain, capillary rise, landscape irrigation, and type of plant selection.

The excavation for the basement levels will be below the zone of seasonal moisture change and expansive soil, if present, should not impact the foundations or basement slabs. For improvements at-grade, the volume changes from expansive soils can cause cracking of foundations, floor slabs and exterior flatwork. Therefore, foundations, slabs and concrete flatwork near existing grades should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by moisture conditioning the expansive soil and providing select, non-expansive fill below interior and exterior slabs and supporting foundations below the zone of severe moisture change.

In addition, the expansive clay may be susceptible to pumping and rutting during construction, especially if it becomes wet. If localized soft or wet areas of material are encountered it may be necessary to overexcavate the material 18 to 24 inches, place a geotextile fabric such as Mirafi 500X or its equivalent, and backfill with granular material to stabilize the area and bridge the soft material.

Alternatives to importing select fill include lime treatment of the near surface soil. The addition of lime can reduce the swell potential and increase the shear strength of the soil. Lime stabilization of the subgrade for exterior concrete flatwork may be a cost-effective means of improving on-site soils for use as non-expansive fill beneath the improvements. In addition, if the surface soil becomes wet, it may be difficult to compact during the winter. Lime treatment could be used to winterize the site and to aid in compaction.

The degree to which lime will react with soil depends on such variables as type of soil, minerals present, quantity of lime, and the length of time the lime-soil mixture is cured. The quantity of lime added generally ranges from 5 to 7 percent by weight and should be determined by laboratory testing. If lime is intended to reduce swelling potential and/or increase the strength of the soil, the lime treatment contractor should collect a bulk sample of the soil and perform laboratory tests to determine if the lime will react with the soil, the amount of lime required and the resulting plasticity index. We should be provided with the results to evaluate the effectiveness of the lime.

7.2 Foundations and Settlement

Based on the design development drawings (KPF, 2023), we understand the residential, retail, and office buildings located west of N. Wolfe Road at Blocks 1 and 2, 4, and 13 will have one basement level (basement finished floor at approximately 13 feet below street grade) and the office buildings located east of N. Wolfe Road at Blocks 14 and 15 will have three basement levels (basement finished floor at approximately 39 feet below street grade). The residential, retail, and office buildings at Blocks 3, and 5 through 12 will be at-grade.

7.2.1 Settlement of Buildings with Basements

Where the buildings will have basements, we judge the soil at the bottom of proposed excavations will consist of stiff to hard clay and medium dense to very dense sand and gravel. Therefore, provided the estimated settlements are tolerable, we conclude that buildings with basements can be supported on mat foundations. Design recommendations for the foundations for buildings with basements are presented in Section 8.2.

Laboratory test results indicate the clay below the proposed bottom of the excavations is overconsolidated, with overconsolidation ratios (OCRs) of about 2.1 to 2.2. The average net pressure from the weight of the structures (considering the stress relief from the existing and proposed basement excavations) is generally less than the preconsolidation pressure, therefore static settlements should be limited to immediate settlement. For the purposes of estimating immediate settlements under the applied foundation bearing pressures provided by DCI, we did not include the self-weight of the foundation. We assumed that the portion of immediate settlement of a mat foundation will occur during concrete curing of the mat and is not counted as part of the settlement of the structure immediately after construction.

Initially, as the proposed excavations are made, we expect the removal of soil will create pressure relief and the base of the excavation should rebound (rise), especially near the center of the excavation. We estimate rebound near the center of the excavation should be about ¾ inch after excavation of a one-level basement and 1¾ inches after the excavation of a three-level basement. After the new foundation is constructed and new building loads are applied, the pressure will increase, and the clay layer should partially recompress. Table 5 provides estimates of the static total and differential settlement for the proposed structures under the preliminary foundation bearing pressures provided by DCI. The estimates do not include the rigidity of a mat foundation system, which would tend to reduce the differential settlement.

7.2.2 Settlement of At-Grade Buildings

Where the buildings will be at-grade, we judge the soil at the foundation subgrade elevation will generally consist of stiff to hard clay. Therefore, provided that the estimated settlements are acceptable, we conclude that the at-grade buildings can be supported on spread footings or mat foundations. As noted in Section 1.0, per correspondence with DCI on 17 November 2023, we understand spread footings are only under consideration for the support of the Block 6 townhomes and ancillary structures and that mat foundations are planned for all other at-grade buildings. Design recommendations for the foundations for at-grade buildings are presented in Section 8.2. If the settlements are deemed to be excessive, ground improvement consisting of drilled displacement columns (DDC), or rigid inclusions could be performed, and shallow foundations could then bear on these elements. These types of ground improvement are typically designed and installed by specialty contractors.

Laboratory test results indicate the clay below the proposed shallow foundation subgrade is overconsolidated, with OCRs of about 2.1 to 2.2. For the purposes of estimating immediate settlements under the applied foundation bearing pressures provided by DCI, we did not include the self-weight of the foundation. We assumed that the portion of immediate settlement of a mat foundation will occur during concrete curing of the mat and is not counted as part of the settlement of the structure immediately after construction.

After the new foundations are constructed and new building loads are applied, the pressure will increase, and the clay layer should compress. Table 5 provides estimates of the static total and differential settlement for the proposed structures under the preliminary foundation bearing pressures provided by DCI. The estimates do not include the rigidity of a mat foundation system, which would tend to reduce the differential settlement.

TABLE 5
Summary of Building Settlement Estimates

Building Number	Basement Levels	Preliminary Dead Plus Live Foundation Bearing Pressures[1] (psf)	Total Mat Foundation Settlement[2] (inches)	Differential Mat Foundation Settlement[2,3] (inches)
1A	One	1,450	¼	¼
1B		1,450	¼	
2A		2,000	¼	¼
2B		2,000	¼	
3	–	1,300	½	¼
4	One	5,050	1¼	¾
5A	–	1,550	¾	¼
5B	–	1,550	¾	
6	–	350	¼	¼
7	–	1,500	¾	⅓
8A	–	1,750	1	¾
8B	–	4,450	2	
9	–	1,550	½	¼
10A	–	1,550	½	¼
10B	–	1,550	½	
11A	–	1,550	¾	½
11B	–	1,550	¾	
12A	–	1,550	1	½
12B	–	3,800	2	
13A	One	3,700	¾	⅓
13B		3,450	½	
14	Three	3,950	¼	¼
15A		4,300	¼	¼
15B		4,550	¼	
15C		5,100	¼	

Notes:
1. psf = pounds per square foot. Dead plus live foundation bearing pressures do not include the foundation weight and are based on the maximum height of each building, i.e., not including reduced foundation pressures at tower setbacks.
2. Total and differential settlements are static and based on the foundation bearing pressures proved by DCI Engineers on 27 October 2023. Total settlements for buildings including excavations are net settlements including rebound effects where soil unloading is estimated assuming excavation depths of 13 feet per basement level plus 4 feet for mat foundation.
3. Differential settlements are over a horizontal distance of 30 feet. The estimates do not include the rigidity of a mat foundation system, which would tend to reduce the differential settlement

Footings supporting the townhomes or lightly loaded, ancillary at-grade structures designed in accordance with the recommendations provided in Section 8.2.2 should not settle more than one inch; differential settlement between adjacent footings, typically 30 feet apart, should not exceed ½ inch. Additional recommendations for footings are presented in Section 8.2.2.

7.3 Groundwater Considerations

Groundwater levels encountered in the borings range from Elevation 146 feet at B-1 to Elevation 86 feet at B-4. On the basis of our knowledge of groundwater in the area, we conclude design groundwater elevations on the project site can be linearly interpolated between Elevation 146 feet at the southwest end and Elevation 86 feet at the northeast end.

7.4 Shoring Considerations

We understand that Blocks 1 and 2, Block 4, and Block 13 will have a one-level basement for parking with an average depth of about 17 feet, which accounts for a four-foot-thick mat. We understand Blocks 14 and 15 will have a three-level basement for parking with an average depth of about 43 feet, which accounts for a four-foot-thick mat. Temporary shoring recommendations for the one- and three-level basements are presented herein.

The excavation for the one-level basements may be sloped back if there is sufficient space, which is likely not possible on sides where the excavation would abut existing roadways. Alternatively, during excavation of any basements, the adjacent property and streets may be supported by temporary shoring. There are several key considerations in selecting a suitable shoring system. Those we consider to be primary concerns are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures;
- penetration of shoring supports into the predominantly sand and gravel soils below the bottom of the excavation;
- proper construction of the shoring system to reduce the potential for ground movement;
- cost.

Based on our experience on projects with similar excavation depths, soldier pile and timber lagging or overlapping soil-cement-mixed columns, in lieu of timber lagging may be the most economical shoring system for the excavations for this project.

Soldier pile and lagging consists of soldier piles placed in predrilled holes, which are backfilled with concrete or installed with a soil-cement mixing drill rig. Wood lagging is typically placed between the soldier beams as the excavation proceeds.

Alternatively overlapping soil-cement-mixed columns between soldier piles may be used in lieu of wood lagging. Soil-cement-mixed columns are installed by advancing hollow-stem augers and pumping cement slurry out through the tips of the augers during auger penetration. The soil is mixed with the cement slurry in situ, forming continuous overlapping soil-cement columns or continuous walls. Steel beams are placed in the soil-cement columns or walls at pre-determined spacing to provide rigidity.

The contractor should review the available boring logs in the previous reports and be prepared to encounter dense to very dense sand and gravel layers at various depths. Drilling of the shafts for the soldier piles may require casing and/or the use of drilling mud to prevent caving of any sand layers that are present. To reduce movements and caving, it may be necessary to limit the unsupported height of the excavation to the height of the lagging boards.

Excavations deeper than about 10 to 15 feet may require either post grouted tiebacks or internal bracing for lateral support unless the shoring is stiffened. The adjacent property owners should be notified of the planned excavation and consulted regarding any special requirements they may have for construction. It may be difficult to obtain permission to install tiebacks on their property.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle and move. The magnitude of shoring movements and resulting settlements are difficult to estimate because they depend on many factors, including the method and the shoring contractor's skill in the installation. If cohesionless layers are encountered, some caving may occur while lagging boards are installed. We estimate a properly installed shoring system will limit lateral movements and settlements to adjacent improvements to about 2/3 inch for a one-level basement. Typical maximum movement for a properly designed and constructed shoring system for the three-level basement should be within about 1½ inches. The settlement should decrease linearly with distance from the excavation and should be relatively insignificant at a distance twice the excavation depth. A monitoring program should be established to evaluate the effects of the construction on surrounding improvements, as discussed in Section 8.13.

The soil cement-mixed columns would be relatively rigid compared to wood lagging and could further limit lateral deflections and ground subsidence related to the shoring. Where movements could be detrimental to adjacent existing improvements the soil cement mixed columns could be used. A combination of the soldier pile and lagging and soil cement mixed column systems could be used depending on the required performance along the various excavation faces.

The selection, design, construction, and performance of the shoring system (see Section 8.7) should be the responsibility of the contractor. A civil engineer knowledgeable in this type of construction should be retained to design the shoring. We should review the final shoring plans to check that they are consistent with the recommendations presented in this report.

7.5 Excavation and Monitoring

The soil to be excavated from the site consists of materials that can be excavated with conventional earthmoving equipment such as loaders and backhoes, except where foundations and slabs of existing buildings are encountered. The presence of any existing structures within the zone of planned excavation will need to be verified in the field. Removal of these may require the use of jackhammers or hoe-rams. The equipment to be used should conform to vibration requirements set forth in Section 17.04 of the City of Cupertino Municipal Code. Excavations resulting from the removal of foundations, slabs and underground utilities that extend below the bottom of the proposed foundation/floor level should be cleaned of any loose soil/debris and backfilled with lean concrete or properly compacted fill. Existing basement walls and footings that interfere with the shoring system will need to be removed prior to installing the shoring.

The surficial soil is clayey and moderately to highly plastic. If earthwork is performed in wet weather conditions, it may be difficult to compact the soil; it may need to be aerated during dry weather. Light grading equipment may be needed to avoid damaging the subgrade.

7.6 Corrosion Potential

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed to evaluate the corrosivity of the near surface soil.

CERCO Analytical performed tests on soil samples to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Table 6 and Appendix F.

TABLE 6
Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfates (mg/kg)	Resistivity (ohms-cm)	Redox (mV)	Chlorides (mg/kg)
B-3	18.5	7.56	210	1,200	350	32
B-4	63.5	7.77	N.D.	3,900	350	N.D.
B-5	26	7.95	21	1,700	350	21

N.D. = None Detected

Based upon resistivity measurements, the soil samples tested are classified as "moderately corrosive" to "corrosive" to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. The chemical analysis indicates reinforced concrete and cement mortar coated steel, will be affected by the corrosivity of the soil. To protect reinforcing steel from corrosion, adequate coverage should be provided as required by the building code. Corrosivity test results are presented in Appendix F.

8.0 RECOMMENDATIONS

Recommendations for site preparation foundation design, temporary shoring and other geotechnical aspects of this project are presented in the following sections.

8.1 Earthwork

The following subsections present recommendations for site preparation and lime treatment.

8.1.1 Site Preparation

Demolition in areas to be developed should include removal of existing pavement and underground obstructions, including foundations of existing structures. Any vegetation and organic topsoil should be stripped in areas to receive new site improvements. Stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the owner and architect; organic topsoil should not be used as compacted fill.

Demolished asphalt and concrete at the site may be crushed to provide recycled construction materials, including sand, free-draining crushed rock, and Class 2 aggregate base (AB) provided it is acceptable from an environmental standpoint.

Existing underground utilities beneath areas to receive new improvements should be removed or abandoned in-place by filling them with grout. The procedure for in-place abandonment of utilities should be evaluated on a case-by-case basis and will depend on location of utilities relative to new improvements. However, in general, existing utilities within four feet of final grades should be removed, and the resulting excavation should be properly backfilled.

We recommend at least 18 inches of select material be placed beneath slab-on-grades for proposed at-grade structures that will be at or near existing grades and 12 inches beneath exterior concrete flatwork. Materials for the capillary break (sand and gravel) do not count as part of the select fill. The select fill should extend at least five feet beyond structure footprints and two feet beyond exterior concrete flatwork. Criteria for select fill are presented later in this section. Prior to placing fill, the subgrade exposed after stripping and site clearing, as well as other portions of the site that will receive new fill or site improvements, should be scarified to a depth of at least eight inches, moisture-conditioned to at least three percent above the optimum moisture content, and compacted to at least 88 percent relative compaction[9], where the exposed material consists of moderately to highly expansive soil. Expansive surface soil that has a moisture content of less than 20 percent (the approximate plastic limit of the soil) should be excavated, moisture-conditioned to at least three percent above optimum moisture content, and recompacted to between 88 and 93 percent relative compaction to reduce its expansion potential. Where lean clay or sandy soil are encountered during grading, the scarified surface should be moisture-conditioned to above the optimum moisture content and compacted to at least 90 percent relative compaction. An exception to this general procedure is within any proposed at-grade vehicle pavement areas supported on soil, where the upper six inches of the pavement subgrade should be compacted to at least 95 percent relative compaction regardless of expansion potential.

Heavy construction equipment should not be allowed directly on the final basement subgrade. The clay or sand exposed at the foundation/basement level may be susceptible to disturbance under construction equipment loads. It may be necessary to place a minimum 12-inch working pad consisting of crushed rock on top of the subgrade to minimize disturbance; the need for a working pad should be evaluate during construction as the bottom of the excavation is reached.

Any select fill placed during grading should meet the following criteria:

- be free of organic matter;
- contain no rocks or lumps larger than three inches in greatest dimension;
- have a low expansion potential (defined by a liquid limit of less than 40 and plasticity index lower than 12)
- have a low corrosion potential[10]
- be approved by the geotechnical engineer.

All fill placed beneath the basement and other improvements should meet the criteria for select fill. All select fill should be moisture-conditioned to near optimum moisture content, placed in horizontal lifts not exceeding eight inches in loose thickness, and be compacted to at least

9 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-12 laboratory compaction procedure.

10 Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

90 percent relative compaction, except for fill that is placed within the proposed pavement areas. In these situations, the upper six inches of the final soil subgrade and aggregate baserock should be compacted to at least 95 percent relative compaction. Where used, sand containing less than 10 percent fines (particles passing the No. 200 sieve) should also be compacted to at least 95 percent relative compaction. Samples of on-site and proposed import fill materials should be submitted to Langan for approval at least three business days prior to use at the site.

8.1.2 Lime Treatment (Optional)

Alternatively, the upper 18-inches of the existing surface soil may be lime treated to reduce the expansion potential and help winterize the site. We recommend that at least 5 percent lime by weight of the soil be used to treat the upper 18-inches of native soil for at-grade structures. A specialty contractor should be engaged to evaluate the type and amount of lime needed to reduce the plasticity index of the soil to meet the select fill criteria and provide laboratory test results to confirm the plasticity index of the soil after treatment.

Lime treatment of fine-grained soils generally includes site preparation, application of lime, mixing, compaction, and curing of the lime treated soil. Field quality control measures should include checking the depth of lime treatment, degree of pulverization, lime spread rate measurement, lime content measurement, and moisture content and density measurements, and mixing efficiency. Quality control will also include laboratory tests for unconfined compressive strength tests on representative samples.

The lime treatment process should be designed by a contractor specializing in its use and who is experienced in the application of lime in similar soil conditions. Based on our experience with lime treatment, we judge that the specialty contractor should be able to treat the moderate to highly expansive on-site material to produce a non-expansive fill for building subgrade.

If the lime treatment alternative is selected, we recommend that the specialty contractor prepare a treatment specification for our review prior to construction.

8.2 Foundations

The following section provides recommendations for mat foundations and spread footings.

8.2.1 Mat Foundation

Provided the static settlements estimates discussed in Section 7.2 are acceptable, we conclude the proposed buildings can be supported on a mat bearing on stiff to hard clay or medium dense to very dense sand and gravel.

The estimated static settlements associated with an estimated allowable dead plus live foundation bearing pressures provided by project structural engineer are presented in Table 5. The proposed buildings are currently anticipated to have average foundation bearing pressures ranging from 350 to 5,100 psf, excluding the weight of the foundation. We should provide revised static settlement estimates if the average dead plus live foundation bearing pressures exceed those shown in Table 5 by more than 10 percent. The structural engineer should evaluate the settlements of the structure using a modulus of subgrade reaction method.

The recommended static and dynamic moduli of subgrade reaction for mat foundations constructed at-grade, with a one-level basement, and with a three-level basement are presented in Table 7.

TABLE 7
Moduli of Subgrade Reaction for Mat Foundations

Case	Static Modulus of Subgrade Reaction (kcf)[1]	Dynamic Modulus of Subgrade Reaction (kcf)
At-Grade	25	30
One-Level Basement	60	70
Three-Level Basement	200	240

Note: 1. kcf = kips per cubic foot

The moduli values are representative estimates of the anticipated settlement under the building foundation bearing pressures. After the mat analysis is completed, we should review the computed settlement and bearing pressure profiles to check that the modulus values are appropriate.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. To calculate the passive resistance against the vertical faces of the basement walls or mat foundations supporting buildings with basement levels, we recommend an equivalent fluid weight of 400 pounds per cubic foot (pcf) with a maximum value of 2,000 pcf. To calculate the passive resistance against the vertical faces of mat foundations supporting at-grade structures, we recommend an equivalent fluid weight of 250 pcf with a maximum value of 1,250 pcf. The upper foot should be ignored unless confined by a slab. If waterproofing is used, the allowable friction factor will depend on the type of waterproofing used at the base of the foundation. For bentonite-based waterproofing membranes, such as Paraseal and Voltex, a friction factor of 0.15 should be used.

Friction factors for other types of waterproofing membranes should be provided by the manufacturer. If waterproofing is not used, frictional resistance should be computed using a base friction coefficient of 0.3. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

If weak soil is encountered at the mat excavation bottom, it should be over-excavated and replaced with engineered fill or lean concrete. The bottom and sides of mat excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will heave when exposed to moisture, which may result in cracking and distress.

We should observe mat subgrade prior to placement of reinforcing steel. The excavation for the mat should be free of standing water, debris, and disturbed materials prior to placing concrete.

8.2.2 Spread Footing Foundations

For footings supporting at-grade ancillary structures and the Block 6 townhomes, we recommend a minimum embedment of 36-inches below the lowest adjacent soil subgrade. For the recommended minimum embedment, footings bearing on firm native soil or engineered fill may be designed for an allowable bearing pressure of 3,000 psf for dead plus live loads, with a one-third increase for total loads, including wind and/or seismic loads.

Footings should be at least 18 inches wide for continuous footings and 24 inches for isolated spread footings. Footings adjacent to utility trenches (or other footings) should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the utility trench (or adjacent footings).

Lateral forces can be resisted by a combination of friction along the base of the footing, and passive resistance against the vertical faces of the foundation and, where applicable, the basement walls perpendicular to the direction of earthquake shaking. Frictional resistance should be computed using a base friction coefficient of 0.30. If waterproofing is used, the allowable friction factor will depend on the type of waterproofing used at the base of the foundation. For bentonite-based waterproofing membranes, such as Paraseal and Voltex, a friction factor of 0.15 should be used. Friction factors for other types of waterproofing membranes should be provided by the manufacturer. If passive pressure on the walls is relied upon for lateral resistance, the walls should be designed to resist the passive pressure. To calculate the passive resistance against the vertical faces of the basement walls or footings, we recommend an equivalent fluid weight of 400 pcf with a maximum value of 2,000 pcf. To calculate the passive resistance against the vertical faces of footings supporting at-grade structures, we recommend an equivalent fluid weight of 250 pcf with a maximum value of 1,250 pcf. The upper foot should be ignored unless confined by a slab. The values for the friction coefficient and passive pressures include a factor of safety of 1.5 and may be used in combination without reduction.

A firm subgrade should be exposed at the bottom of the proposed footing excavations. If isolated areas of soft material are encountered in the bottom of the excavation, they should be removed to expose firm material. Resulting overexcavations should be backfilled with lean or structural concrete. The bottom and sides of the footing excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will heave when exposed to moisture, which may result in cracking and distress.

We should observe the footing subgrade prior to placement of reinforcing steel. The excavation for the footings should be free of standing water, debris, and disturbed materials prior to placing concrete.

8.3 Floor Slab

The subgrade soil for buildings with basements should be very stiff or dense. The subgrade soil for at-grade buildings should be stiff to hard. Therefore, we conclude the basement slabs can be supported on grade. Where soft or loose soil is present at the basement slab subgrade, the weak soil should be removed and replaced with engineered fill or lean concrete.

Where slab-on-grade floors are to be cast, the soil subgrade should be scarified to a depth of six inches, moisture conditioned to near (or above) optimum moisture content, and rolled to provide a firm, non-yielding surface compacted to at least 90 percent relative compaction. Lime treated soil should be compacted to at least 90 percent relative compaction. If the subgrade is disturbed during excavation for shallow foundations and utilities, it should be re-rolled. Loose, disturbed materials should be excavated, removed, and replaced with engineered fill during final subgrade preparation.

Moisture is likely to condense on the underside of the slabs, even though they will be above the design groundwater table. Consequently, a moisture barrier should be installed beneath the slabs if movement of water vapor through the slabs would be detrimental to its intended use. A moisture barrier is generally not required beneath parking garage slabs, except for areas beneath mechanical, electrical, and storage rooms. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder.

The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in the latest edition of ASTM E1745. The vapor retarder should be placed in accordance with the requirements of the latest edition of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The particle size of the gravel/crushed rock should meet the gradation requirements presented in Table 8.

TABLE 8
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.45. Water should not be added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer’s requirements.

8.4 Permanent Below-Grade Wall Design

To construct the basement walls, the site may be open cut and/or temporarily shored. It is the responsibility of the contractor to determine the safe excavation slopes; however, we recommend cuts greater than four feet be no steeper than 1.5:1 (horizontal:vertical).

Because the on-site soil is expansive, we recommend designing below grade walls, such as the permanent shoring wall, for at-rest lateral pressures corresponding to equivalent fluid unit weights of 70 pcf and 90 pcf for drained and undrained conditions, respectively. Because the site is in a seismically active area, the design should also be checked for seismic conditions. Under seismic loading conditions, there will be an added seismic increment that should be added to active earth pressures (Sitar et al. 2012). We used the procedures outlined in Sitar et al. (2012)

and the peak ground acceleration based on the DE ground motion level (see Section 8.6) to compute the seismic pressure increment. Basement walls should be designed for the equivalent fluid weights and pressures presented in Table 9A.

TABLE 9A
Basement Wall Design Earth Pressures Backfilled with Native Soil (Drained Conditions above Design Groundwater Level)

	Static Conditions		Seismic Conditions[1]
	Unrestrained Walls – Active (pcf[3])	Restrained Walls – At-rest (pcf)	Total Pressure – Active Plus Seismic Pressure Increment (pcf)
Drained Condition[2]	45	70	80
Undrained Condition	80	90	100

Notes:
1. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.
2. Applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure.
3. pcf = pounds per cubic foot

If open cuts are made for the basement walls and select fill is used as backfill, then the walls may be designed with the earth pressures presented in Table 9B.

TABLE 9B
Basement Wall Design Earth Pressures with Select Fill Backfill (Drained Conditions above Design Groundwater Level)

	Static Conditions		Seismic Conditions[1]
	Unrestrained Walls – Active (pcf[3])	Restrained Walls – At-rest (pcf)	Total Pressure – Active Plus Seismic Pressure Increment (pcf)
Drained Condition[2]	35	55	70
Undrained Condition	80	90	100

Notes:
1. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.
2. Applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure.
3. pcf = pounds per cubic foot

Non-expansive wall backfill should consist of select fill, as described in Section 8.1. For cantilever walls retaining level backfill (i.e., landscape walls), the pressures presented on Table 9A or Table 9B may be used and will depend if the wall retains native soil (expansive) or select fill.

If surcharge loads occur above an imaginary 45-degree line projected up from the bottom of a retaining wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the added pressure on a case-by-case basis. Where truck traffic will pass within 10 feet of retaining walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot applied in the upper 10 feet of the walls.

The lateral earth pressures recommended for the sections above the water table are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the walls. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or wrapped in filter fabric (Mirafi 140N or equivalent). We should check the manufacturer’s specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. The pipe should be connected to a suitable discharge point. As an alternative to using prefabricated drainage panel, the wall may be drained using Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68-1.025) or clean drain rock wrapped in a geotextile filter fabric (Mirafi 140N or equivalent). The gravel drain should be at least 12 inches wide and should extend up the back of the wall to about 2 feet below the ground surface; the upper 2 feet should be covered with a clay cap to reduce infiltration of surface water. A four-inch-diameter perforated PVC collector pipe should be placed within the gravel blanket near the base of the wall to drain the water to a suitable discharge. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or drain rock and should be connected to a suitable discharge point.

Wall backfill should be compacted to at least 90 percent relative compaction using light compaction equipment. Wall backfill with less than 10 percent fines, or deeper than five feet, should be compacted to at least 95 percent relative compaction for its entirety. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

8.5 Concrete Pavement and Exterior Slabs

Differential ground movement due to expansive soil and settlement will tend to distort and crack the pavements and exterior improvements such as courtyards and sidewalks. Periodic repairs and replacement of exterior improvements should be expected during the life of the project. Mastic joints or other positive separations should be provided to permit any differential movements between exterior slabs and the buildings.

To reduce the potential for cracking related to expansive soil, we recommend exterior concrete flatwork be underlain by at least 12-inches of select fill, of which the upper four inches should consist of aggregate base compacted to at least 95 percent relative compaction. The subgrade should be compacted to at least 90 percent relative compaction and should provide a smooth, non-yielding surface for support of the concrete slabs.

Where rigid pavement is required for loading and service areas, we recommend a minimum of six inches of concrete for medium traffic and a minimum of eight inches of concrete for heavy traffic. The upper six inches of subgrade should be compacted to at least 95 percent relative compaction and should provide a smooth, non-yielding surface. The concrete should be underlain by at least 6 inches of Class 2 Aggregate base. Aggregate base material should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications.

8.6 Seismic Design

The following subsections present the recommended site-specific response spectra performed for the previous development scheme under CBC 2016/ASCE 7-10 (Section 8.6.1) and the code based mapped values per 2019 CBC/ASCE 7-16 (Section 8.6.2).

8.6.1 Site-Specific Response Spectra

We expect this site will experience strong ground shaking during a major earthquake on any of the nearby faults. To estimate ground shaking at the site, we developed site-specific response spectra. We performed a Probabilistic Seismic Hazard Analysis (PSHA) and deterministic analysis to develop site-specific horizontal response spectra for two levels of shaking corresponding to the Risk-targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake (DE) per the 2016 CBC. The MCE_R is defined in the 2016 CBC as the lesser of the probabilistic spectrum having 2 percent probability of exceedance in 50 years or the 84th percentile deterministic event on the governing fault, both in the maximum direction; the DE is defined as 2/3 of the MCE_R.

The probabilistic seismic hazard analysis (PSHA) was performed using the computer code EZFRISK 8.06 (Risk Engineering 2019). This approach is based on the probabilistic seismic hazard model developed by Cornell (1973) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources and earthquake activities were assigned to the faults based on historical and geologic data.

Details of our analyses are presented in Appendix G. The recommended horizontal ground surface spectra are shown on Figure 8. Digitized values of the recommended MCE_R and DE spectra for the site and a damping ratio of 5 percent are presented in Table 10.

TABLE 10
Digitized Values of the Recommended MCE_R and DE Spectra

Period (seconds)	MCE_R	DE
0.01	0.806	0.537
0.10	1.608	1.072
0.20	1.997	1.331
0.30	1.912	1.274
0.40	1.717	1.145
0.50	1.568	1.046
0.60	1.412	0.942
0.75	1.230	0.820
1.00	1.012	0.674
1.50	0.736	0.490
2.00	0.578	0.385
3.00	0.411	0.274
4.00	0.319	0.213
5.00	0.258	0.172
6.00	0.205	0.136
7.00	0.171	0.114
8.00	0.143	0.095

Because site-specific procedure was used to determine the recommended MCE_R and DE response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-10 should be used as shown in Table 11. We recommend that the site-specific values be used for design.

TABLE 11
Design Spectral Acceleration Value

Parameter	**Spectral Acceleration Value (g's)**
S_{MS}	1.997
S_{M1}	1.156*
S_{DS}	1.331
S_{D1}	0.770*

* S_{M1} and S_{D1} are based on the site-specific response spectra and are governed by the spectral acceleration at a period of two seconds.

8.6.2 Code Based Mapped Values

For seismic design in accordance with the provisions of 2019 CBC/ASCE 7-16, we recommend the following:

- Risk Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.768g and 0.626g, respectively.
- Site Class C
- Site Coefficients F_A and F_V of 1.2 and 1.4
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 2.122g and 0.876g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.415g and 0.584g, respectively.
- PGA_M is 0.873g

8.7 Shoring Design

As discussed in the Section 7.4, a soldier-pile-and-wood-lagging system or soil-cement-mixed columns between soldier piles are acceptable methods to retain the excavation where open cuts are not feasible. A cantilever soldier-pile-and-lagging shoring system can be designed to resist an active earth pressure of 45 pounds per cubic foot (pcf). The lateral pressures recommended for designing tied-back or braced shoring systems are presented on Figures 9 and 10 for temporary soldier pile with wood lagging and soldier pile with soil-cement columns, respectively.

The passive pressures presented on Figures 9 and 10 include a safety factor of 1.5 and may be used for the design of both cantilever and tieback shoring.

Recommendations for computing penetration depth of soldier piles to resist vertical loads are presented in Section 8.7.1.

Shoring that will support remaining buildings should be designed for additional surcharge pressures from the nearby footings. Estimated surcharge pressures from shallow foundations are provided in Figures 11 through 13.

If traffic occurs within 10 feet of the shoring, a uniform surcharge load of 100 psf should be added to the upper 10 feet for the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable.

The shoring system should be designed by a licensed civil engineer experienced in the design of retaining systems and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

8.7.1 Penetration Depth of Soldier Piles

Although the shoring designer should evaluate the required penetration depth of the soldier piles, we recommend that the soldier piles should extend below the excavation bottom a minimum of five feet. The soldier piles should have sufficient axial capacity to support the vertical load component of the tiebacks and the vertical load acting on the piles, if any. To compute the axial capacity of the piles, we recommend using an allowable friction of 1,000 psf on the perimeter of the piles below the excavation level.

8.7.2 Tieback Design Criteria and Installation Procedure

Tiebacks may be used to restrain the shoring. The vertical load from the tiebacks should be accounted for in the design. Design criteria for tiebacks are presented on Figures 9 and 10.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point 0.2H feet away from the bottom of the excavation and sloping upwards at

60 degrees from the horizontal, where H is the wall height in feet. Tiebacks with bar and strand tendons should have a minimum unbonded lengths of 10 and 15 feet, respectively. All tiebacks should have a minimum bonded length of 15 feet and should be spaced at least four feet on center. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, and workmanship. The existing sandy soil may cave; therefore, solid flight augers should not be used for tieback installation. We recommend a smooth cased tieback installation method (such as a Klemm type rig) be used. For estimating purposes, we recommend using the skin friction values presented on Figures 9 and 10. These values include a factor of safety of about 1.5. Higher skin friction values may be used, if confirmed with pre-production performance tests.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with their installation method. The computed bond length should be confirmed by a performance- and proof-testing program under the observation of an engineer experienced in this type of work. Replacement tiebacks should be installed for tiebacks that fail the load test. Recommendations for tieback testing are presented in Section 8.7.3.

8.7.3 Tieback Testing

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other tiebacks should be proof-tested to at least 1.25 times the design load. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inches of movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inches of movement between six and 60 minutes and the total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that fail to meet the first criterion will be assigned a reduced capacity.

If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the contractor should replace the tiebacks.

8.7.4 Internal Bracing

Where internal bracing is selected for lateral support of the shoring instead of, or in addition to tiebacks, rakers or struts (diagonal or cross-lot) may be used.

If struts are used as internal bracing, temporary pin piles can be installed within the excavation to support the vertical load from the bracing system. The axial capacity for piles should be determined using an allowable skin friction value of 1,000 psf, which includes a factor of safety of at least 2.0. End bearing should be neglected. Because the piles are temporary and are being designed for a factor of safety of at least 2.0, load testing is not required. Alternatively, new footings or mats can be used to provide lateral and vertical support for internal bracing elements.

If rakers are used as internal bracing, raker installation should be sequenced in such a manner as to avoid excessive deflection of the shoring system. The initial excavation, prior to raker installation, can be a cantilever excavation that should be designed such that shoring deflection does not exceed the design criteria for the shoring system. The maximum cantilever height will be a function of the stiffness of the soldier piles and lagging system.

A soil berm should be maintained from the bottom of the cantilever section of the shoring to the bottom excavation until the rakers are installed and the concrete for the pin piles has reached the required strength. The soil berm should have a five-foot-wide bench at the top and a slope no steeper than 1.5:1 (horizontal to vertical). At a 1.5:1 (horizontal to vertical) slope, the soil berm can provide a passive resistance of 75 pcf. The passive resistance value includes a factor of safety of about 1.5. The berm can be slot cut if required to facilitate installation of the rakers, however the maximum slot width cut should not exceed two feet. Depending on final spacing of the rakers, raker installation may need to be sequenced such that one raker and pin pile are installed prior to slot cutting of the berm and installation of an adjacent raker. The soil berm in front of the excavation should remain in place until the rakers and the supporting pin pile are installed, and the concrete has reached the minimum strength required by the shoring design engineer.

The rakers should remain in place until the below grade walls and floor slabs up to the top of the raker have been installed and the concrete has reached sufficient strength to support the soil and surcharge pressures.

8.8 Asphalt and Resin Pavements

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of stiff to hard clay or engineered fill. On the basis of the laboratory test results on this soil, we selected an R-value of 9 for design. Subgrade soils in paved areas, whether at-grade or on the roof, should have an R-value of 9 or higher. Therefore, additional tests should be performed on the proposed subgrade soil to measure its R-value prior to use in pavement areas. Depending on the results of the tests, the pavement design may need to be revised.

For pavements subjected to vehicle loads, we assumed a Traffic Index (TI) of 4 for automobile parking areas with occasional trucks, and 5 and 6 for driveways and truck-use areas; these TIs should be confirmed by the project civil engineer. Table 12 presents our recommendations for asphalt or resin pavement sections.

TABLE 12
Pavement Section Design

TI	Asphaltic Concrete or Resin Pavement (inches)	Class 2 Aggregate Base R = 78 (inches)
4	2.5	7
5	3	9
6	4	11

For pavements not subjected to vehicle loads, we recommend a minimum of 2.5 inches of asphalt or resin pavement over 4 inches of Class 2 aggregate base. These sections should be checked against City of Cupertino minimum standards.

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

8.9 Utilities

The corrosivity report provided in Appendix F of this report should be reviewed and corrosion protection measures used if needed. A corrosion engineer should be retained if detailed recommendations are needed.

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench excavations should be shored and braced, in accordance with all safety regulations, to prevent cave-ins. If sheet piling is used as shoring, and is to be removed after backfilling, it should be placed a minimum of two feet away from the pipes or conduits to prevent disturbance to them as the sheet piles are extracted. It may be difficult to drive sheet piles if cobbles, coarse grained gravel layers or buried obstructions are encountered.

Backfill for utility trenches should be compacted according to the recommendations presented for the general site fill. Jetting of trench backfill should not be permitted. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped or compacted with a vibratory plate. Backfill should be placed in lifts of eight inches or less, moisture-conditioned, and compacted to at least 90 percent relative compaction. If sand or gravel with less than 10 percent fines (particles passing the No. 200 sieve) is used, it should be compacted to 95 percent relative compaction.

Special care should be taken in controlling utility backfilling in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to exterior improvements.

Where utility trenches backfilled with sand or gravel enter the building pads, an impermeable plug consisting of low-expansion potential clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

8.10 Site Drainage

Positive surface drainage should be provided around the buildings to direct surface water away from the existing building foundations. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the buildings be designed to slope down and away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

8.11 Bioretention Systems

Bioretention areas are landscaping features used to treat stormwater runoff within a development site. They are commonly located in parking lot islands and landscape areas. Surface runoff is directed into shallow, landscaped depressions, which usually include mulch and a prepared soil mix. Typically, the filtered runoff is collected in a perforated underdrain beneath the bioretention system and returned to the storm drain system. For larger storms, runoff is generally diverted past the bioretention areas to the storm drain system.

The soil within a bioretention system should typically have an infiltration rate sufficient to draw down any pooled water within 48 hours after a storm event. Based on the "C.3 Stormwater Handbook" prepared by Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP, 2016), the infiltration rate of the soil should allow standing water to drain within 72 hours; cohesionless soils like sand meet this criterion. Cohesive soils like clay and silts do not meet the infiltration rate requirement and are considered unsuitable in a bioretention system, particularly when they are expansive. For areas where there are unsuitable in-situ soils, the bioretention system can be created by importing a suitable soil mix and providing an underdrain. Based on our observation of the soil at the site, the in-situ clays are relatively impervious will likely not meet the infiltration rate requirements. The bioretention systems will need to be constructed with suitable imported soil and include an underdrain system.

Underdrains are typically at the invert of the bioretention system to intercept water that does not infiltrate into the surrounding soils. Underdrains consist of a perforated PVC pipe surrounded by two to three inches of Class 2 Permeable material (Caltrans Standard Specifications Section 68-2.02F(3)). The perforated PVC pipe cross-section area should be determined based on the desired hydraulic conductivity of the underdrain. Underdrains should be installed in accordance with the Santa Clara County's C.3 stormwater technical guidelines.

Because of the presence of near surface expansive soil, unlined bioretention systems should be set back a minimum of five feet from building foundations, slabs, concrete flatwork, or pavements. If bioretention systems are closer than five feet, passive resistance of foundation elements should be neglected. Overflow from bioretention areas should be directed to the storm drain system away from building foundations and slabs.

In Santa Clara County, the bottom of the bioretention system is recommended to be a minimum of five feet or more above the groundwater table (SCVURPPP, 2016).

8.12 Construction Monitoring

The conditions of existing buildings and other improvements within 100 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction.

To monitor ground movements, groundwater levels, and shoring movements, we recommend installing survey points on the adjacent buildings and streets that are within 100 feet of the site. In addition, survey points should be installed at the tops of the shoring walls at 20-foot-spacing.

The survey points should be read regularly, and the results should be submitted to us in a timely manner for review.

Where critical structures or improvements will be supported behind the temporary shoring, periodic monitoring of survey points should be based on the minimum recommendations provided in Table 13.

TABLE 13
Recommended Minimum Frequency of Survey Point Monitoring

Stage	Critical Structure or Improvement	Non-Critical Structure or Improvement
Prior to Construction	Once (baseline)	Once (baseline)
During Installation of Shoring System	Weekly	--
During Excavation	Weekly	Weekly
Throughout Construction Until Structure is Above Street Level and Basement Walls are in Place	Monthly	Monthly

9.0 ADDITIONAL GEOTECHNICAL SERVICES

During final design we should be retained to consult with the design team as geotechnical questions arise. Prior to construction, we should review the project plans and specifications to check their conformance with the intent of our recommendations. We should also review shoring design and installation submittals. During construction, we should observe site preparation, excavation, shoring installation, tieback testing, compaction of fill and backfill, preparation of mat subgrade and subgrade of footing excavations. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractors' work conforms to the geotechnical aspects of the plans and specifications.

10.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings as well as architectural information provided by KPF. Actual subsurface conditions could vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others. Any proposed changes in structures, depths of excavation, or their locations should be brought to Langan’s attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation, as they may affect our recommendations.

This report has been prepared to assist the Owner, architect, and structural engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities on adjacent properties which are beyond the limits of that which is the specific subject of this report.

Environmental issues (such as permitting or potentially contaminated soil and groundwater) are outside the scope of this study and should be addressed in a separate evaluation.

REFERENCES

2014 Working Group on California Earthquake Probabilities. (2015). "UCERF3: A new Earthquake Forecast for California's Complex Fault System", U.S. Geological Survey 2015–3009. http://dx.doi.org/10.3133/fs20153009.

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

Abrahamson, N.A., Silva, W.J., and Kamai, R. (2014). Summary of the ASK14 ground-motion relation for active crustal regions: Earthquake Spectra, v. 30, n. 3, p. 1025-1055.

Abrahamson, N. A. (2000). "Effect of rupture directivity on probabilistic seismic hazard analysis." Proceedings of Sixth International Conference on Seismic Zonation, Palm Springs, California, November.

ASCE/SEI 7-10 (2010). Minimum Design Loads for Buildings and Other Structures.

ASCE/SEI 7-16 (2016). Minimum Design Loads for Buildings and Other Structures.

Bozorgnia, Y. and Campbell, K. W. (2004). "The vertical-to-horizontal response spectra ratio and tentative procedures for developing simplified V/H and vertical design spectra" Journal of Earthquake Engineering, 8(2), 175-207.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M. (2014). NGA-West 2 equations for predicting PGA, PGV, and 5%-damped PSA for shallow crustal earthquakes, Earthquake Spectra, v. 30, n. 3, p. 1057-1085.

California Building Standards Commission (2016). California Building Code.

California Building Standards Commission (2019). California Building Code.

California Department of Conservation Division of Mines and Geology (1997). *Guidelines for Evaluating and Mitigating Seismic Hazards in California*. Special Publication 117.

California Division of Mines and Geology (1996). *Probabilistic Seismic Hazard Assessment for the State of California*, CDMG Open-File Report 96-08.

California Division of Mines and Geology (1974). "State of California Special Studies Zones, Cupertino Quadrangle," prepared by the California Geologic Survey.

California Division of Mines and Geology (2002a). "Seismic Hazard Zone Report for the Cupertino 7.5-Minute Quadrangle, Santa Clara County, California, prepared by the California Geologic Survey," Seismic Hazard Zone Report 068.

REFERENCES
(Continued)

California Division of Mines and Geology (2002b). "State of California Seismic Hazard Zones, Cupertino Quadrangle," prepared by the California Geologic Survey.

Campbell, K.W., and Bozorgnia, Y. (2014). NGA-West2 ground motion model for the average horizontal components of PGA, PGV, and 5%-damped linear acceleration response spectra: Earthquake Spectra, v. 30, n. 3, p. 1087-1115.

Chiou, B.S.-J. and Youngs, R.R. (2014). Update of the Chiou and Youngs NGA model for the average horizontal component of peak ground motion and response spectra, Earthquake Spectra, v. 30, n. 3, p. 1117-1153.

Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J. (2003). "The Revised 2002 California Probabilistic Seismic Hazard Maps."

Chiou, B. S.-J., and Youngs, R. R. (2008). "An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra." Earthquake Spectra, 24(1), 173-215.

Cornell, C. A. (1968). "Engineering Seismic Risk Analysis." Bulletin of the Seismological Society of America, 58(5).

County of Santa Clara (2015). "Geologic Hazard Zones" Maps. Scale 1:24,000.

DCI Engineers (2020). "Vallco Town Center, Shoring and Mass Excavation Documents," dated 15 January 2020

Fugro Consultants (2019). "EZFRISK computer program." Version 8.06.

Hanks, T. C., and W. H. Bakun (2008). "M-log A observations of recent large earthquakes." Bulletin of Seismological Society of America, Vol. 98, No. 1, pp 490-503.

Holzer, T.L. et al. (2008). "Liquefaction Hazard Maps for Three Earthquake Scenarios for the Communities of San Jose, Campbell, Cupertino, Los Altos, Los Gatos, Milpitas, Mountain View, Palo Alto, Santa Clara, Saratoga and Sunnyvale, Northern Santa Clara County." USGS Open File Report 2008-1270.

Idriss, I.M. and Boulanger, R.W. (2008). "Soil Liquefaction During Earthquakes." Earthquake Engineering Research Institute. Monograph MNO-12.

Idriss, I. M. (1993). "Procedures for Selecting Earthquake Ground Motions at Rock Sites." National Institute of Standards and Technology, NIST GCR 93-625, 12 p. plus appendix.

Lazarte, C.A., Robinson, H., Gomez, J.E., Baxter, A., Cadden, A., and Berg, R., 2015, "Geotechnical Engineering Circular No. 7, Soil Nail Walls – Reference Manual," U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-14-017, March.

REFERENCES
(Continued)

Lienkaemper, J. J. (1992). “Map of recently active traces of the Hayward Fault, Alameda and Contra Costa counties, California.” Miscellaneous Field Studies Map MG-2196.

McGuire, R. K. (1976). “FORTRAN computer program for seismic risk analysis.” U.S. Geological Survey, Open-File Report 76-67.

Pradel, Daniel (1998). “Procedure to Evaluate Earthquake-Induced Settlements in Dry Sand,” Journal of Geotechnical and Geoenvironmental Engineering, April, and errata October 1998, pp1048.

Nichols, D.R., and N.A. Wright (1971). “Preliminary map of historic margins of marshland, San Francisco Bay, California: USGS Open-File-Report.

Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP) (2016). Guidance for Implementing Stormwater Requirements for New Development and Redevelopment Projects, June.

Sandis (2016). “The Hills at Vallco, Demolition Package, Cupertino, CA, Topography Survey,” Sheets CD.00.01.00, CD.02.01.01, CD.02.01.02, CD.01.01.03 through CD.01.01.07, dated 9/20/16.

Seed, H.B. and Idriss, I.M. (1971). “Simplified Procedure for Evaluating Soil Liquefaction during Earthquakes,” Journal of Geotechnical Engineering Division, ASCE, 97 (9), 1249-1273.

Seed, H.B., R.B. Seed, L.F. Harder and H.L. Jong, 1988, “Re-evaluation of the Slide in the Lower San Fernando Dam in the Earthquake of February 9, 1971.” Report No. UCB/EERC-88/04, University of California, Berkeley, April.

Shahi, S. K. and Baker J. W. (2014). “NGA-West 2 Models for Ground Motion Directionality.” *Earthquake Spectra*. Volume 30. No. 3. Pages 1285-1300.

Sitar, N., E.G. Cahill and J.R. Cahill (2012). “Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls.”

Treadwell & Rollo (2013). “Geotechnical Data Report, Pruneridge Theatre Site, Cupertino, California.”

Tokimatsu, K. and Seed, H.B. (1987). “Evaluation of Settlements in Sand due to Earthquake Shaking.” Journal of Geotechnical Engineering, Vol. 113, No. 8, pp. 861-878.

Toppozada, T. R. and Borchardt G. (1998). “Re-Evaluation of the 1836 “Hayward Fault” and the 1838 San Andreas Fault earthquakes.” Bulletin of Seismological Society of America, 88(1), 140 159.

REFERENCES
(Continued)

Townley, S. D. and Allen, M. W. (1939). "Descriptive catalog of earthquakes of the Pacific coast of the United States 1769 to 1928." Bulletin of the Seismological Society of America, 29(1).

TRC (2015). "Preliminary Geotechnical Investigation, The Hills at Vallco, Cupertino, California." Report Number 228550.

Wells, D. L. and Coppersmith, K. J. (1994). "New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement." Bulletin of the Seismological Society of America, 84(4), 974-1002.

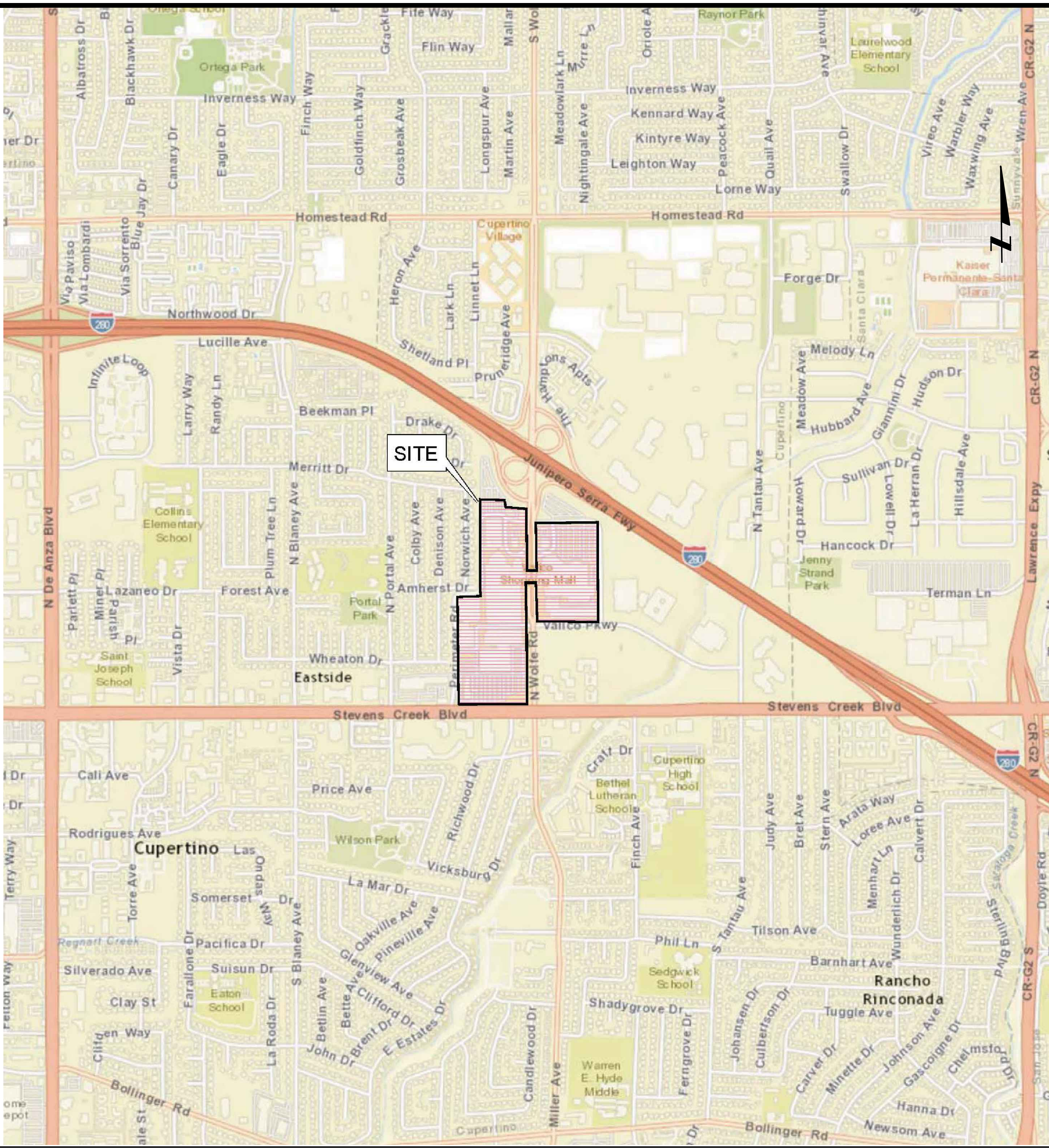
Wesnousky, S. G. (1986). "Earthquakes, Quaternary Faults, and Seismic Hazards in California." Journal of Geophysical Research, 91(1312).

Youd, T.L., and Idriss, I.M. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 4.

Youd, T.L., and Garris, C.T. (1995). "Liquefaction-induced ground-surface disruption." Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 121, 805-809.

Youngs, R. R., and Coppersmith, K. J. (1985). "Implications of fault slip rates and earthquake recurrence models to probabilistic seismic hazard estimates." Bulletin of the Seismological Society of America, 75( ), 939-964.

FIGURES

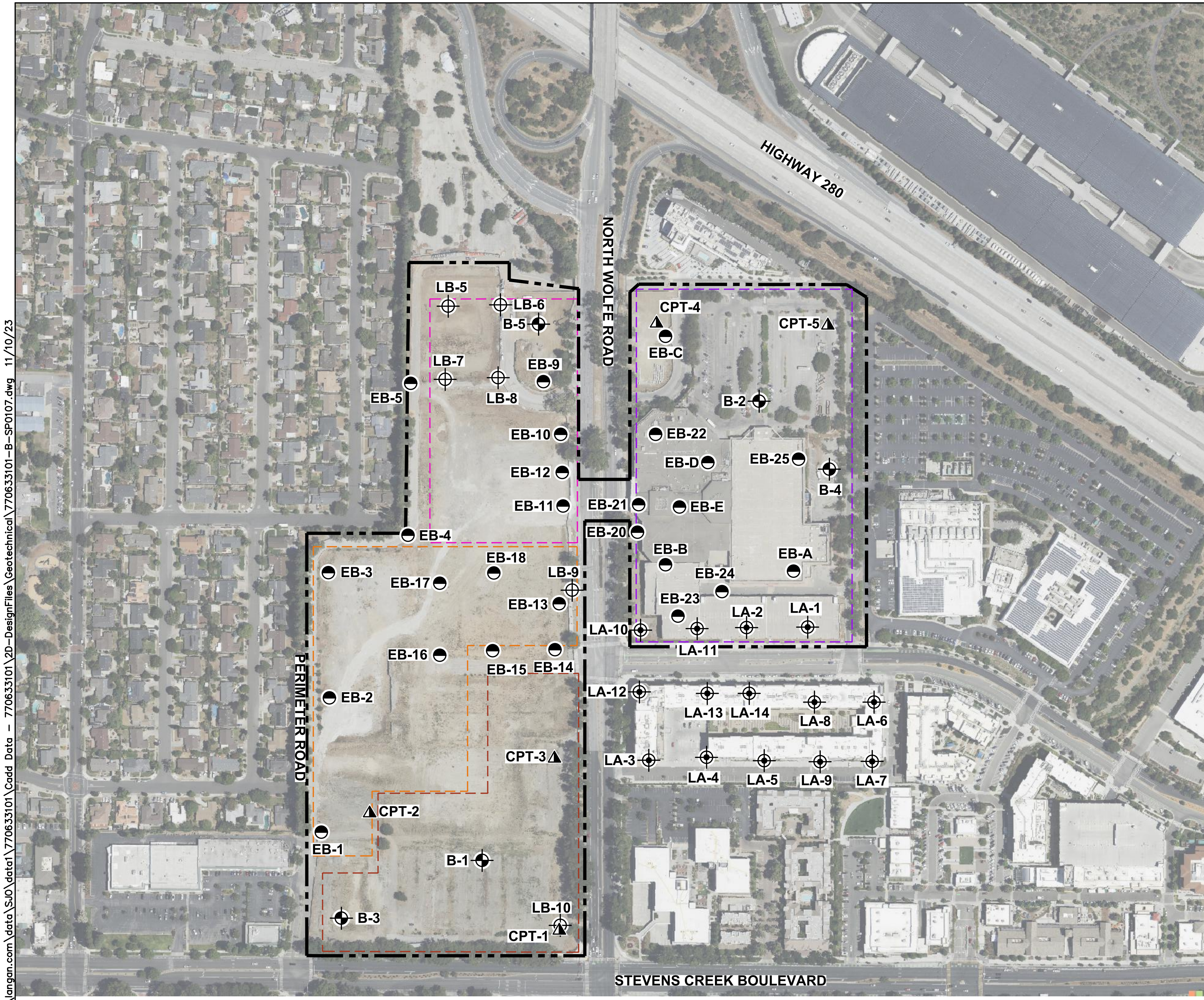


Note:
Base map is provided through Langan's Esri Arc GIS software licensing and Arc GIS online, National Geographic Society, i-cubed.

0 2,000 Feet

Approximate scale

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project **THE RISE** **CUPERTINO** **SANTA CLARA COUNTY CALIFORNIA**	Figure Title **SITE LOCATION MAP**	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure 1



EXPLANATION

B-1 Approximate location of boring by Langan, September 2016

CPT-1 Approximate location of cone penetration test by Langan, September 2016

LB-5 Approximate location of boring by Lowney Associates, 2005

LA-1 Approximate location of boring by Lowney Associates, 1999

EB-1 EB-A Approximate location of boring by Lowney-Kaldveer Associates, 1974

Approximate site boundary

Approximate Phase 1 Extents

Approximate Phase 2 Extents

Approximate Phase 3 Extents

Approximate Phase 4 Extents

Reference: Phase extents based on plan titled "Parking Basements, By Phase - Alternate Scheme" by KPF dated 15 August 2023.
Aerial photo taken from latest 2023 Bing Maps.

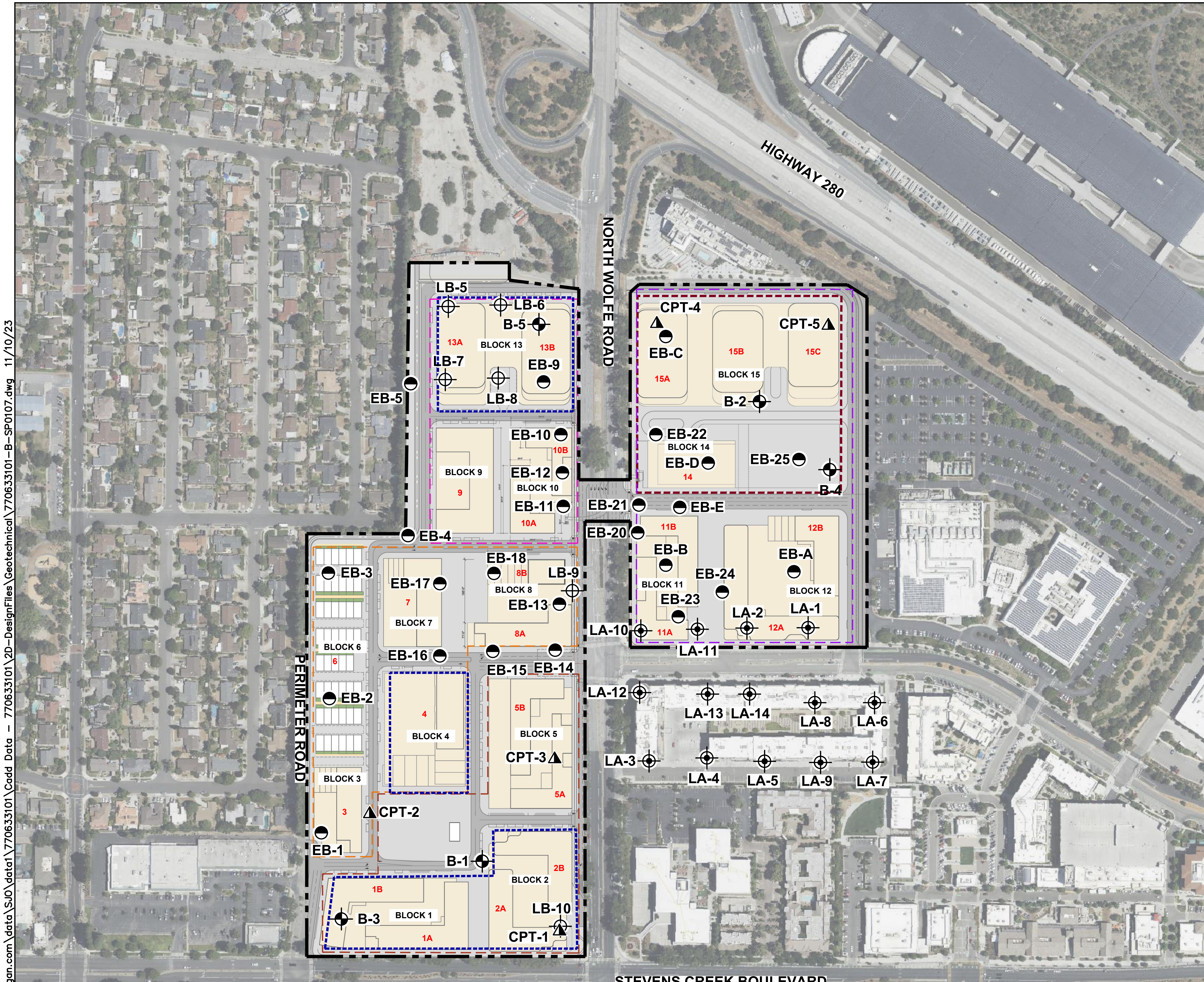
THE RISE
Cupertino, California

SITE PLAN WITH PROJECT PHASING

Date 09/27/23	Project No. 770633101	Figure 2

LANGAN

\\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B-SP0107.dwg 11/10/23



EXPLANATION

B-1 — Approximate location of boring by Langan, September 2016

CPT-1 — Approximate location of cone penetration test by Langan, September 2016

LB-5 — Approximate location of boring by Lowney Associates, 2005

LA-1 — Approximate location of boring by Lowney Associates, 1999

EB-1 / EB-A — Approximate location of boring by Lowney-Kaldveer Associates, 1974

Approximate site boundary

Approximate footprint of one-level basement

Approximate footprint of three-level basement

Approximate Phase 1 Extents

Approximate Phase 2 Extents

Approximate Phase 3 Extents

Approximate Phase 4 Extents

1A — Building numbers

Reference: Proposed development based on roof plan by KPF dated 3 November 2023.
Basement footprint based on basement plan by KPF dated 3 November 2023.
Phase extents based on plan titled "Parking Basements, By Phase - Alternate Scheme" by KPF dated 15 August 2023.
Aerial photo taken from latest 2023 Bing Maps.

0 300 Feet
Approximate scale

THE RISE
Cupertino, California

SITE PLAN WITH PROPOSED DEVELOPMENT

Date 11/07/23	Project No. 770633101	Figure 3

LANGAN

\\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B-SP0107.dwg 11/10/23

Notes:
1. See Figures 2 and 3 for location of subsurface profiles
2. The above profile represents a generalized soil cross section interpreted from widely spaced borings and CPTs. Soil deposits may vary in type, strength, and other important properties between points of exploration.
3. Existing ground surface base on Topographic Survey by Sandis, dated November 2015.
4. Lowney Kaldveer Associates borings designated as "EB". Lowney Associates borings designated as "LB".
5. Year of drilling or CPT noted in parentheses.

20
0
0 50
SCALE IN FEET

THE RISE
Cupertino, California

IDEALIZED SUBSURFACE PROFILE A-A'

Date 11/07/23	Project No. 770633101	Figure 4

LANGAN

\\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B-XS0101.dwg 11/7/23

Notes:
1. See Figures 2 and 3 for location of subsurface profiles
2. The above profile represents a generalized soil cross section interpreted from widely spaced borings and CPTs. Soil deposits may vary in type, strength, and other important properties between points of exploration.
3. Existing ground surface base on Topographic Survey by Sandis, dated November 2015.
4. Lowney Kaldveer Associates borings designated as "EB". Lowney Associates borings designated as "LB".
5. Year of drilling or CPT noted in parentheses.

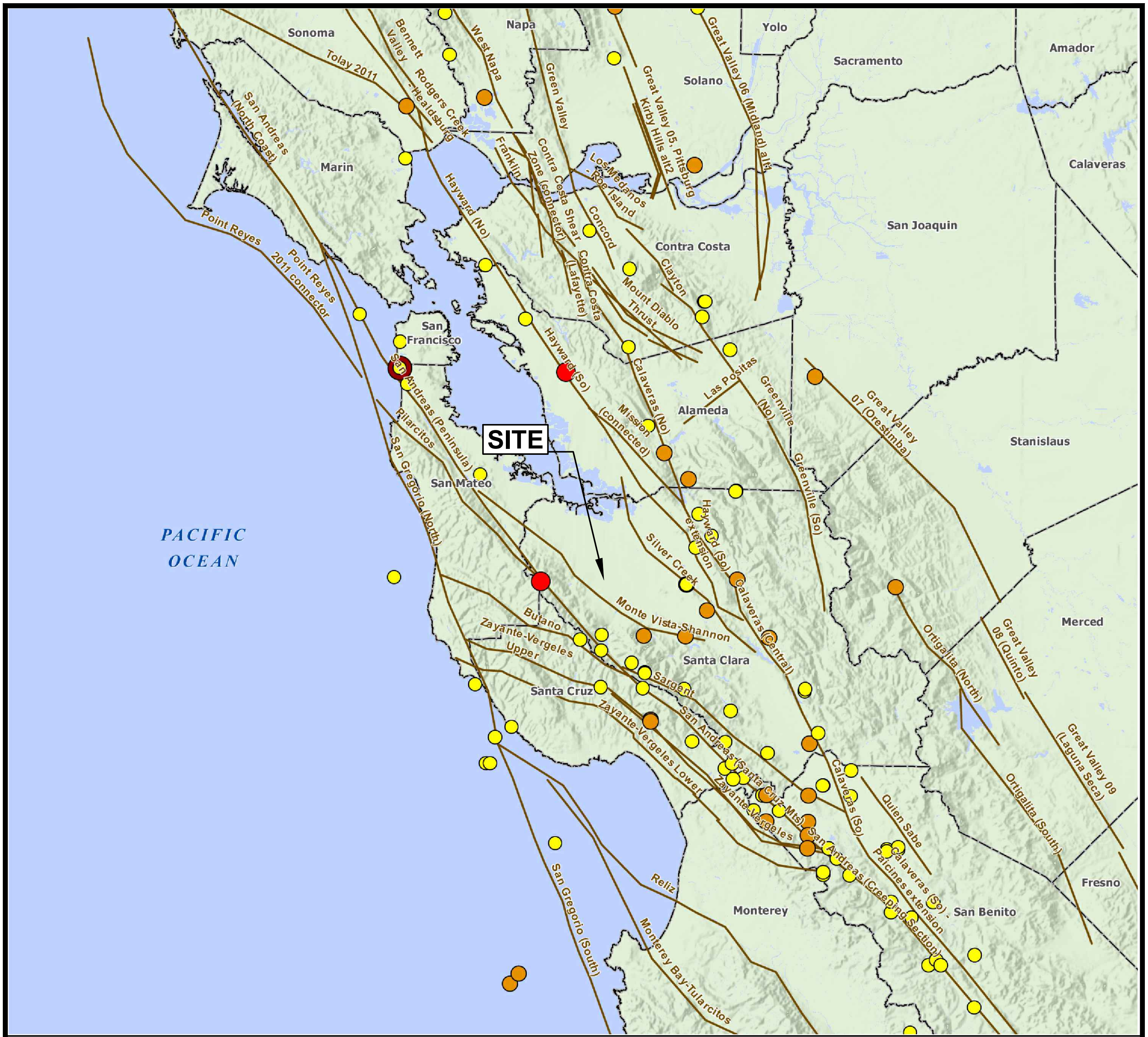
THE RISE
Cupertino, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 11/07/23	Project No. 770633101	Figure 5

LANGAN

\\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B-XS0101.dwg 11/7/23



LEGEND

County Boundary

Fault

Earthquake Epicenter Magnitude

- Magnitude 5 to 5.9
- Magnitude 6 to 6.9
- Magnitude 7 to 7.4
- Magnitude 7.5 to 8

0 20 km

Approximate scale

Notes:

1. Quaternary fault data displayed are provided by the CGS Map Sheet 48: Fault based seismic sources used in the Uniform California Earthquake Rupture Forecast, Version (UCERF3).
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 2015.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No. / Date / Drawn By / Checked By	Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	6

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B-GI0101_SiteLocMap.dwg Date: 11/15/2023 Time: 09:02 User: agekas Style Table: Langan.stb Layout: Figure 6 FaultMap

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

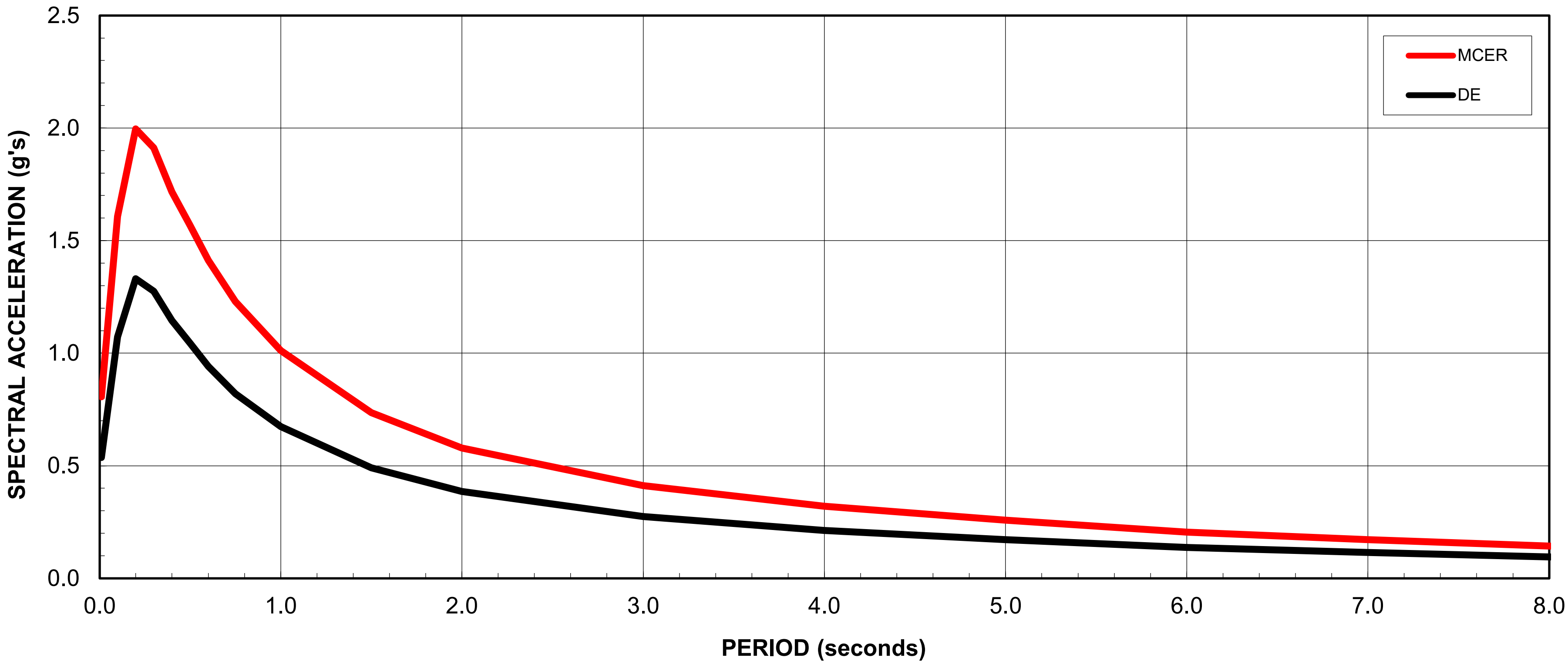
X Panic is general.
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No.	Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	MODIFIED MERCALLI INTENSITY SCALE	770633101 Date: 11/07/2023 Drawn By: AG Checked By: JF	7



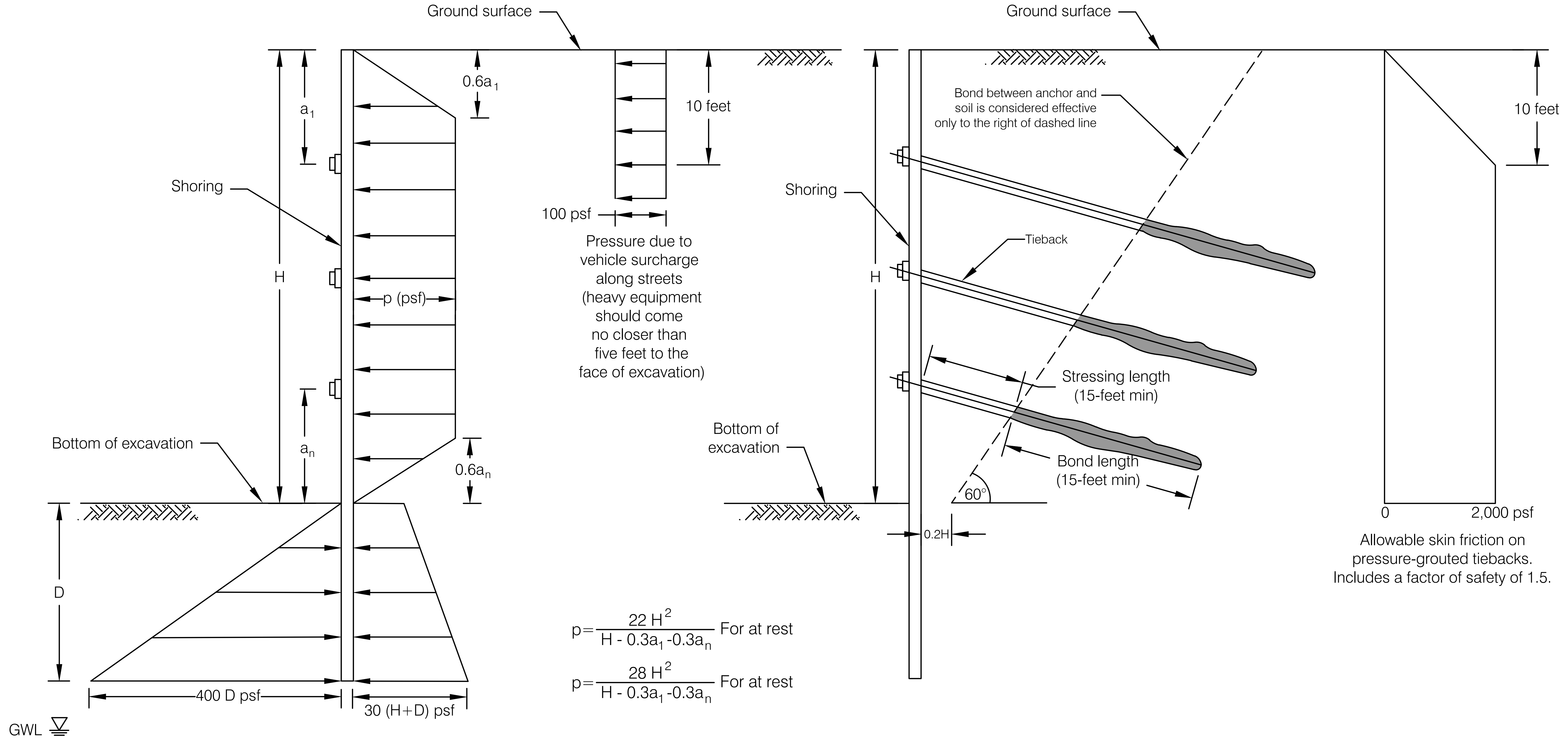
Damping Ratio = 5%

THE RISE
Cupertino, California

RECOMMENDED SPECTRA

Date 10/06/20	Project No. 770633101	Figure 8

LANGAN



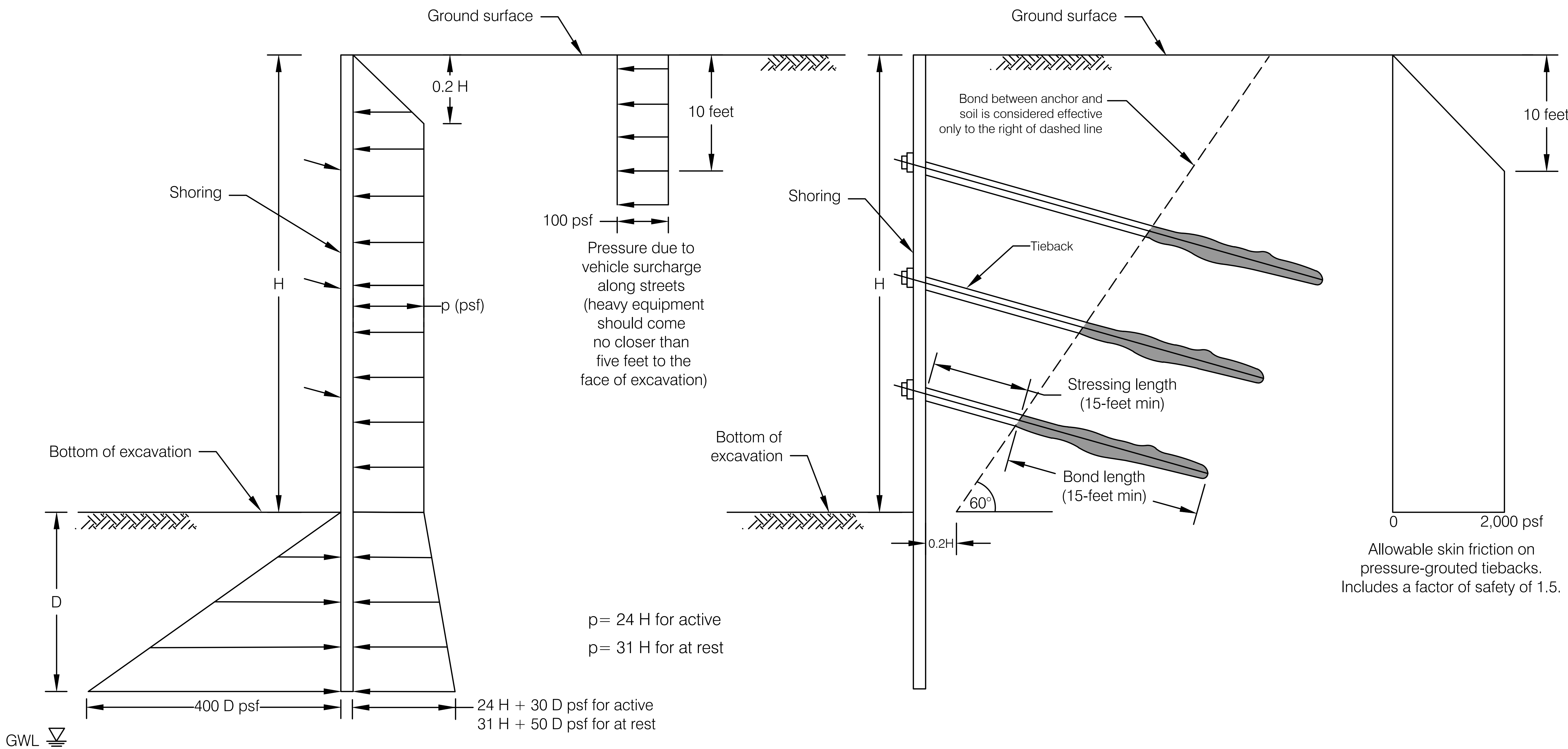
$$p = \frac{22\,H^2}{H - 0.3a_1 - 0.3a_n} \text{ For at rest}$$

$$p = \frac{28\,H^2}{H - 0.3a_1 - 0.3a_n} \text{ For at rest}$$

NOT TO SCALE

Notes:
1. Passive pressure includes a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Active pressure below the excavation should be assumed to act over one pile diameter.
4. Where the shoring is adjacent to buildings, the shoring should be designed for the additional building surcharge loads presented on Figures 11 through 13.

LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING SHORING SYSTEM	770633101 Date 11/07/2023 Drawn By AG Checked By WW	9

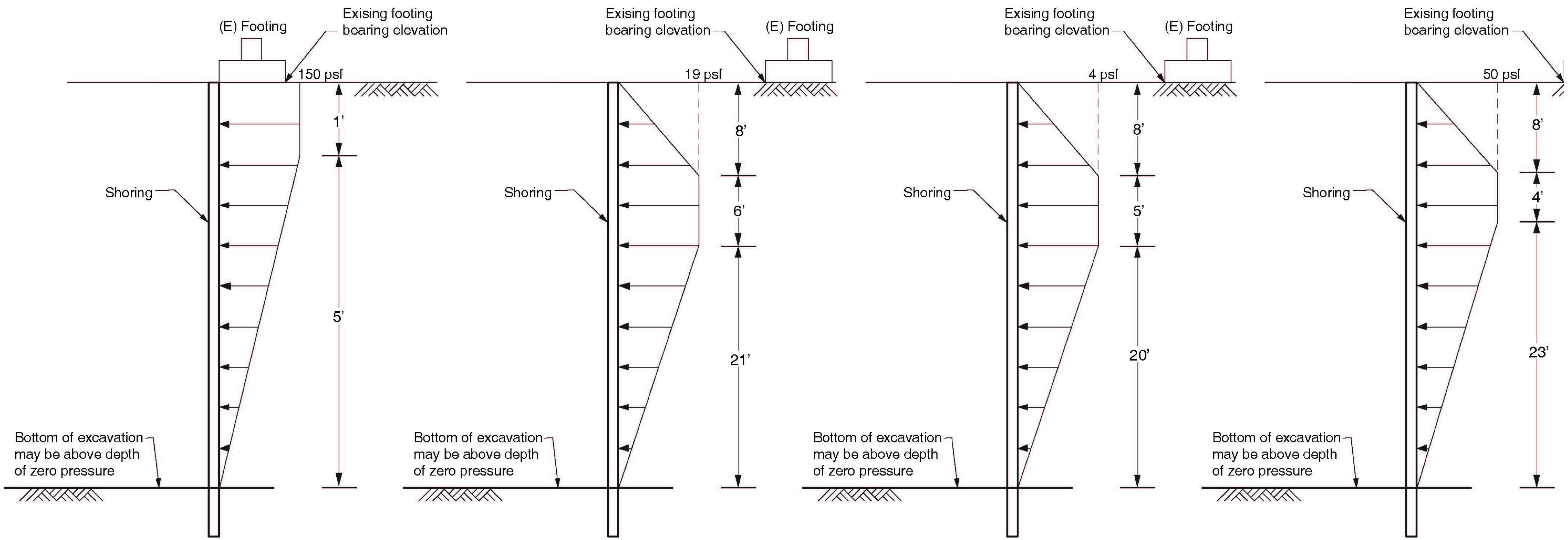


NOT TO SCALE

Notes:

1. Passive pressure includes a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Active pressure below the excavation should be assumed to act over one pile diameter.
4. Where the shoring is adjacent to buildings, the shoring should be designed for the additional building surcharge loads presented on Figures 11 through 13.

LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	DESIGN PARAMETERS FOR SOLDIER-PILE-AND-SOIL-CEMENT SHORING SYSTEM	770633101 Date 11/07/2023 Drawn By AG Checked By WW	10



CASE A
4-FOOT SQUARE FOOTING
AT SHORING

CASE B
9-FOOT 9-INCH SQUARE FOOTING
AT 14-FEET FROM SHORING

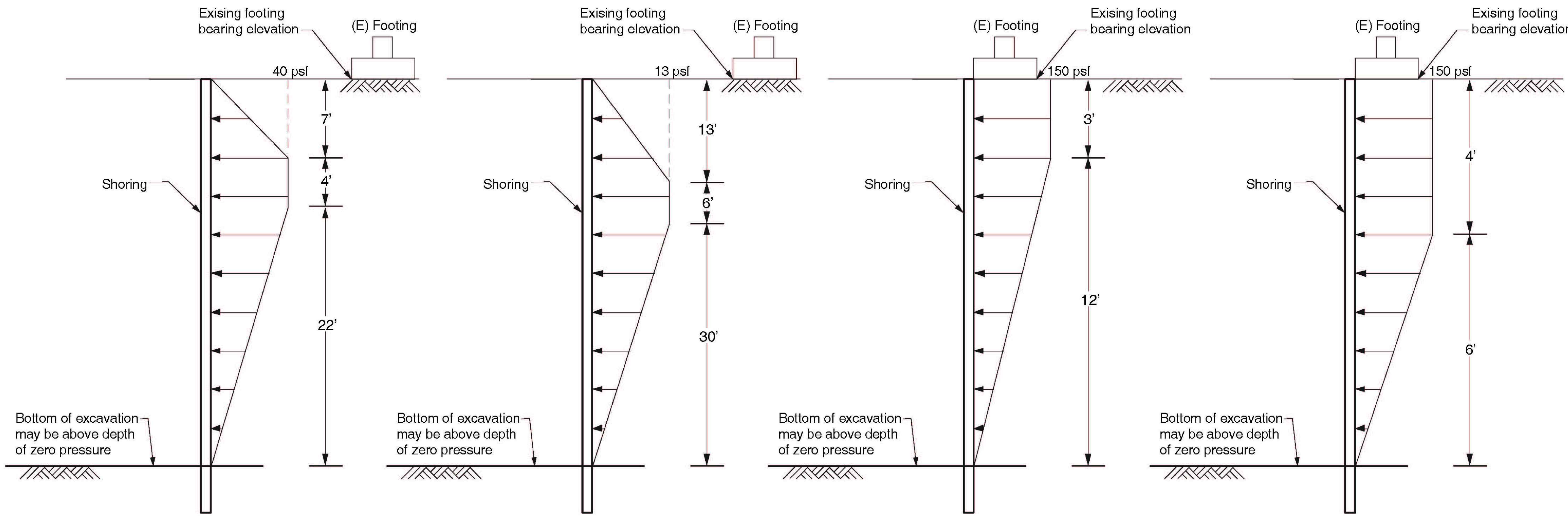
CASE C
4-FOOT SQUARE FOOTING
AT 16-FEET FROM SHORING

CASE D
16-FOOT SQUARE FOOTING
AT 11-FEET FROM SHORING

NOT TO SCALE

Note:
1. Horizontal pressures calculated based on 1 ksf uniform bearing pressure from footing.
2. Apply surcharge pressures over a distance of 14 feet from either side of the footing.

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title SURCHARGE PRESSURE FROM EXISTING FOOTING ON PROPOSED SHORING CASE A THROUGH D	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By WW	Figure 11



CASE E
12-FOOT 9-INCH SQUARE FOOTING
AT 10-FEET FROM SHORING

CASE F
12-FOOT SQUARE FOOTING
AT 22-FEET FROM SHORING

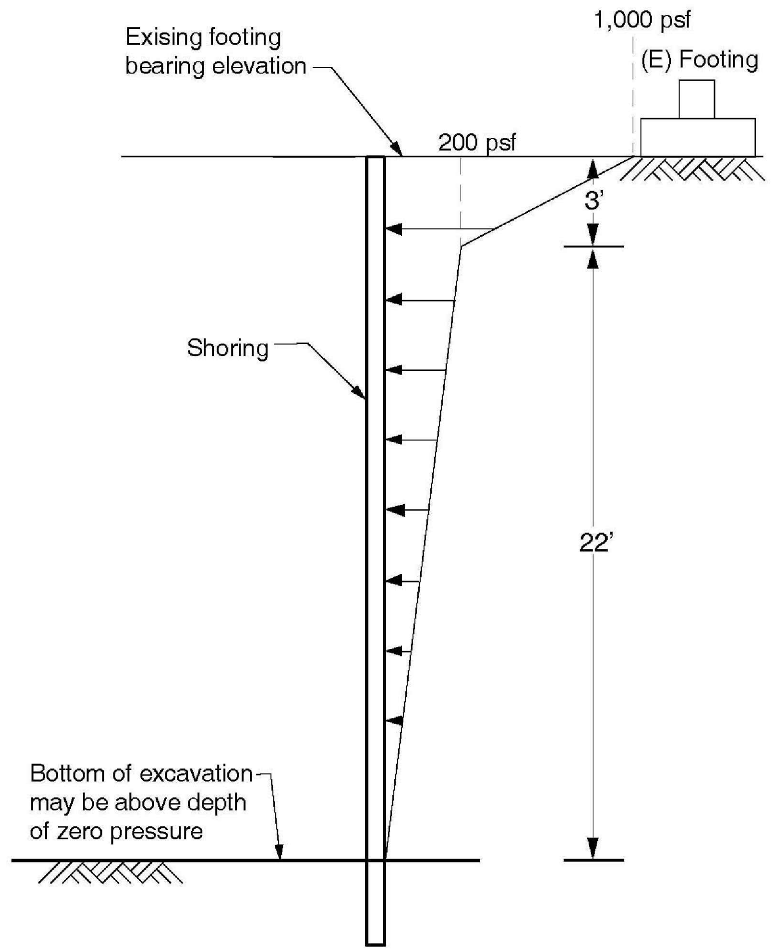
CASE G
9-FOOT 9-INCH SQUARE FOOTING
AT SHORING

CASE H
6-FOOT STRIP FOOTING
AT SHORING

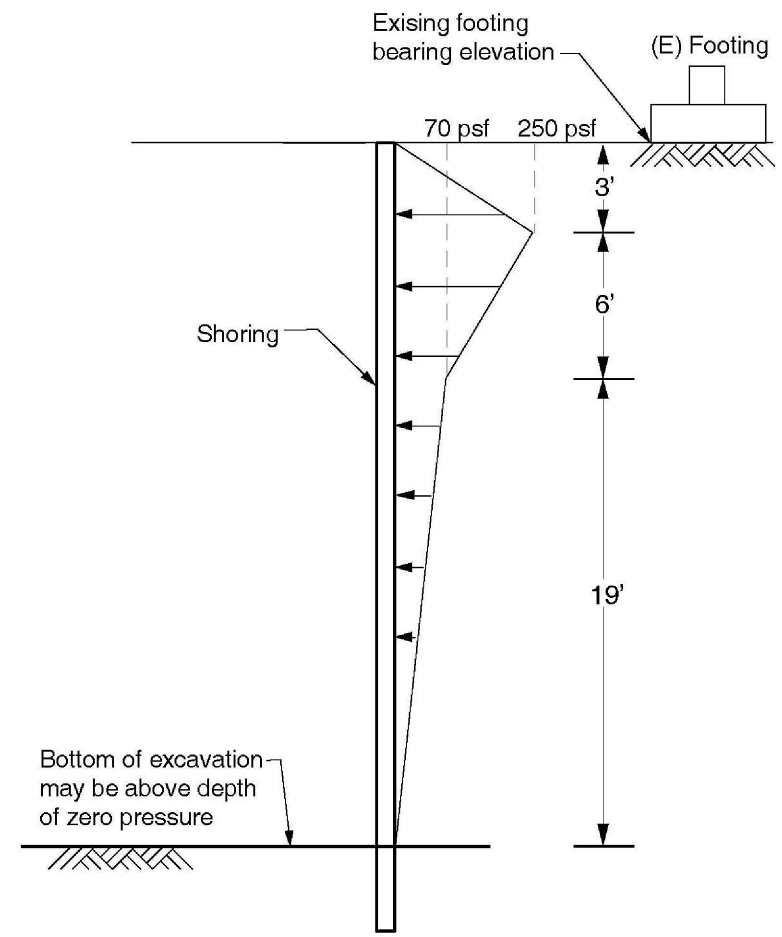
NOT TO SCALE

Note:
1. Horizontal pressures calculated based on 1 ksf uniform bearing pressure from footing.
2. Apply surcharge pressures over a distance of 14 feet from either side of the footing.

LANGAN	Project	Figure Title	Project No. 770633101	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	SURCHARGE PRESSURE FROM EXISTING FOOTING ON PROPOSED SHORING CASE E THROUGH H	Date 11/07/2023 Drawn By AG Checked By WW	12



CASE I
9-FOOT 9-INCH SQUARE FOOTING
CENTERED AT 10-FEET FROM SHORING



CASE J
6-FOOT STRIP FOOTING
AT 1-FOOT FROM SHORING

NOT TO SCALE

Note:
1. Horizontal pressures calculated based on 1 ksf uniform bearing pressure from footing.
2. Apply surcharge pressures over a distance of 14 feet from either side of the footing.

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No.	Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	SURCHARGE PRESSURE FROM EXISTING FOOTING ON PROPOSED SHORING CASE I AND J	770633101 Date: 11/07/2023 Drawn By: AG Checked By: WW	13

APPENDIX A

BORING LOGS AND LABORATORY TEST RESULTS FROM PREVIOUS INVESTIGATIONS

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

D&M UNDERWATER SAMPLER

SHELBY 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)

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE A–1

EXPLORATORY BORING: EB-5

Sheet 1 of 1

DRILL RIG: MOBILE B-53	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-3-04 FINISH DATE: 8-3-04	COMPLETION DEPTH: 25.0 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

SURFACE ELEVATION: 178 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
178.0	0	6 inches asphalt concrete						
177.5		**SANDY LEAN CLAY (CL)** hard, moist, brown, fine sand, some fine gravel, low plasticity	CL	54	9	117		○
				30	8	113		○
	5			52	14	115		○
				31	11	109		○
	10							
	15			42				
160.0		**CLAYEY SAND (SC)** dense, moist, brown, fine to coarse sand	SC					
158.8		**LEAN CLAY (CL)** hard, moist, brown, some fine sand, low plasticity	CL	61	10	117		○
	20							
156.0		**POORLY GRADED SAND WITH CLAY (SP-SC)** very dense, moist, brown, fine sand, some medium to coarse sand, some fine gravel	SP-SC	59	3			
153.0	25	Bottom of Boring at 25 feet						
	30							

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

Northing: 1,945,612
Easting: 6,120,917

LA CORP.GDT 2/11/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-6

Sheet 1 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-3-04 FINISH DATE: 8-3-04	COMPLETION DEPTH: 34.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
176.0	0	SURFACE ELEVATION: 176 FT. (+/-)					
175.9		1½ inches asphalt concrete over 3½ inches aggregate base					
175.5		**LEAN CLAY WITH SAND (CL) [FILL]** stiff, moist, olive green, fine sand, moderate to high plasticity	CL, FILL	16	12	120	
174.0		**LEAN CLAY WITH SAND (CL)** hard, moist, brown, fine sand, some fine gravel, low plasticity	CL	15	15	118	
	5			40	14	114	
167.0	10	**CLAYEY SAND (SC)** loose, moist, brown, fine to medium sand, some coarse sand	SC	7	12	94	42
		medium dense		26	9		
161.0	15	**SANDY LEAN CLAY (CL)** very stiff, moist, brown, fine to coarse sand, low plasticity	CL	19	14		
156.8	20	**LEAN CLAY WITH SAND (CL)** very stiff, moist, brown, fine sand, low plasticity	CL	25	18	106	
153.5	25	**CLAYEY SAND (SC)** medium dense, moist, brown, fine to coarse sand, some fine gravel	SC	65	7	122	
149.0		**LEAN CLAY (CL)** very stiff, moist, brown, some fine sand, low plasticity	CL	35	23	98	
146.8			SP-SM				
146.0	30						

Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

Northing: 1,945,590
Easting: 6,121,038

LA CORP.GDT 2/11/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-6 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-3-04 FINISH DATE: 8-3-04	COMPLETION DEPTH: 34.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

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)
146.0	30		**POORLY GRADED SAND WITH SILT (SP-SM)** medium dense, moist, brown, fine to medium sand, some fine gravel	SP-SM						
143.0			**CLAYEY SAND WITH GRAVEL (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse gravel	SC	50/6"	X				
141.5	35		Bottom of Boring at 34½ feet							
	40									
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

Northing: 1,945,590
Easting: 6,121,038

LA CORP.GDT 2/11/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-7

Sheet 1 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-3-04 FINISH DATE: 8-3-04	COMPLETION DEPTH: 35.0 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

SURFACE ELEVATION: 182 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
182.0 / 181.9 / 181.5	0	1½ inches asphalt concrete over 3½ inches aggregate base					
		SANDY LEAN CLAY (CL) hard, moist, brown, fine to coarse sand, some fine gravel, low plasticity	CL	40	9	125	
				42	7	111	
	5			24	6	107	
	10			19	9	96	
	15			34	7	106	
165.0		**CLAYEY SAND (SC)** medium dense, moist, light brown, fine sand, some fine gravel	SC				
162.8	20	**SANDY LEAN CLAY (CL)** hard, moist, brown, fine sand, some medium to coarse sand, some fine and coarse gravel, low plasticity	CL	40	10	112	
160.0	25	**LEAN CLAY WITH SAND (CL)** hard, moist, brown, fine to medium sand, low plasticity	CL	40	15	112	
155.0 / 152.0	30	**LEAN CLAY (CL)** hard, moist, brown, some fine sand, low plasticity	CL	46	23	106	

Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

Northing: 1,945,434
Easting: 6,120,918

LA CORP.GDT 2/11/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-7 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-3-04 FINISH DATE: 8-3-04	COMPLETION DEPTH: 35.0 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

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)
152.0	30		**LEAN CLAY (CL)** hard, moist, brown, some fine sand, low plasticity	CL						
150.5			**CLAYEY SAND (SC)** medium dense, moist, brown, fine sand	SC						
148.0			**LEAN CLAY (CL)** very stiff, moist, brown, some fine sand, low plasticity	CL	29		25	98		○ (approx. 3.3)
147.0	35		Bottom of Boring at 35 feet							
	40									
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

Northing: 1,945,434
Easting: 6,120,918

LA CORP.GDT 2/11/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-8

Sheet 1 of 1

DRILL RIG: MOBILE B-53	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-3-04 FINISH DATE: 8-3-04	COMPLETION DEPTH: 16.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

SURFACE ELEVATION: 182 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
182.0 / 181.8 / 181.5	0	2 inches asphalt concrete over 3½ inches aggregate base					
		LEAN CLAY (CL) [FILL] stiff, moist, olive green, trace fine sand, some organics, moderate to high plasticity	CL, FILL	15	17	98	
				16	22	104	
177.3	5	**LEAN CLAY WITH SAND (CL)** very stiff, moist, dark brown to bown, fine to medium sand, trace fine gravel, low plasticity	CL	21	14	113	
	10	hard		20	14	117	
169.5		**SANDY LEAN CLAY (CL)** very stiff, moist, brown, fine sand, low plasticity	CL	16	11	103	55
	15			15			
165.5		Bottom of Borinng at 16½ feet					
	20						
	25						
	30						

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

Northing: 1,945,431
Easting: 6,121,039

LA CORP.GDT 2/17/05 MV° FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9

Sheet 1 of 3

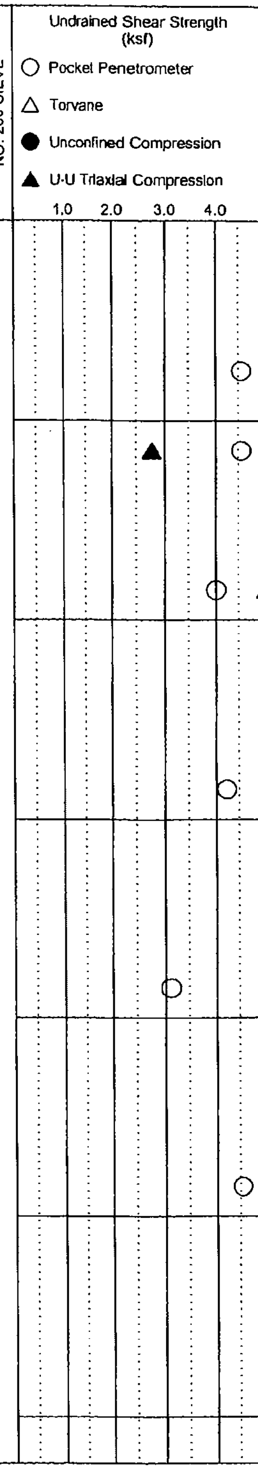
DRILL RIG: MOBILE B-61	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-4-04 FINISH DATE: 8-4-04	COMPLETION DEPTH: 84.5 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 177 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
177.0	0						
176.7		3 inches asphalt concrete over 4 inches aggregate base					
176.4		**LEAN CLAY WITH SAND (CL)** hard, moist, brown, fine to medium sand, some fine gravel, low to moderate plasticity	CL	41	12	101	
				51	14	111	
172.0	5	**LEAN CLAY (CL)** hard, moist, brown, some fine sand, trace fine gravel, low plasticity	CL	52	19	109	
				36	19	99	
	10						
				32	21	102	
	15	sandier					
				39	17	105	
	20	very stiff					
				34	15	115	
	25						
150.0		**CLAYEY SAND WITH GRAVEL (SC)** medium dense, moist, brown, fine to medium sand, some fine gravel	SC				
				36	10		
147.0	30						



Continued Next Page

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 68.0 FEET

LA CORP.GDT 2/17/05 MV* FLL

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9 Cont'd

DRILL RIG: MOBILE B-61	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-4-04 FINISH DATE: 8-4-04	COMPLETION DEPTH: 84.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
147.0	30	**CLAYEY SAND WITH GRAVEL (SC)** medium dense, moist, brown, fine to medium sand, some fine gravel	SC					
145.0		**POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC)** dense, moist, brown, medium to coarse sand, some fine sand, fine to coarse gravel	SP-SC	42	4		9	
139.0		**CLAYEY SAND WITH GRAVEL (SC)** dense to very dense, moist, brown, fine to coarse sand, fine gravel, some coarse gravel	SC	75	5			
	45			39	7			
				62	7		14	
	55			36	8			
118.0			CL	18	22			○ (approx. 4.5)
117.0	60							

Continued Next Page

GROUND WATER OBSERVATIONS:

∇: FREE GROUND WATER MEASURED DURING DRILLING AT 68.0 FEET

LA CORP.GDT 2/17/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9 Cont'd

Sheet 3 of 3

DRILL RIG: MOBILE B-61	PROJECT NO: 259-5E
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: VALLCO
LOGGED BY: BM	LOCATION: CUPERTINO, CA
START DATE: 8-4-04 FINISH DATE: 8-4-04	COMPLETION DEPTH: 84.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
117.0	60	**LEAN CLAY (CL)** hard, moist, brown, some fine sand, low plasticity	CL					
116.0		**CLAYEY SAND WITH GRAVEL (SC)** very dense, moist, brown, fine to coarse sand, fine gravel	SC					
				54	11		16	
	65							
	(68.0 ▽)							
				27	25			○ (~2.8)
107.5	70	**LEAN CLAY (CL)** very stiff to hard, moist, brown, some fine sand, low plasticity	CL					
				50	17	116		○ (>4.0)
102.3	75	**CLAYEY SAND WITH GRAVEL (SC)** dense to very dense, moist, brown, fine to coarse sand, fine to coarse gravel	SC					
				50/6"	8	125		
	80							
				50/6"				
92.5	85	Bottom of Boring at 84½						
	90							

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 68.0 FEET

LA CORP.GDT 2/17/05 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-10

Sheet 1 of 1

DRILL RIG: MOBILE B-61
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: BM
START DATE: 8-4-04 FINISH DATE: 8-4-04

PROJECT NO: 259-5E
PROJECT: VALLCO
LOCATION: CUPERTINO, CA
COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 189 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
189.0 / 188.8 / 188.6	0	1½ inches asphalt concrete over 3 inches aggregate base						
		CLAYEY SAND WITH GRAVEL (SC) loose, moist, brown, fine to medium sand, fine gravel	SC	8	14	98		
				6	9	100	23	
	5	medium dense		15				
179.5				17				○ (>4.0)
	10	**LEAN CLAY WITH SAND (CL)** hard, moist, brown, fine sand, trace fine gravel, low plasticity	CL					
	15			73	16	113		○ (>4.0)
169.0	20			51	11	113		○ (>4.0)
		Bottom of Boring at 20 feet						
	25							
	30							

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 2/11/05 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-1

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-17-99 FINISH DATE: 5-17-99	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 179.0 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
179.0	0	3 inches asphaltic concrete over 10 inches aggregate base					
178.7 / 177.9		SILTY CLAY (CL) very stiff, moist, brown, trace subrounded gravel to 3/4 inch, mottled gray, trace rootlets	CL	27	23	106	
				22	26	98	
	5	trace fine to medium sand		31	24	102	
	10			44	15	113	
167.0		SILTY SAND (SM) medium dense, moist, fine to coarse grained, occasional fine to medium subrounded gravel Estimated angle of interior friction: 37°-42°	SM	41	11		
164.0	15	SILTY CLAY (CL) very stiff, moist, brown, low plasticity	CL	18	21		
	20						
155.5	25	SILTY SAND (SM) very dense, moist, fine to medium grained, some coarse sand to fine sand, occasional subrounded sandstone fragments to 3/4 inch Estimated angle of internal friction: >42°	SM	50/4"	4		
152.5		SILTY CLAY (CL) very stiff, moist, orange-brown, low plasticity	CL	22	21		
149.0	30	Bottom of Boring at 30 feet					
	35						

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-17-99 FINISH DATE: 5-17-99	COMPLETION DEPTH: 29.5 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 180.0 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0 2.0 3.0 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
180.0 – 179.7 – 178.9	0	3 inches asphaltic concrete over 10 inches aggregate base	
178.9 – 164.5		SILTY CLAY (CL) very stiff to hard, moist, brown, low to moderate plasticity, trace rootlets, mottled black	CL
164.5 – 157.0	15	SILTY SAND (SM) medium dense, moist, orange brown, with some gravel, occasional subrounded to subangular sandstone fragments up to 3/4 inch. Estimated angle of internal friction: 33°-38°	SM
157.0 – 150.5		GRAVELLY SAND (SP-SM) dense, moist, brown, some silt, trace clayey sand seams, gravel to 1 1/2 inch. Estimated angle of internal friction: >42°; very dense	SP-SM
150.5	29.5	Bottom of Boring at 29 1/2 feet	

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
22	21	107	
35	23	106	
24	19	115	
60	19	113	
52	21	105	
14	6		
48	3		7
50/6"	4		

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-3

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-17-99 FINISH DATE: 5-17-99	COMPLETION DEPTH: 29.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
173.5	0	SURFACE ELEVATION: 173.5 FT. (+/-)						
173.2		3 inches asphaltic concrete over 10 inches aggregate base						
172.4		SANDY CLAY (CL) hard, moist, orange brown, subrounded gravel, low plasticity, occasional thin clayey sand lense	CL	53	11	115		>○
				48	10	114	17	>○
	5			33	11	109		>○
				42	7			
	10							
161.0		SILTY CLAY (CL) hard, moist, orange brown, trace to some fine sand, low plasticity	CL	30	19			
	15							
156.5		CLAYEY SAND (SC) dense, moist, orange-brown, fine to coarse grained, trace silt, occasional subangular gravel	SC	53	10	125		○
	20							
151.5		SILTY SAND (SM) very dense, moist, gray brown, fine to coarse grained, occasional subrounded to subangular gravel to 1 1/4 inch	SM	54	8			
148.5	25	Estimated angle of internal friction: >40° Bottom of Boring at 25 feet						
	30							
	35							

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-17-99 FINISH DATE: 5-17-99	COMPLETION DEPTH: 35.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 173.0 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
173.0 – 172.7 – 171.9	0	3 inches asphaltic concrete over 10 inches aggregate base	
171.9 – 165.5		SANDY CLAY (CL) very stiff to stiff, moist, orange brown, fine sand, some silt, trace subangular gravel, low plasticity; increase fine sand (at ~5 ft)	CL
165.5 – 140.0		SILTY SAND (SM) dense, moist, brown, subrounded gravel to 1 inch, trace to some clay. Estimated angle of internal friction: 37°-44°	SM
140.0 – 138.0	35	SILTY CLAY (CL) very stiff, moist, orange-brown, some fine sand, low plasticity	CL
138.0	35	Bottom of Boring at 35 feet	

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
24	19		53
27	27		
13	25		
43	8		
50/5"	6		
31	5		10
46	7		
43	5		
28	21	105	

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-5

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-17-99 FINISH DATE: 5-17-99	COMPLETION DEPTH: 24.5 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 173.0 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
173.0	0	3 inches asphaltic concrete over 5 inches aggregate base						
172.7 / 172.2		SILTY CLAY (CL) very stiff to hard, moist, brown, trace subrounded gravel to 1/2 inch, trace sand, occasional competely weathered sandstone fragments and fine sandy pocket	CL	39	21	108		△; >○
				52	19	111		>○
	5			51	19	108		>○
165.5		SILTY SAND (SM) dense, moist, orange-brown, uniform fine grained, trace clay Estimated angle of internal friction: >40°	SM	49	12			△
161.5	10	SILTY SAND (SM) dense, moist, fine to coarse grained, some subrounded to angular fractured gravel to 1 1/4 inch, some iron oxide coatings on fractures, occasional clayey sand to sandy clay seam Estimated angle of internal friction: 38°- >42°	SM	32	9		12	
	15			50/6"	5			
	20	very dense		50/5"	7			
148.5	25	Bottom of Boring at 24 1/2 feet						
	30							
	35							

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-6

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-18-99 FINISH DATE: 5-18-99	COMPLETION DEPTH: 26.5 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

SURFACE ELEVATION: 173.5 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH
173.5	0							
173.2 / 172.7		3 inches asphaltic concrete over 5 inches aggregate base						
		SILTY CLAY (CL) very stiff to hard, moist, orange brown, trace subrounded gravel, some fine sand, occasional completely weathered sandstone fragments and fine sand, pockets up to 1/2 inch	CL	35	17	116		Torvane ~2.5; >4.0
				34	16			Pocket Penetrometer ~3.5
167.5		SILTY SAND (SM) dense, moist, orange brown, uniform fine grained , trace clay	SM	48	19	113		
165.0		SILTY SAND (SM) very dense, moist, orange brown, some gravel to 3/4 inch, some clay and sandy clay seams Estimated angle of internal friction: >42°	SM	53	7		14	
				78	9			
156.0		SILTY CLAY (CL) very stiff, moist, orange-brown, mottled black, trace fine sand, low plasticity, becomes dense	CL	35	20			Pocket Penetrometer ~3.5
152.0		SILTY SAND (SM) medium dense, moist, orange-brown, uniform fine grained, trace fine gravel, low plasticity, trace fine gravel Estimated angle of internal friction: 33°-39°	SM	25	16			Pocket Penetrometer ~3.0
148.5	25	SILTY CLAY (CL) very stiff, moist, orange brown, trace fine sand, low plasticity	CL	24	23			
147.0		Bottom of Boring at 26 1/2 feet						

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-7

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-18-99 FINISH DATE: 5-18-99	COMPLETION DEPTH: 25.0 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
174.5	0	SURFACE ELEVATION: 174.5 FT. (+/-)						
174.2		3 inches asphaltic concrete over 5 inches aggregate base						
173.7		SILTY CLAY (CL) hard, moist, brown to orange-brown, occasional completely weathered, sandstone fragments, trace coarse sand and fine subrounded gravel, low plasticity, trace rootlets	CL	19	19	108		△; >○
				31	24	105	84	○
	5	increase in fine sand		63	16	119		△; >○
166.5		CLAYEY SAND (SC) dense, moist, orange-brown, subangular to subrounded gravel to 1 inch Estimated angle of internal friction: 37°-42°	SC	39	8			
	10							
		increase in clay content		30	11		17	
160.0	15	SILTY SAND (SM) dense, moist, orange-brown, fine uniform grained	SM					
158.5		SILTY CLAY (CL) hard to very stiff, moist, orange brown, low plasticity, occasional thin fine grained silty sand lense	CL	56	21	109		>○
	20							
		very stiff		29	20			○
149.5	25	Bottom of Boring at 25 feet						
	30							
	35							

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-8

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-18-99 FINISH DATE: 5-18-99	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 173.5 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
173.5 – 173.2 – 172.7	0	3 inches asphaltic concrete over 5 inches aggregate base	
172.7 – 163.5	0.8 – 10	SANDY CLAY (CL) very stiff, moist, orange brown, with some silt, fine sand	CL
163.5 – 157.0	10 – 16.5	SILTY SAND (SM) very dense, moist, orange brown, subangular gravel to 1 inch, trace clay, fine to coarse grained sand increase sand Estimated angle of internal friction: >40°	SM
157.0 – 155.0	16.5 – 18.5	SILTY CLAY (CL) hard, moist, orange brown, low plasticity	CL
155.0 – 150.5	18.5 – 23	SILTY SAND (SM) very dense, moist, yellowish to olive brown, fine to coarse grained, some subangular to subrounded gravel up to 1 1/2 inch increase gravel Estimated angle of internal friction: >40°	SM
150.5 – 144.5	23 – 29	SILTY CLAY (CL) very stiff, moist, brown, low plasticity, trace coarse sand, fine gravel, some fine to medium sand increase gravel, increase medium to fine sand	CL
144.5 – 143.5	29 – 30	CLAYEY SAND with gravel (SC) dense, moist, orange brown to brown, subrounded gravel to 1 1/4 inch	SC
143.5	30	Bottom of Boring at 30 feet	

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
41	15	112		>4.0
42	18	112	61	
37	19	111		>4.0
48	14	121		
51	5			
50/6"	14			>4.0
27	18			
38	7			

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression; scale 1.0, 2.0, 3.0, 4.0

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-18-99 FINISH DATE: 5-18-99	COMPLETION DEPTH: 25.0 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 173.5 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
173.5	0							
173.2		3 inches asphaltic concrete over 6 inches aggregate base						
172.7		SANDY CLAY (CL) hard, moist, brown to orange brown, fine sand, trace fine gravels, low plasticity	CL	62	14		68	>○
				34	15			>○
	5			56	15	112		△ >○
	10			57	14	114		>○
162.0		GRAVELLY SAND (SP) medium dense, moist, brown. Estimated angle of internal friction: 38°-43°	SP	42	9			○
158.0	15	SANDY CLAY (CL) very stiff, moist, orange brown, low plasticity, trace fine gravel	CL	30	20			
155.0		CLAYEY SAND (SC) very dense, moist, brown, fine grained sand, trace clay. Estimated angle of internal friction: 33°-38°	SC	61				○
	20							
		medium dense		28	14			
148.5	25	Bottom of Boring at 25 feet						
	30							
	35							

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-10

Sheet 1 of 2

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-18-99 FINISH DATE: 5-18-99	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 180.5 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
180.5	0		
180.1		4 inches asphaltic concrete over 8 inches aggregate base	
179.5		SILTY CLAY (CL) very stiff to hard, moist, dark brown, trace fine sand, trace gravel, rootlets, low plasticity	CL
	5		
		increase gravel, alternating clayey sand lenses	
	10		
167.5		CLAYEY SAND (SC) very dense, moist, with some gravel, occasional completely weathered sandstone fragments and fine silty sand pockets Estimated angle of internal friction: >40°	SC
	15		
	20		
		dense	
	25		
153.5		SILTY CLAY WITH SAND (CL) very stiff, moist, orange brown, trace fine gravel, low plasticity	CL
	30		
146.5			SP
145.5	35		

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
42	13	107	
30	12	106	
38	15	108	
50/5"	7	117	
50/5"	4		
84	4		
63	6		18
68	5		
30	19	119	
50/6"	5		

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-10 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-18-99 FINISH DATE: 5-18-99	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, 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 (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
145.5	35	GRAVELLY SAND (SP) very dense, moist, orange-brown, subangular gravel to 1 inch Estimated angle of internal friction: >42°	SP				
				50/5"	3		
				64	4		
131.8		SILTY CLAY (CL) very stiff	CL	25	23		91
130.5	50	Bottom of Boring at 50 feet					

Depth scale: 35, 40, 45, 50, 55, 60, 65, 70

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-11

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-19-99 FINISH DATE: 5-19-99	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 180.5 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH (ksf)
180.5 / 180.2	0	3 inches asphaltic concrete over 9 inches aggregate base						
179.5		SILTY CLAY (CL) very stiff to hard, moist, brown to orange brown, trace fine sand, some medium to fine gravel, low plasticity	CL	41	12	118		△ ~2.0; ○ >4.5
				36	16			○ >4.5
	5			53				△ ~2.7; ○ >4.5
				50/5"	15 10	97		
	10	Clayey Sand Lense						
167.0		SANDY CLAY (CL) hard, moist, orange brown, some fine sand, low plasticity	CL	50/6"	11	108		△ ~3.0; ○ >4.5
	15							
	20			34	9			
157.0		GRAVELLY SAND (SC) medium dense, dry, orange brown, with some clay, subrounded gravel to 3/4 inch. Estimated angle of internal friction: 35°-41°	SC	26	4			
	25							
152.0		SILTY CLAY (CL) hard, moist, orange brown, some fine to medium sand		50/6"	19			○ >4.5
150.5	30	Bottom of Boring at 30 feet						
	35							

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-12

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-19-99 FINISH DATE: 5-19-99	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 173.0 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
173.0	0	3 inches asphaltic concrete over 5 inches aggregate base	
172.7			
172.2		SILTY CLAY (CL) hard to stiff, dry to moist, orange brown, trace fine to medium sand, low plasticity, rootlets increase sand sand lense	CL
156.5		SILTY SAND (SM) medium dense, moist, orange brown, fine uniformly graded sand, trace clay Estimated angel of internal friction: 33°-39° increase clay	SM
148.5		GRAVELLY SAND (SC) medium dense, dry to moist, brown, with some clay, subangular gravel to 3/4 inch	SC
145.0		GRAVELLY SAND (SP) very dense, dry, brown, gravel to 1 inch, trace clay Estimated angle of internal friction: >42°	SP
143.0	30	Bottom of Boring at 30 feet	

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
54	18	115	
45	17		
58	16	113	
41	17	98	
15	12		
25	10		35
29	8		
70	5		

Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

>

>

>

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/1/99 MV*

EXPLORATORY BORING: EB-13

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-19-99 FINISH DATE: 5-19-99	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 172.5 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
172.5	0	3 inches asphaltic concrete over 5 inches aggregate base	
172.2 / 171.7		SANDY CLAY (CL) hard, moist, orange brown, fine to coarse grained, some silt, occasional gravel, thin sandy lense at 2 feet	CL
168.5		SILTY CLAY (CL)	CL
167.0	5	GRAVELLY SAND (SC) dense to very dense, brown, subangular gravel to 3/4 inch, with trace to some clay decreasing clay Estimated angle of internal friction: 38°- >42° increase sand	SC
154.5	10, 15	SILTY CLAY (CL) very stiff, moist, brown, low plasticity, trace fine sand	CL
142.5	20, 25, 30	Bottom of Boring at 30 feet	
	35		

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
60	12	120	25	>
50/6"	8	82		
33	7			>
62	5			
92	6			
46	19			>
49	16	113		
45	15	118		

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression; 1.0 2.0 3.0 4.0

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-14

Sheet 1 of 1

DRILL RIG: MOBILE B-40	PROJECT NO: 259-5D
BORING TYPE: 8-INCH HOLLOW STEM	PROJECT: VALLCO EXPANSION
LOGGED BY: LML	LOCATION: CUPERTINO, CALIFORNIA
START DATE: 5-19-99 FINISH DATE: 5-19-99	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, 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.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 172.5 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
172.5 – 172.2 – 171.7	0	3 inches asphaltic concrete over 5 inches aggregate base	
171.7 – 167.5		SANDY CLAY (CL) hard, moist, orange brown, fine sand, some silt, trace coarse gravel; increase sand and gravel	CL
167.5 – 159.5	5	CLAYEY SAND (SC) medium dense, dry, brown, with some fine gravel. Estimated angle of internal friction: 36°-40°; decrease clay	SC
159.5 – 155.5		SANDY GRAVEL (GC) dense, dry to moist, brown, trace to some clay	GC
155.5 – 148.0		SILTY CLAY (CL) hard, moist, brown, some fine sand, trace gravel, low plasticity	CL
148.0 – 143.0	25	SANDY CLAY (CL) hard, moist, orange brown, low plasticity	CL
143.0 – 142.5	30	SANDY GRAVEL (GC) very dense, moist, brown, subangular gravel to 1 inch, trace to some clay	GC
		Bottom of Boring at 30 feet	

Approx. Depth (FT)	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
2	42	14	123	
4	42	11	108	55
6	32	10		
8.5	42	28		
11	34	7		
14	43	8		
19	66	22	107	89
24	55	22	105	
29	50/6"	14		

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 7/1/99 MV*

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

DRILL RIG: Continuous Flight Auger	SURFACE ELEVATION: 190' (Approx.)	LOGGED BY: R.R.
DEPTH TO GROUNDWATER: Not Established	BORING DIAMETER: 6 Inches	DATE DRILLED: 6/4/74

DESCRIPTION AND CLASSIFICATION										
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
3" Asphaltic Concrete over 6" Baserock				1						
CLAY, silty with trace of sand and gravel	brown	stiff	CL		x					13
				2						
				3	x				21	28
				4						
					x					13
				5						
				6						
(grading more sandy and gravelly)		very stiff		7						
				8						
				9						
					x				15	24
				10						
Bottom of Boring = 10 Feet										
				11						
				12						
				13						
				14						
				15						
				16						
				17						
				18						
				19						
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	1

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 188' (approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/4/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
3" Asphaltic Concrete over 6" Baserock										
CLAY, sandy, gravelly	brown	stiff	CL	1						
				2	x				13	10
	gray-brown	very stiff		3	x					17
				4						
				5	x				17	17
				6						
				7						
				8						
				9						
				10	x					20
Bottom of Boring = 10 Feet				11						
				12						
				13						
				14						
				15						
				16						
				17						
				18						
				19						
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	2

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 187' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/4/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty	brown	stiff	CL	1	x					15
		very stiff		2	x				17	16
				3						
(trace of coarse sand and gravel)				4	x					18
GRAVEL, sandy, silty	brown	medium dense	GM	5						
				6						
SAND, gravelly, silty	yellow-brown	loose	SM	7						
				8						
				9	x				10	7
Bottom of Boring = 10 Feet				10						
Note: The stratification lines represent the approximate boundary between soil types and the transitions may be gradual.				11–20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	3

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	184' (Approx.)	LOGGED BY	R.R.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/4/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty	brown	very stiff	CL	1	x				7	18
(trace of gravel)				2	x					24
SAND, gravelly, clayey	brown	medium dense	SC	3						
				4	x				11	13
(grading more gravelly)			GC	6						
				8–9	x				7	29
Bottom of Boring = 9 Feet										

Note: The stratification line represents the approximate boundary between soil types and the transition may be gradual.

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	4

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	183' (Approx.)	LOGGED BY	R.R.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/4/74

DESCRIPTION AND CLASSIFICATION										
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
GRAVEL, clayey with some cobbles	brown	medium dense	GC	1	x					37
(grading less clayey, more silty)			GM	2	x				4	28
		dense to very dense		3						
				4	x					66
				5						
				6						
SAND, gravelly, clayey	brown	medium dense	SC	7						
				8						
				9	x				7	19
Bottom of Boring = 10 Feet				10						
Note: The stratification line represents the approximate boundary between soil types and the transition may be gradual.				11						
				12						
				13						
				14						
				15						
				16						
				17						
				18						
				19						
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	5

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 173' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/5/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty	dark brown	stiff	CL							
Liquid Limit = 44%				1	x				20	14
Plasticity Index = 22%				2						
Passing #200 Sieve = 76%					x				22	9
				3						
	brown			4	x				17	9
				5						
				6						
				7						
Note: The stratification line represents the approximate boundary between soil types and the transition may be gradual.				8						
				9	x					12
				10						
				11						
				12						
				13						
				14						
SAND, gravelly, clayey to GRAVEL, sandy, clayey	gray-brown	medium dense	SC-GC		x				8	19
				15						
				16						
				17						
		dense		18						
(grading less gravelly, more silty)			SM	19	x				7	40
				20						
Bottom of Boring = 20 Feet										

LOWNEY·KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	9

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 179' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/5/74

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty	brown	stiff	CL							
				1	x					14
(grading sandy)				2						
					x				12	11
				3						
				4						
					x					7
				5						
				6						
GRAVEL, sandy with clay binder	brown	dense	GC	7						
				8						
				9						
					x				5	49
				10						
				11						
CLAY, silty	brown	stiff	CL	12						
				13						
				14						
					x				16	16
				15						
				16						
		very stiff		17						
				18						
				19	x					
SAND, silty, fine grained	light brown	medium dense	SM		x					20
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 2	10

DRILL RIG: Continuous Flight Auger	SURFACE ELEVATION 179' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/5/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, silty, fine grained (Continued)	light brown	medium dense	SM	21						
SAND, gravelly, silty	gray-brown	very dense	SM	22						
				23						
				24	x				5	58
				25						
				26						
				27						
				28						
				29	x					55
				30						
Bottom of Boring = 30 Feet										

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

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	2 OF 2	10

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 181' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE
CLAY, silty	brown	stiff	CL
Dry Density = 105 pcf Unconfined Compressive Strength = 4,400 psf		very stiff to hard	
GRAVEL, sandy, clayey Dry Density = 116 pcf	gray-brown	dense	GC
CLAY, silty	brown	very stiff to hard	CL
Dry Density = 101 pcf Unconfined Compressive Strength = 5,300 psf			

DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
1	x					13
2						
3					19	34
4						
5						
6						
7						
8						
9					10	40
10						
11						
12						
13						
14					23	41
15						
16						
17						
18						
19	x					34
20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 3	11

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 181' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty	brown	very stiff	CL	21						
				22						
				23						
				24						
					x					29
				25						
				26						
				27						
				28						
				29						
					x				22	17
				30						
				31						
				32						
				33						
SAND, silty, fine to medium grained	brown	medium dense	SM	34						24
CLAY, silty	brown	very stiff	CL	35						
				36						
(occasional lenses of silty sand)				37						
				38						
				39						
					x				19	17
				40						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	2 OF 3	11

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 181' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty (Continued)	brown	very stiff	CL							26
				41						
				42						
				43						
				44	x					
				45						
Bottom of Boring = 45 Feet										
Note: The stratification lines represent the approximate boundary between soil types and the transitions may be gradual.										

LOWNEY·KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	3 OF 3	11

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, gravelly	dark brown	very stiff	CL							
				1	x					22
				2						
					x				15	33
				3						
				4					11	21
				5						
				6						
				7						
GRAVEL, sandy, silty	brown	dense	GM	8						
				9	x				8	39
				10						
				11						
CLAY, silty	brown	hard	CL	12						
				13						
				14	x					35
				15						
				16						
Dry Density = 106 pcf Unconfined Compressive Strength = 3,800 psf				17						
				18						
				19						
(grading very silty)			CL-ML		x				21	43
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 2	12

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty to SILT, clayey (Continued)	brown	hard	CL-ML							
				21						
				22						
				23						
Dry Density = 98 pcf Unconfined Compressive Strength = 1,800 psf				24					26	45
				25						
				26						
		very stiff		27						
				28						
				29	x					30
Bottom of Boring = 30 Feet				30						
Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.										

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	2 OF 2	12

DRILL RIG 'Continuous Flight Auger	SURFACE ELEVATION 183' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty with occasional lenses of very fine grained sand	brown	firm	CL							
				1						
				2	x				25	7
		stiff		3						
				4						
				5						
				6						
				7						
Dry Density = 109 pcf Unconfined Compressive Strength = 3,800 psf				8						
		very stiff to hard		9					19	40
				10						
				11						
				12						
				13						
Dry Density = 101 pcf Unconfined Compressive Strength = 4,200 psf				14					24	68
				15						
				16						
		very stiff		17						
				18						
				19	x					28
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 2	13

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	183' (Approx.)	LOGGED BY	R.R.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/6/74

DESCRIPTION AND CLASSIFICATION										
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty (Continued)	brown	very stiff	CL							
				21						
				22						
		hard		23						
				24						
					x					49
				25						
				26						
		very stiff		27						
				28						
				29						
					x				20	31
				30						
Bottom of Boring = 30 Feet										

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	2 OF 2	13

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 184' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty with trace of coarse sand	brown	stiff	CL	1						
				2						
				3						
				4	x				21	10
				5						
				6						
		very stiff to hard		7						
Dry Density = 107 pcf Unconfined Compressive Strength = 2,700 psf				8						
				9						
				10					19	53
				11						
				12						
SAND, gravelly with some clay binder	brown	dense to very dense	SC	13						
Dry Density = 118 pcf				14					15	68
				15						
				16						
CLAY, silty to SILT, clayey	brown	very stiff	CL-ML	17						
				18						
				19						
				20	x				18	27

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 2	14

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 184' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/6/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty to SILT, clayey (Continued)	brown	very stiff	CL-ML	21						
				22						
				23						
(grading less silty)				24						
			CL		x					32
				25						
				26						
				27						
CLAY, sandy	brown	hard	CL							
				28						
				29						
					x				17	41
				30						
Bottom of Boring = 30 Feet										
Note: The stratification lines represent the approximate boundary between soil types and the transitions may be gradual.										

LOWNEY·KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	2 OF 2	14

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 186' (Approx.)	LOGGED BY A.K.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/7/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty, trace of fine sand	dark brown	very stiff	CL	0 – 7						
				4				X	19	21
CLAY, silty, sandy, gravelly Dry Density = 109 pcf Unconfined Compressive Strength = 3,500 psf	brown	hard	CL	7 – 12.5						
				9.5				X	22	39
CLAY, silty Dry Density = 107 pcf Unconfined Compressive Strength = 5,100 psf	tan	hard	CL-CH	12.5 – 16						
				14.5				X	20	57
(grading siltier with depth)		very stiff	CL	16 – 20						
				19	x		X		21	28

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 2	15

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	186' (Approx.)	LOGGED BY	A.K.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/7/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT
CLAY, very silty (Continued)	tan	very stiff	CL	21						
				22						
		hard		23						
(grading sandy and gravelly with depth)				24	x					48
				25						
				26						
				27						
(rock blocked end of split spoon sampler)				28						
				29	x					99
Bottom of Boring = 29.5 Feet				30						

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

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	2 OF 2	15

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	186' (Approx.)	LOGGED BY	A.K.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/7/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty, trace of fine sand	dark brown	very stiff	CL	0–6						
Dry Density = 104 pcf Unconfined Compressive Strength = 6,400 psf				3–4.5				X	20	24
CLAY, silty, sandy (well graded) gravelly (fine)	brown	hard	CL	6–12.5						
Dry Density = 115 pcf Unconfined Compressive Strength = 4,500 psf				8–9.5				X	15	91
				10	179					
CLAY, silty	tan	hard	CL	12.5–20						
				13–14.5				X		91
(grading siltier with depth)		very stiff		16–17						
				18.5–20	x		X		22	23

LOWNEY·KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 2	16

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	186' (Approx.)	LOGGED BY	A.K.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/7/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, very silty (Continued)	tan	very stiff	CL	21						
(grading with fine sand with depth)				22						
		hard		23						
				24	x					37
				25						
(grading less sandy with depth)				26						
				27						
				28						
				29	x				17	53
Bottom of Boring = 29.5 Feet				30						
Note: The stratification lines represent the approximate boundary between soil types and the transitions may be gradual.										

LOWNEY·KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 2	16

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 185' (Approx.)	LOGGED BY A.K.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/7/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty, trace of fine sand	dark brown	very stiff	CL	0–7				3–4.5	20	18
CLAY, silty, sandy (well)	brown	hard	CL	7–8.5						
SAND (well), gravelly (fine and medium), clayey	brown	dense	SC-SW	8.5–12				8–9.5	9	38
GRAVEL, sandy	brown	dense	GW	12–14						
				14	x		13–14.5			39
SAND, clayey, gravelly	brown	dense very dense	SC-SW	14–20						
				19	x		18–19.5		8	50/7"

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 2	17

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 185' (Approx.)	LOGGED BY A.K.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/7/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, clayey, gravelly (Continued)	brown	very dense	SC-SW	21						
				22						
				23						
				24	x					83
				25						
				26						
				27						
				28						
				29	x				6	84
Bottom of Boring = 29.5 Feet				30						

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

LOWNEY·KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-9	June 1974	2 OF 2	17

Continuous Flight Auger	SURFACE ELEVATION 184' (Approx.)	LOGGED BY A.K.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/7/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, gravelly	brown	dense	SW							
				1						
				2						
				3						
				4						43
				5						
				6						
		medium dense		7						
				8						
				9					9	20
				10						
				11						
CLAY, silty	brown	hard	CL-CH	12						
				13						
				14						50
(grading siltier with depth)				15						
		very stiff	CL	16						
				17						
				18						
				19 x					19	18
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG			
VALLCO PARK REGIONAL SHOPPING CENTER Cupertino, California			
PROJECT NO. 259-5	DATE June 1974	SHEET NO. 1 OF 2	BORING NO. 18

EB-18

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 184' (Approx.)	LOGGED BY A.K.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/7/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty (Continued)	brown	very stiff	CL	21						
				22						
(grading with some fine sand)		hard		23						
				24	x				21	41
				25						
				26						
				27						
				28						
				29	x					34
Bottom of Boring = 29.5 Feet				30						
Note: The stratification line represents the approximate boundary between soil types and the transition may be gradual.										

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 2	18

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)
CLAY, silty	brown	firm	CL	0
		stiff		2
Dry Density = 102 pcf Unconfined Compressive Strength = 1700 psf				3.5
CLAY, gravelly to GRAVEL clayey	brown	very stiff to medium dense	CL-GC	7.5
CLAY, silty	brown	hard	CL	10.5
Dry Density = 113 pcf Unconfined Compressive Strength = 7200 psf				13
GRAVEL, clayey	brown	very dense	GC	14.5
(grading silty and sandy)			GM	18

DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
0–1.5	x		x			6
2–3	x		x			9
3.5–5				x	20	15
9–10	x		x			22
13.5–15				x	11	78/10"
19–20	x		x			65

LOWNEY · KALDVEER ASSOCIATES Foundation/Soil/Geological Engineers	EXPLORATORY BORING LOG VALLCO PARK REGIONAL SHOPPING CENTER Cupertino, California			
	PROJECT NO. 259-5	DATE June 1974	SHEET NO. 1 OF 2	BORING NO. 20

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	180' (Approx.)	LOGGED BY	R.R.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
GRAVEL, sandy, silty (Continued)	brown	very dense	GM	21						
				22						
				23						
				24	x				6	57
				25						
				26						
SAND, clayey	brown	dense	SC	27						
				28						
				29	x				15	40
				30						
Bottom of Boring = 30 Feet										
Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.										

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 2	20

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty with occasional gravel	brown	stiff	CL	0–1	x		x			10
Dry Density = 104 pcf Unconfined Compressive Strength = 4300 psf				2–3				x	21	28
		very stiff		5–6						
SAND, gravelly, clayey	brown	very dense	SC	8–9	x		x		8	50/6"
CLAY, silty	brown	hard	CL	12–14	x		x			52
SAND, gravelly, clayey	brown	very dense	SC	17–18						
Dry Density = 109 pcf				19				x	7	53/6"

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 3	21

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, gravelly, clayey (Continued)	brown	very dense	SC	21, 22						
SAND, silty, very fine grained	brown	dense	SM	23, 24, 25	x		24–25			36
CLAY, silty Dry Density = 106 pcf Unconfined Compressive Strength = 3100 psf	brown	hard	CL	26, 27, 28, 29, 30				28.5–30	16	57
(occasional gravel)				31, 32, 33, 34, 35				33.5–35		91
SAND, gravelly with some clay binder	brown	very dense	SC	36, 37, 38, 39, 40	x		38.5–39.5		7	50/6"

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 3	21

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	180' (Approx.)	LOGGED BY	R.R.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, gravelly with some clay binder (Continued)	brown	very dense	SC							
				41						
				42						
				43						
(grading more gravelly)			SC-GC							
				44	x				5	50/6"
				45						
Bottom of Boring = 44.5 Feet										
Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.										

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	3 OF 3	21

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 178' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, gravelly, clayey	brown	loose	SC		x				13	7
				1						
Liquid Limit = 29%										
Plasticity Index = 12%				2						
Passing No. 200 Sieve = 42%					x					9
		medium dense		3						
Dry Density = 127 pcf				4						
Unconfined Compressive Strength = 1,200 psf									17	19
				5						
				6						
GRAVEL, sandy, clayey	brown	medium dense	GC	7						
				8						
				9	x				8	30
		dense								
				10						
				11						
SAND, clayey with some gravel	brown	dense	SC	12						
				13						
				14						40
					x					
				15						
				16						
(grading more gravelly)		very dense		17						
				18						
				19						
					x				8	66
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 2	22

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 178' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, gravelly, clayey (Continued)	brown	very dense	SC	21						
CLAY, silty with silty sand lenses	brown	very stiff	CL	22						
				23						
				24	x				24	26
Bottom of Boring = 25 Feet				25						

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

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 2	22

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 181' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND CLASSIFICATION				DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE							
CLAY, silty with trace of coarse grained sand	dark brown	stiff	CL	1	x					14
		very stiff		2	x				24	27
				3						
				4	x					18
Bottom of Boring = 5 Feet				5						
				6						
				7						
				8						
				9						
				10						
				11						
				12						
				13						
				14						
				15						
				16						
				17						
				18						
				19						
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 1	23

EB-23

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty with trace of coarse grained sand	dark brown	firm	CL		x				18	8
				1						
Liquid Limit = 37%		stiff		2						
Plasticity Index = 18%					x					10
Passing No. 200 Sieve = 64%				3						
		very stiff		4					18	22
Dry Density = 104 pcf										
Unconfined Compressive Strength = 2300 psf				5						
				6						
		hard		7						
				8						
(grading more sandy)	brown			9					16	57
Dry Density = 115 pcf										
Unconfined Compressive Strength = 6800 psf				10						
				11						
				12						
				13						
		very stiff		14						
					x					26
				15						
				16						
(grading less sandy)				17						
				18						
				19	x					23
				20						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 3	24

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	180' (Approx.)	LOGGED BY	R.R.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	6/10/74

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty with trace of coarse grained sand (Continued)	brown	very stiff	CL	21						
				22						
SAND, gravelly, clayey	brown	medium	SC	23						
				24	x					21
				25						
		dense to very dense		26						
				27						
				28						
				29	x					88/9"
				30						
GRAVEL, sandy, silty	gray-brown	very dense	GM	31						
				32						
				33						
				34	x				6	54/6"
				35						
SILT, clayey to CLAY silty	brown	very stiff	ML-CL	36						
				37						
				38						
				39	x					28
				40						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 3	24

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 180' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND CLASSIFICATION										
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SILT, clayey to CLAY silty (Continued)	brown	very stiff	ML-CL	41						
				42						
(grading more clayey with occasional lenses of fine grained sand)				43						
				44						
			CL		x				24	18
Bottom of Boring = 45 Feet				45						

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

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	3 OF 3	24

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 176' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty	dark brown	firm	CL	1	x					6
				2	x					16
				4						17
SAND, gravelly, clayey	brown	dense to very dense	SC	7–9						
				10	x				7	50
CLAY, silty with occasional lenses of silty sand	brown	very stiff	CL	12–13						
				14	x					25
				20	x				24	20

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	1 OF 2	25

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION 176' (Approx.)	LOGGED BY R.R.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 6/10/74

DESCRIPTION AND CLASSIFICATION				DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Modified Calif.	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE							
CLAY, silty with occasional lenses of silty sand (Continued)	brown	very stiff	CL	21						
				22						
				23						
				24	x				19	23
				25						
Bottom of Boring = 25 Feet										
Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.										

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June 1974	2 OF 2	25

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	---	LOGGED BY	J. C. P.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	9/15/72

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, clayey and silty with charcoal (Burn Pile Area)	black-brown	loose	SM-SC		x				15	5
CLAY, sandy and silty	brown	firm	CL	2	x				30	3
(grading with more sand)	light brown	stiff		6						
		very stiff		8	x				19	25
SAND, clayey and silty	light brown	medium dense	SM-SC	12	x				19	30
				18	x				20	23
SILT, very sandy to SAND, silty, fine grained	light brown	medium dense	ML-SM	22	x				19	30

Bottom of Boring = 23.5 Feet

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

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	A

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	----	LOGGED BY	J.C.P.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	9/15/72

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	Calif. Sampler	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty and sandy	dark	firm	CL		x				19	13
(Dry Density = 95 & 97 pcf)				2	x				21	9
(grading with more sand)	brown			4	x				19	6
	dark brown	stiff		6						
				8						
GRAVEL and SAND, silty and clayey	brown	medium dense	GM-GC	10	x					22
				12						
(grading with sand lenses)		dense		14						
				16	x					41
SAND, silty	brown	dense	SM	18						
GRAVEL, sandy and silty	brown	dense	GM	20	x					45
				22						
SILT, sandy	brown	medium dense	ML	24						
				26	x				8	14
Bottom of Boring = 26.5 Feet				28						
Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.				30						

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	B

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION ---	LOGGED BY J.C.P.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 9/15/72

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty and sandy	dark brown	firm	CL		x				14	8
				2	x				16	6
				4						
SAND, silty, fine grained	light brown	loose	SM							
CLAY, sandy and silty	light brown	firm	CL	6	x				10	9
				8						
(grading with more sand)		stiff								
				10						
		very stiff			x				20	25
				12						
				14						
SAND, silty and clayey	brown	medium dense	SM-SC	16	x				17	28
(grading with very silty lenses)				18						
				20						
SAND, silty with lenses of SILT, sandy	light brown	medium dense	SM-ML		x				19	30
				22						
				24						
				26	x				15	17
Bottom of Boring = 26.5 Feet				28						
				30						

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

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	C

EB-C

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION ---	LOGGED BY J.C.P.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 9/15/72

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
SAND, silty and clayey with fine gravel (Dry Density = 112 pcf)	brown	medium dense	SM-SC	2	x				16	
CLAY, silty and sandy	brown	firm	CL	4	x				24	4
	dark brown	stiff to very stiff		6, 8	x				24	20
GRAVEL, sandy	brown	medium dense	GP	10						
CLAY, sandy and silty with some gravel	brown	very stiff	CL		x				10	14
				12, 14	x				18	22
SAND, clayey and silty	brown	dense	SM-SC	16						
GRAVEL, sandy with some silt	brown	dense	GM	18, 20	x					33
(grading with little silt and less sand)		very dense		22, 24, 26	x					40/6"
SILT, very sandy with some clay	brown	dense	ML	28, 30	x				21	35
GRAVEL, sandy	brown	dense	GP	32, 34						
SAND, silty and clayey with some gravel (grading with more gravel)	brown	dense to very dense	SM	36, 38, 40	x				12	51

LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 2	D

EB-D

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION ----	LOGGED BY J.C.P.
DEPTH TO GROUNDWATER Not Established	BORING DIAMETER 6 Inches	DATE DRILLED 9/15/72

DESCRIPTION AND CLASSIFICATION				DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE							
GRAVEL, sandy with some cobbles	brown	dense to very dense	GP	42 44 46						
Bottom of Boring = 47 Feet Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.				48 50						

LOWNEY · KALDVEER ASSOCIATES Foundation/Soil/Geological Engineers	EXPLORATORY BORING LOG VALLCO PARK REGIONAL SHOPPING CENTER Cupertino, California			
	PROJECT NO.	DATE	SHEET NO.	BORING NO.
	259-5	June, 1974	2 OF 2	D

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	---	LOGGED BY	J.C.P.
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	9/15/72

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (feet)	JARS	SACKS	SPLIT SPOON	SHELBY TUBE	MOISTURE CONTENT %	PENETRATION RESISTANCE BLOWS/FT.
CLAY, silty and sandy with some organic matter near surface	light brown	firm to stiff	CL		x				19	9
				2	x				17	8
(Dry Density = 108 pcf)					x				18	
(grading more clay with some fine gravel)	dark brown	very stiff		4						
				6	x				22	17
				8						
GRAVEL, sandy with some silt	brown	dense	GM-GP	10						
					x					40
				12						
(grading with more sand)				14						
				16	x					43
				18						
SILT, sandy to SAND, silty	brown	medium dense	ML-SM	20						
					x				19	28
				22						
SAND, silty	brown	medium dense	SM	24						
				26	x				23	16
Bottom of Boring = 26.5 Feet				28						
Note: The stratification lines represent the approximate boundary between soil types and the transition may be gradual.				30						

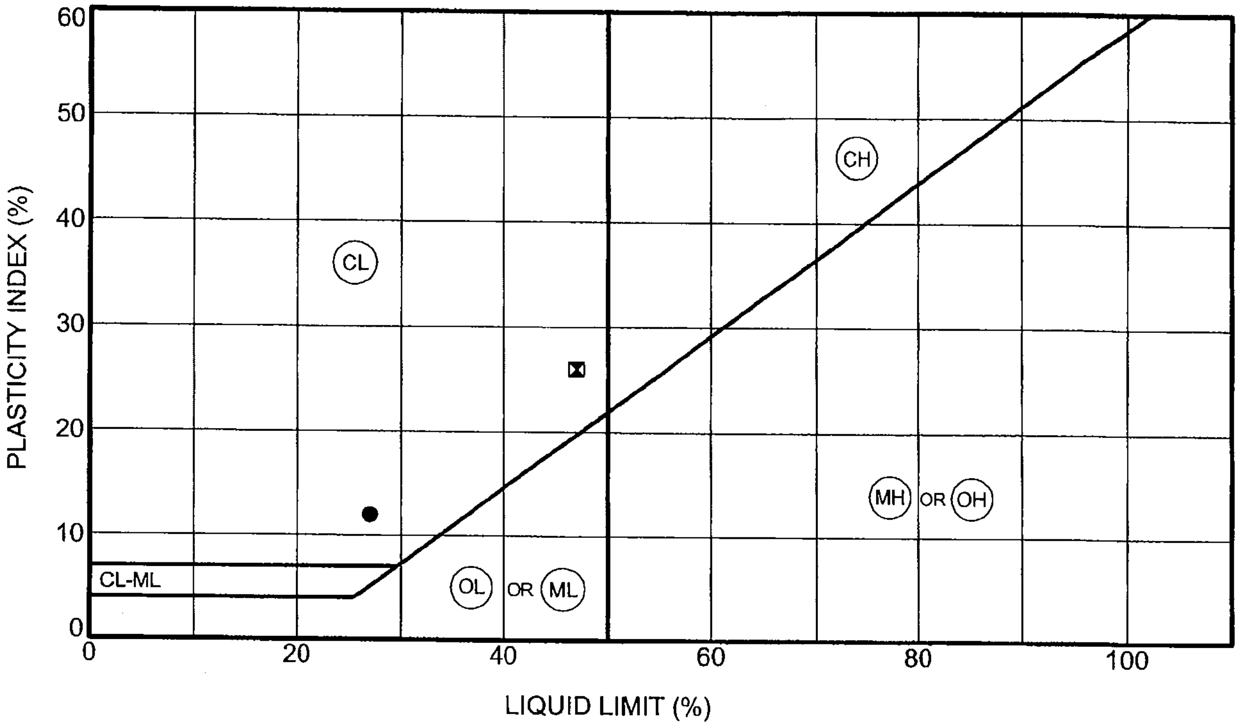
LOWNEY · KALDVEER ASSOCIATES

Foundation/Soil/Geological Engineers

EXPLORATORY BORING LOG

VALLCO PARK REGIONAL SHOPPING CENTER
Cupertino, California

PROJECT NO.	DATE	SHEET NO.	BORING NO.
259-5	June, 1974	1 OF 1	E



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-1** LB-1	**1.5**	**13**	**27**	**15**	**12**		**LEAN CLAY (CL)**
⊠	**EB-4** LB-4	**1.5**	**18**	**47**	**21**	**26**		**LEAN CLAY WITH SAND (CL)**

LA CORP.GDT 8/20/04 MV* FLL

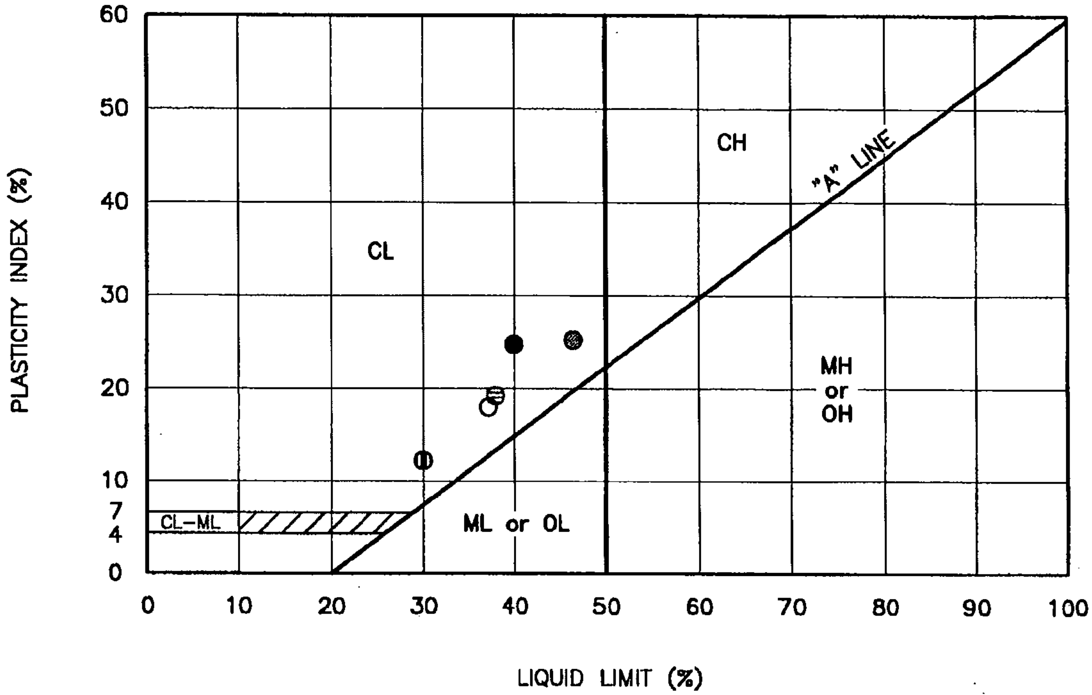
LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

PLASTICITY CHART AND DATA

Project: VALLCO
Location: CUPERTINO, CA
Project No.: 259-5E

2004 Geotechnical Investigation

FIGURE B-1



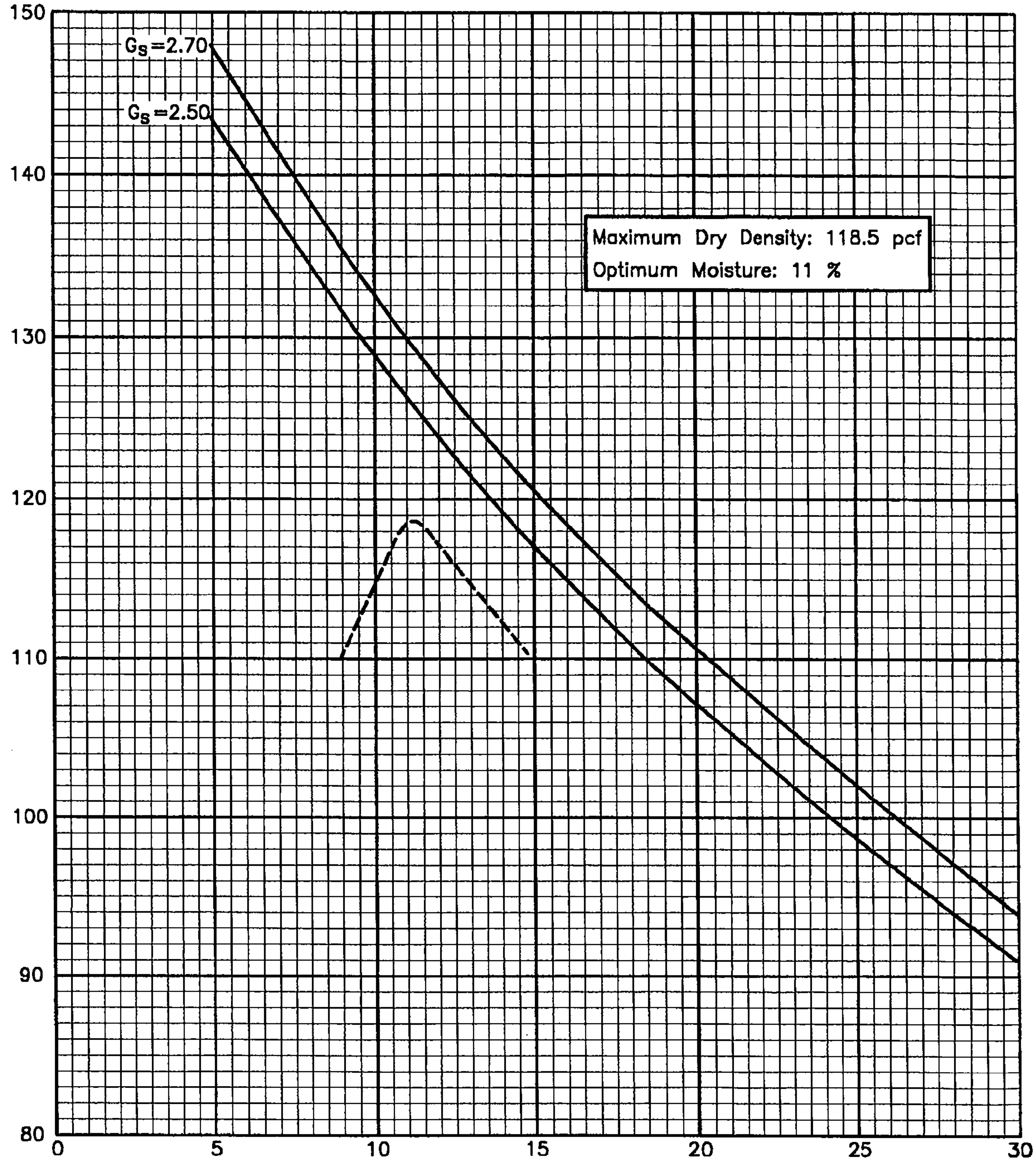
KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	PASSING #200 SIEVE (%)	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	EB-4 LA-4	2.0	19	40	24	53	--	CL
⊖	EB-9 LA-9	1.5	14	38	19	68	--	CL
○	B-24 EB-24	0.5	18	37	18	64	--	CL
◍	EB-E	0-1.5	19	30	12	62	--	CL
◉	EB-E	5.0-6.5	22	46	25	77	--	CL

PLASTICITY CHART AND DATA

1999 Geotechnical Investigation

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-1
259-5D



LA-1, LA-2, LA-10, and LA-11

Sample Description: Bulk composite sample from boring EB-1, EB-2, EB-10, and EB-11 at depth of 0.5 to 5 feet. Dark brown silty clay (CL)

COMPACTION CURVE

VALLCO EXPANSION

Cupertino, California

1999 Geotechnical Investigation

FIGURE B-2

259-5D

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

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.

The natural water content was determined on 83 samples of the materials recovered from the borings; these water contents are recorded on the boring logs at the appropriate sample depths.

Atterberg Limits determinations were performed on three samples of the surface soils at the site to determine the range of water content over which these materials exhibit plasticity. The Atterberg Limits are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's expansion potential. The results of these tests, as well as the results of three tests performed during the previous investigation, are presented on Figure B-1 and on the logs of borings at the appropriate sample depths.

The percent passing the No. 200 sieve was determined on three samples of the surface soils to aid in the classification of these soils; the results of these tests, as well as the results of three tests performed during the previous investigation are presented on Figure B-1 and on the boring logs at the appropriate sample depths.

Dry density determinations were performed on 21 samples of the subsurface soils to evaluate their physical properties. The results of these tests as well as the result of three tests performed during the previous investigation are presented on the boring logs at the appropriate sample depths.

Unconfined compression tests were performed on 18 undisturbed samples of the clayey subsurface soils to evaluate the undrained shear strengths of these materials. The unconfined tests were performed on samples having a diameter of 2.8 inches and a height-to-diameter ratio of at least 2. Failure was taken as the peak normal stress. The results of these tests are presented on the boring logs at the appropriate sample depths.

Resistance "R" value tests were performed on two representative samples of the surface soils at the site to provide data for pavement design. The tests indicated that the expansion pressure controls the design of pavement sections with the "R" values by expansion equal to 4, 12 and 23 for traffic indices of 3.5, 4.5 and 6.0, respectively.

RESULTS OF "R" VALUE TESTS

Sample No.	Description of Material	Water Content (%)	Dry Density (pcf)	Exudation Pressure (psi)	"R" Value	Expansion Pressure (psf)
S-1	CLAY, silty	13	120	160	15	110
		12	122	270	24	140
		11	124	520	46	240
S-2	SAND, gravelly, silty and clayey	15	117	190	21	70
		13	118	410	32	80
		13	121	530	36	190

APPENDIX B

LOGS OF TEST BORINGS

PROJECT:	THE RISE Cupertino, California	Log of Boring B-1

Boring location:	See Site Plan, Figure 2
Date started:	9/7/16
Date finished:	9/8/16
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	D. Wagstaffe

MATERIAL DESCRIPTION — Ground Surface Elevation: 194.2 feet[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					4 inches asphalt concrete (AC)						
1					3 inches aggregate base (AB)						
2	HA				CLAY with GRAVEL (CH) brown to dark brown, moist, fine subangular gravel, trace fine sand, trace organics R-Value Test, see Figure D-14						
3											
4											
5											
6	S&H	4 7 11	13		decrease in gravel content, hard	PP		6,500		20.5	108
7											
8											
9											
10											
11	S&H	7 14 17	22		yellow-brown, very stiff LL = 59, PI = 39, see Figure D-1 Triaxial Test, see Figure D-2 Particle Size Analysis, see Figure D-12	TxUU	600	4,750		20.0	111
12											
13				CH							
14											
15											
16	S&H	4 7 10	12		stiff					16.5	116
17											
18											
19											
20					grades silty						
21	S&H	3 7 7	10			PP		3,500			
22											
23											
24											
25					SANDY CLAY with GRAVEL (CL)						
26	S&H	14 14 17	22	CL	brown to yellow-brown, very stiff, moist, fine sand LL = 31, PI = 16, see Figure D-1 Consolidation Test, see Figure D-9					13.4 17.7	112
27											
28					CLAYEY SAND with GRAVEL (SC)						
29				SC	brown, medium dense, moist, fine- to medium-grained sand,						
30											

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		11 15 20	25	SC	CLAYEY SAND with GRAVEL (SC) (continued) some fine subrounded gravel Triaxial test, see Figure D-3 Particle Size Analysis, see Figure D-12	TxUU	3,700	2,040	22	12.0	127
32												
33–34					GC	CLAYEY GRAVEL (GC) brown, very dense, moist, fine subangular gravel, medium to coarse sand						
35	SPT		35 50/ 6"	55/ 6"								
36					SP	SAND with CLAY (SP) yellow, very dense, moist, medium to coarse-grained						
37–39					SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, medium to coarse-grained, fine subangular gravel						
40–41	SPT		16 35 42	85	SC	Particle Size Analysis, see Figure D-12 yellow and red mottling, fine-grained sand, weakly cemented				17.1	10.1	
42–44					SC							
45–46	SPT		20 37 50	96	SC							
47–48					SC	∇ (09/08/16, 6:20 a.m.)						
49–51	S&H		14 12 32	31	SC	dense, medium-grained sand, fine subrounded to subangular gravel					10.7	
52					CL	SANDY CLAY with GRAVEL (CL) yellow-brown, very stiff to hard, wet, fine- to coarse sand, fine subrounded to subangular gravel						
53–57	SPT		22 32 50	90	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, wet, fine to medium-grained, fine subangular gravel						
58–60					CL	CLAY (CL) brown, hard, wet, trace fine subangular gravel						

PROJECT: **THE RISE**
Cupertino, California

Log of Boring B-1

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	SPT	9 25 30	61	CL	CLAY (CL) (continued)					20.6	
63				GC	CLAYEY GRAVEL with SAND (GC) brown, medium dense, wet, fine to coarse subrounded and subangular, fine to coarse sand						
66	S&H	4 11 15	18								
68	SPT	10 42 30	79	ML	SILT (ML) red, hard, wet						
69				GC	CLAYEY GRAVEL with SAND (GC) brown, medium dense, wet, fine to coarse subrounded and subangular, fine to coarse sand						
71	SPT	4 7 8	17								
74				CL	SANDY CLAY (CL) brown, hard, wet, fine sand						
76	S&H	4 19 50/ 3.5"	48/ 9.5"		Triaxial test, see Figure D-4	TxUU	9,100	640		18.0	112
76				SC	CLAYEY SAND (SC) brown, very dense, wet, fine to medium-grained					11.2	
78				SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, wet, medium-grained, subangular gravel						
80	SPT	27 50/ 6"	55/ 6"								
83				CL	SANDY CLAY (CL) brown, very stiff, wet, fine to medium sand, trace fine subangular gravel						
86	SPT	8 12 12	26	CL	CLAY (CL) brown, very stiff, wet, trace fine sand					19.4	
89				SC	CLAYEY SAND (SC) brown, very dense, wet, fine to medium-grained, some						

LANGAN

Project No.: 770633101

Figure: B-1c

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		30 50/ 6"	55/ 6"		fine subangular gravel				19.0	10.3	
91												
92												
93					SC							
94												
95						dense, fine-grained						
96	S&H		22 22 24	32							20.0	
97					CL	SANDY CLAY (CL) brown, hard, wet, fine sand						
98						CLAYEY SAND with GRAVEL (SC) brown, medium dense, wet, fine to coarse-grained, fine subangular gravel						
99												
100												
101	SPT		8 10 10	22	SC						18.4	
102												
103												
104												
105						CLAY (CL) brown, hard, moist, trace fine sand						
106	S&H		10 22 32	38	CL	grades sandy with increase sand content	PP		6,000		19.3	111
107												
108						CLAYEY SAND with GRAVEL (SC) brown, very dense, wet, fine to coarse-grained, fine subangular gravel						
109												
110	SPT		32 50/ 2.5"	55/ 2.5"						17.1	13.0	
111					SC							
112												
113												
114												
115						SANDY CLAY (CL) brown, hard, wet, fine sand						
116	SPT		10 10 17	30								
117					CL							
118												
119												
120					SC							

PROJECT:	THE RISE Cupertino, California	Log of Boring B-1

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
120	SPT		50/ 6"	55/ 6"	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, wet, fine to coarse-grained, fine subangular gravel, weak to moderate cementation				19.0	9.9	
121–129												
130–131	S&H		15 33 50/ 5.5"	58/ 11.5"	CL	CLAY with SAND (CL) brown, hard, wet, fine sand					14.6	122
131–139					SC	CLAYEY SAND (SC) brown to orange-brown, very dense, wet, fine to coarse-grained						
140–141	SPT		27 50/ 6"	55/ 6"	SC	CLAYEY SAND with GRAVEL (SC) orange-brown, very dense, wet, fine to coarse-grained, fine subangular to angular gravel						
142–150												

Boring terminated at a depth of 141 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 48 feet below ground surface on 09/08/16 at 6:20 a.m.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.1, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88 Datum.

LANGAN

Project No.: 770633101

Figure: B-1e

TEST GEOTECH LOG 770633101_THE RISE.GPJ TEMPLATE_CA-MODIFIED.GDT 11/14/23

PROJECT:	THE RISE Cupertino, California	Log of Boring B-2

Boring location: See Site Plan, Figure 2

Logged by: D. Wagstaffe

Date started: 9/6/16 | Date finished: 9/6/16

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

Ground Surface Elevation: 197.6 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches asphalt concrete (AC)						
					4 inches aggregate base (AB)						
1–4	HA			CL	CLAY (CL) brown, moist, trace fine sand grades sandy with fine subangular gravel						
5–8	S&H	9 12 20	22	CL	CLAY with GRAVEL (CL) dark brown, very stiff, moist, fine subangular gravel, some fine sand	PP		8,000		16.0	121
9–15	S&H	10 17 22	27	CL	CLAY (CL) brown, very stiff, moist, some fine to coarse sand, fine subrounded gravel increased gravel content					15.1	118
15–20	S&H	7 14 20	24	CL	CLAY with SAND (CL) dark brown, very stiff, moist, fine to medium sand Triaxial test, see Figure D-5 6-inch thick gravel layer	TxUU	1,900	4,580		18.6	113
21–25	S&H	10 14 23	26	CL	CLAY with SAND (CL) gray, very stiff, moist, fine sand, with trace coarse sand, with wood debris 6-inch thick gravel layer					17.8	116
25–29	S&H GRAB	8 14 20	24	CL	CLAY with SAND (CL) dark brown, very stiff, moist, fine sand, trace fine subangular gravel increased gravel content					20.1	110
29–30				SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist						

LANGAN

Project No.: 770633101 | Figure: B-2a

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT	11 27 23	55	SC							
33					increased gravel content						
35–36	SPT	5 10 14	26	CL	SANDY CLAY (CL) yellow-brown, very stiff, moist, fine sand					20.1	
39–41	S&H	10 24 27	36	CL	SANDY CLAY (CL) brown, hard, moist, fine sand Consolidation Test, see Figure D-10					17.2	111
45					increased gravel content						
45–46	SPT	10 9 8	19	SM	SILTY SAND (SM) yellow-brown, medium dense, moist, fine-grained, trace fine subrounded gravel Particle Size Analysis, see Figure D-12				25.0	24.2	
47–48	SPT	6 12 22	37	CL	CLAY (CL) brown, hard, moist, some sand, and gravel					20.4	
50	S&H	27 50/ 4.5"	35/ 4.5"	GC	CLAYEY GRAVEL with SAND (GC) brown, very dense, moist, fine subrounded, fine sand					9.8	
54–56	SPT	31 37 50/ 3.5"	96/ 9.5"	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine to coarse-grained, fine to coarse subangular to angular gravel Particle Size Analysis, see Figure D-12				16.7	9.8	
58–60				SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, very dense, moist, medium to coarse-grained, fine subangular gravel						

PROJECT:	THE RISE Cupertino, California	Log of Boring B-2

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60–61	SPT	27 50/ 6"	55/ 6"	SC	CLAYEY SAND with GRAVEL (SC) (continued)					11.2	
65–66	SPT	14 18 35	58	SC	fine to medium-grained, fine to coarse gravel, less clay						
70–71	SPT	17 50/ 6"	55/ 6"	SC	increased clay content, weak cementation, wet				16.7	10.5	
75–76	SPT	10 17 25	46	CL	SANDY CLAY (CL) brown, hard, wet, fine to coarse sand, trace fine subrounded to subangular gravel					13.7	
78				GC	CLAYEY GRAVEL with SAND (GC) yellow-brown, very dense, wet, coarse and subangular, fine to coarse sand						
80–81	SPT	25 32 32	70	GC							
85–86	SPT	32 50/ 6"	55/ 6"	GC	LL = 29, PI = 15, see Figure D-1					12.2	
90											

LANGAN

Project No.: 770633101

Figure: B-2c

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	SPT		35 34 38	79	GC	CLAYEY GRAVEL with SAND (GC) (continued) red and orange oxidation staining						
92												
93					SC	CLAYEY SAND (SC) yellow-brown, dense, wet, fine to medium-grained						
94												
95												
96	S&H		15 29 24	37							16.7	
	GRAB					with coarse subrounded gravel					16.4	
97					CL	SANDY CLAY (CL) yellow-brown, hard, wet, fine to coarse sand						
98												
99					CL	CLAY (CL) brown, very stiff, wet, with silt						
100												
101	S&H		11 22 24	32		Triaxial test, see Figure D-6					24.3	103.5
						fine gravel	TxUU	12,100	2,090		23.1	105
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 101.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.1, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88 Datum.

LANGAN

PROJECT:	THE RISE Cupertino, California	Log of Boring B-3 PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 9/14/16 | Date finished: 9/14/16

Drilling method: Hollow Stem Auger (B-61)

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Safety

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: D. Wagstaffe

Ground Surface Elevation: 196.1 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches asphalt concrete (AC)						
0–4.5	HA			CL	CLAY with SAND and GRAVEL (CL) brown, moist, fine sand, fine subangular gravel						
4.5					CLAY (CL) brown, hard, moist, trace medium sand						
5–6.5	S&H	21 30 49	47			PP		>4,500			
8				CL							
8.5–10	S&H	30 29 23	31		abundant fine sand	PP		>4,500			
11.5					SANDY CLAY (CL) brown, hard, moist, fine sand						
13.5–15	S&H	26 30 37	40			PP		>4,500			
18.5–20	SPT	12 13 14	27		very stiff						
21				CL							
23.5–25	S&H	22 16 20	22			PP		>4,500			
28.5–30	SPT	17 18 19	37		hard						

LANGAN

Project No.: 770633101 | Figure: B-3a

TEST GEOTECH LOG 770633101_THE RISE.GPJ TEMPLATE_CA-MODIFIED.GDT 11/14/23

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SANDY CLAY (CL) (continued)						
31												
32					CL							
33												
34	S&H		20 30 40	42		with fine sand						
35					SP	SAND (SP) yellow-brown, dense, moist, medium-grained sand, trace clay						
36					CL	SANDY CLAY (CL) brown, hard, moist, fine sand						
37						SAND with GRAVEL (SW) yellow-brown, very dense, moist, fine to coarse-grained, fine to coarse subangular gravel, trace clay						
38												
39	SPT		20 26 26	52								
40												
41												
42					SW							
43												
44	SPT		28 18 26	44		dense						
45												
46												
47						SANDY CLAY (CL) yellow-brown, stiff, moist, fine sand, with silt						
48					CL							
49	S&H		14 12 12	14			PP		3,500			
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88 Datum.

LANGAN

PROJECT:	THE RISE Cupertino, California	Log of Boring B-4 PAGE 1 OF 4

Boring location:	See Site Plan, Figure 2
Date started:	9/13/16
Date finished:	9/14/16
Drilling method:	Hollow Stem Auger (B-56 and B-61)
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Safety
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	D. Wagstaffe

Ground Surface Elevation: 182.4 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches asphalt concrete (AC)						
0.25–4.5	HA			CL	CLAY with SAND and GRAVEL (CL) brown, moist, fine to medium sand, fine subangular gravel R-Value Test, see Figure D-15						
4.5–12				CL	CLAY (CL) gray-brown, medium stiff to stiff, moist, trace fine sand						
5–6.5	S&H	3 4 7	7		LL = 44, PI = 25, see Figure D-1	PP		1,000			
8.5–10	S&H	6 14 20	20		stiff, trace medium-grained sand	PP		1,750			
12–14.25				CL	SANDY CLAY (CL) brown, hard, moist, fine sand						
13.5–15	S&H	8 26 30	34							8.7	
14.25–17.5				SC	CLAYEY SAND with GRAVEL (SC) brown, dense, moist, fine to coarse-grained, fine subangular gravel						
17.5–19.5				SW-SC	SAND with CLAY and GRAVEL (SW-SC) brown, medium dense, moist, fine- to coarse-grained, fine subangular gravel Particle Size Analysis, see Figure D-13						
18.5–20	SPT	20 10 9	19						11.5	7.7	
19.5–24.25				CL	CLAY (CL) brown, very stiff, moist, trace fine sand						
23.5–25	S&H	6 10 20	18			PP		3,500			
24.25–27				SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine-grained sand, trace coarse sand, trace fine subrounded gravel						
27–28.5				CL	CLAY (CL) brown, moist, trace fine sand						
28.5–30	S&H	7 7 12	11	SC ML	CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine-grained, trace coarse sand	PP		2,500			

LANGAN

Project No.: 770633101

Figure: B-4a

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31					ML	SILT (ML) yellow-brown, very stiff, moist, with clay						
32					CL	CLAY with SAND (CL) brown, hard, moist, fine sand						
33												
34	S&H		10 24 34	35								
35							PP		4,500			
36–38												
39	S&H		10 24 40	38		trace coarse sand; Triaxial test, see Figure D-7						
40							TxUU	2,300	21,510		21.4	104
41–43						with fine sand						
44	S&H		22 50/ 5"	30/ 5"	GP-GM	GRAVEL with SILT and SAND (GP-GM) brown, dense, moist, subangular to subrounded gravel, fine to medium sand; Particle Size Analysis, see Figure D-13				5.9	5.6	
46					SP-SM	SAND with SILT and GRAVEL (SP-SM) yellow-brown, very dense, moist fine to coarse-grained, trace subangular gravel, weakly cemented						
49	SPT		50/ 6"	50/ 6"		Particle Size Analysis, see Figure D-13				9.7	4.3	
52						cuttings have a cobble						
53					CL	SANDY CLAY (CL) brown with gray-brown mottling, hard, moist, fine to medium sand						
54	S&H		22 24 30	32								
55							PP		3,000			
56–58												
59	S&H		8 16 32	29		brown, with fine subrounded gravel						
60							PP		4,500			

PROJECT: **THE RISE**
Cupertino, California

Log of Boring B-4

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SANDY CLAY (CL) (continued)						
61					CL							
62												
63						CLAYEY SAND with GRAVEL (SC) yellow-brown, very dense, moist, fine to medium-grained, fine subangular gravel						
64	SPT		40 50/ 5"	50/ 5"								
65												
66						interbedded sand and clay layers						
67												
68					SC							
69	S&H		40 50/ 6"	30/ 6"								
70												
71												
72												
73						CLAY (CL) brown, hard, moist, trace fine sand						
74	S&H		12 25 38	38								
75						Consolidation Test, see Figure D-11	PP		4,500		20.7	105
76					CL							
77												
78												
79	S&H		12 25 50/ 5"	30/ 11"		SANDY CLAY (CL) yellow-brown, very stiff, moist, fine sand	PP		3,000			
80												
81												
82												
83					CL							
84	S&H		5 10 26	22								
85						Triaxial test, see Figure D-8	TxUU	10,100	1,220		21.8	105
86												
87												
88												
89	S&H		12 50/ 2"	30/ 2"	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine- to						
90						CLAYEY SAND with GRAVEL (SC) (continued)						

LANGAN

Project No.: 770633101

Figure: B-4c

PROJECT: **THE RISE**
Cupertino, California

Log of Boring B-4

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					SC	medium-grained, fine subangular gravel						
91–93					CL	SANDY CLAY (CL) yellow-brown, hard, moist, fine sand, trace fine subrounded gravel						
94	S&H	■	21 40 50/1"	54/7"			PP		4,500			
95–96					CL	GRAVELLY CLAY with SAND (CL) yellow-brown, hard, moist, fine subangular gravel, fine sand ▽ (09/14/16, 10:40 a.m.)						
97–98					SM	SILTY SAND (SM) yellow-brown, dense, wet, fine-grained						
99	S&H	■	28 40 41	49	CL	SANDY CLAY (CL) yellow-brown, hard, wet, fine sand, with medium sand						
100–101					SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, dense, wet, fine to coarse-grained, fine subrounded to subangular gravel						
102–120												

Boring terminated at a depth of 101.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encoutnerd at 96 feet on 09/14/16 at 10:40 a.m.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88 Datum.

LANGAN

Project No.: 770633101
Figure: B-4d

PROJECT:	THE RISE Cupertino, California	Log of Boring B-5

Boring location: See Site Plan, Figure 2
Logged by: D. Wagstaffe
Date started: 9/14/16 | Date finished: 9/14/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Safety
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

Ground Surface Elevation: 179.8 feet[2]

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches asphalt concrete (AC)						
1						CLAY (CL) brown, moist						
2–3	HA				CL							
4						with fine subangular gravel						
5–6	S&H		14 18 23	25		SANDY CLAY (CL) brown, very stiff, moist, fine sand					10.2	109
9–10	S&H		18 28 38	40		yellow-brown, hard, decreased sand content	PP		>4,500			
14–15	S&H		30 21 31	31	CL	with medium to coarse sand and fine subangular gravel	PP		>4,500			
19–20	S&H		15 20 30	30		with silt	PP		>4,500			
23–25	SPT		10 8 7	15	ML	SANDY SILT (ML) light brown, stiff to very stiff, moist, fine sand Particle Size Analysis, see Figure D-13				54.0	8.9	
26–27	SPT		8 10 13	23		CLAY (CL) yellow-brown, very stiff, moist, with silt						
28					CL							
29–30	S&H		12 20 50/ 4"	42/ 10"		hard, decrease silt	PP		4,500			

LANGAN

Project No.: 770633101 | Figure: B-5a

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31					CL	CLAY (CL) (continued)						
32												
33					CL	SANDY CLAY (CL) yellow-brown, hard, moist, fine sand						
34	S&H		14 26 44	42								
35					SW-SC	SAND with CLAY (SW-SC) yellow-brown, dense, moist, fine- to coarse-grained						
36												
37					SW-SC	SAND wIth CLAY and GRAVEL (SW-SC) yellow-brown, dense, moist, fine to coarse-grained, fine subangular gravel						
38												
39	SPT		24 24 20	44								
40					CL	CLAY (CL) yellow-brown, hard, moist, trace fine sand						
41												
42												
43												
44	S&H		18 19 24	26		hard, with silt, decrease sand content						
45					CL		PP		4,500			
46												
47												
48												
49	S&H		14 18 24	25		very stiff						
50							PP		3,000			
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NAVD 88 Datum.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

PP = Pocket Penetrometer

TV = Torvane

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
- Classification sample taken with Standard Penetration Test sampler
- Undisturbed sample taken with thin-walled tube
- Disturbed sample
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample
- Sample taken with Direct Push or Drive sampler
- Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter - see report text

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No.	Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	SOIL CLASSIFICATION CHART	770633101 Date 11/07/2023 Drawn By AG Checked By JF	B-6

APPENDIX C

DOWNHOLE SUSPENSION LOGGING

NORCAL
GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

November 3, 2016

Mr. Wilson Wong, Project Engineer
LANGAN
4030 Moorpark Ave., Suite 210
San Jose, 19117-1845

Subject: P- and S-Wave Borehole Geophysical Logging Investigation
The Hills at Vallco Project
10333 N. Wolf Rd.
Cupertino, California
NORCAL Job No. NS165088

Attention: Mr. Wilson Wong

This report summarizes the findings of a borehole geophysical investigation performed by NORCAL Geophysical Consultants, Inc. at the subject site for LANGAN. The investigation was conducted on September 8, 2016 by NORCAL Professional Geophysicist William J. Henrich (PGp No. 893). Mr. Daniel Wagstaffe, Field Engineer of LANGAN provided background information, coordination and on-site logistical support.

The purpose of the borehole geophysical investigation was to measure P- and S-wave velocities within unconsolidated alluvium to a depth of 120 feet below ground surface (bgs). These data will be used by others to help characterize subsurface conditions for a proposed building foundation.

1.0 SCOPE

Geophysical borehole logging was conducted in one borehole labeled as Borehole B-1. The borehole was situated in a parking lot northwest of the intersection of Wolf Road and Stevens Creek Boulevard in Cupertino, California. Geophysical logging methods consisted of Suspension P- and S-wave velocity profiling and caliper logging.

2.0 BOREHOLE CONDITIONS

The borehole was advanced with a 5-inch diameter rotary wash drilling method. The borehole penetrated Recent and Quaternary unsaturated and saturated, unconsolidated clay, silt, sand and gravel deposits. Total depth of the borehole was 140-ft bgs. Borehole stability was good with minor sloughing. A 5-inch diameter steel conductor casing was set to 5-ft bgs to prevent caving from loose, unconsolidated fill.

3.0 GEOPHYSICAL LOGGING DESCRIPTIONS

The borehole geophysical investigation was conducted using a digital ***Robertson Geologging,*** **Ltd.** Model ***MICROLOGGER2 System***. This system consisted of a control console, computer logging tools, and winch. The borehole logging tools consisted of a Suspension P- and S-wave velocity and a mechanical three-arm caliper. Complete descriptions of the methodology, data acquisition, data analysis procedures and results for the Suspension P- and S-wave logging are presented in Appendix A.

Caliper logs are a measure of the borehole diameter versus depth. The tool was used both as a survey technique to assess borehole stability and quantify the relative consolidation of alluvium. The caliper tool consists of three interconnected mechanical arms that are spring loaded against the borehole wall. The horizontal deflections of the arms gauge the borehole diameter in units of inches with depth. The logging measurement was made in the uphole direction at a speed of approximately 18-ft per minute. The data sampling rate for this instrument was every 0.2-ft.

4.0 INTERPRETATION and DISCUSSION

The results of our Suspension P- and S-wave velocity and caliper logging are presented on Plate 1. The caliper log shows that the upper 78 feet of the borehole to be highly eroded. This means the diameter of the borehole has expanded beyond the drill bit diameter. Geologically, this may be a zone that contains layers of loose, poorly consolidated sand and gravel.

The average P-wave velocity (Vp) of the majority of the logged borehole section (36-ft down to 120-bgs) has an average of about 6000 fps. The Vp profile shows a sharp velocity reduction beginning at 34-ft up to 10-ft bgs. This low Vp velocity averages about 4000 fps. We interpreted this reduction to be related to alluvial sediments being unsaturated.

The S-wave profile shows that from 10-ft to 26-ft bgs, the alluvium has an S-wave velocity (Vs) that averages 1000 fps. From 26-ft to 72-ft bgs, the Vs ranges from 1000 to 2000 fps. These Vs variations in profile show distinctive peaks (high velocity) and troughs (low velocities). These peaks and troughs are probably related to sedimentary layers with the high Vs related to sand and gravel mixtures: the low Vs related to silt and clays. Below 72-ft bgs, the Vs velocities oscillated around an average Vs of 2000 fps. These oscillations probably relate to relatively thin alternating layers of sand/gravel and silty sand.

5.0 STANDARD CARE

The scope of NORCAL's services for this project consisted of using geophysical logging techniques to measure P- and S-wave velocities. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the level of skill ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate the opportunity to provide our services to LANGAN for this project. If you have any questions, or require additional geophysical services, please do not hesitate to call on us.

Sincerely,

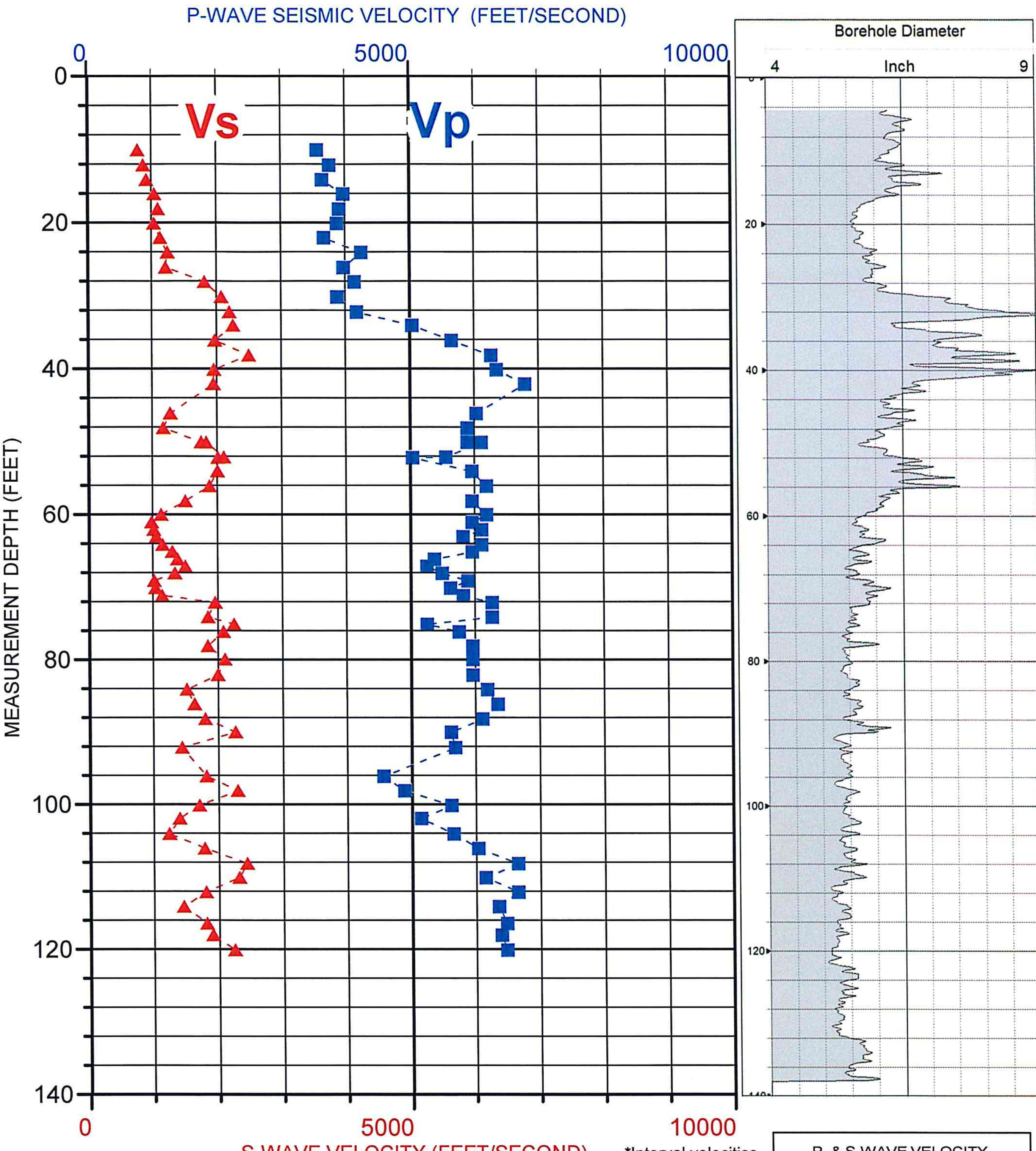
NORCAL Geophysical Consultants, Inc.

REGISTERED GEOPHYSICIST
WILLIAM J. HENRICH
No. 893
STATE OF CALIFORNIA

11/03/2016

William J. Henrich PGp
Professional Geophysicist-893

Enclosures: Plate 1: Suspension P- and S-wave Velocity Profile, Borehole B-1
Appendix A: P- and S-Wave Suspension Velocity Survey



*Interval velocities should be used to calculate elastic moduli values

P- & S-WAVE VELOCITY LEGEND

*Vs- R1-R2 interval

*Vp- R1-R2 interval

11/03/2016

NORCAL

SUSPENSION P- AND S-WAVE VELOCITY PROFILE BOREHOLE B-1

LOCATION: Hills at Vallco, Wolfe Road, Cupertino, CA	
CLIENT: Langan	PLATE 1
JOB #: NS165088	NORCAL GEOPHYSICAL CONSULTANTS INC.
DATE: Sept., 2016	DRAWN BY: W. HENRICH / APPROVED BY: WJH

NORCAL SUSPENSION VELOCITY TABLE FOR BOREHOLE B-1 at Hills at Vallco Project, Wolfe Road, Cupertino, CA

	Interval Velocity Calculations						Direct Velocity Calculations				Depth Reference	
Depth (ft)	VsLeft (m/s)	VsRight (m/s)	VsAvg (m/s)	Vp (m/s)	VsAvg (fps)	Vp (fps)	Vs Ave Near	Vs Ave Far	Vp Near	Vp Far	Near Detector	Far Detector
10.01	240	242	241	1095	790	3570	1625	1296	6724	5569	17.11	15.61
12.04	270	265	267	1154	877	3763	1579	1331	6555	5584	19.14	17.64
14.07	284	280	282	1119	925	3650	1569	1350	6610	5554	21.17	19.67
15.98	314	323	319	1220	1045	3977	1602	1415	6316	5554	23.08	21.58
18.05	342	328	335	1200	1099	3913	1576	1439	5799	5216	25.15	23.65
20.02	326	307	317	1190	1039	3882	1818	1559	5455	4988	27.12	25.62
22.00	347	344	346	1128	1134	3678	2143	1772	5398	4870	29.10	27.60
24.00	375	385	380	1304	1246	4253	2583	2047	4419	4391	31.10	29.60
26.05	368	375	371	1220	1218	3977	3023	2224	5115	4801	33.15	31.65
28.03	564	543	554	1271	1817	4145	3158	2704	5632	5203	35.13	33.63
30.09	620	647	633	1190	2077	3882	3095	2755	5977	5310	37.19	35.69
32.17	652	688	670	1282	2199	4181	2943	2704	6070	5495	39.27	37.77
33.99	694	682	688	1546	2258	5043	2857	2697	6316	5974	41.09	39.59
36.04	595	610	602	1734	1977	5655	2708	2482	6142	6035	43.14	41.64
38.09	758	765	761	1923	2498	6271	2308	2351	7222	6990	45.19	43.69
40.07	605	588	597	1948	1957	6352	1965	1972	6265	6304	47.17	45.67
42.01	600	591	595	2083	1953	6794	1970	1972	6341	6462	49.11	47.61
46.06	395	387	391	1852	1282	6039	2430	2011	6527	6422	53.16	51.66
48.05	355	361	358	1807	1175	5893	2781	2088	6367	6265	55.15	53.65
49.99	532	540	536	1807	1758	5893	2786	2441	6446	6323	57.09	55.59
50.01	564	556	560	1875	1836	6114	2737	2458	6610	6503	57.11	55.61
52.03	652	630	641	1705	2104	5558	2229	2210	6667	6382	59.13	57.63
52.11	602	625	614	1546	2014	5043	2203	2147	5954	5724	59.21	57.71
54.00	615	610	612	1829	2009	5965	1893	1924	6667	6503	61.10	59.60
56.01	577	573	575	1899	1886	6192	1677	1727	6610	6524	63.11	61.61
58.07	449	469	459	1829	1506	5965	1699	1641	6667	6503	65.17	63.67
59.96	349	342	346	1899	1134	6192	1781	1574	6695	6587	67.06	65.56
61.00	298	304	301	1829	986	5965	1848	1524	6695	6524	68.10	66.60
62.01	315	309	312	1875	1023	6114	1769	1513	6582	6483	69.11	67.61
62.99	328	319	323	1786	1061	5823	1757	1527	6667	6462	70.09	68.59
64.10	352	354	353	1875	1158	6114	1753	1562	6420	6362	71.20	69.70
65.05	397	397	397	1829	1302	5965	1703	1588	6610	6462	72.15	70.65
66.08	417	427	422	1648	1385	5375	1812	1683	7059	6587	73.18	71.68
67.03	463	455	459	1613	1505	5260	1898	1793	7123	6587	74.13	72.63
68.01	401	417	409	1685	1341	5496	2047	1809	6933	6545	75.11	73.61
69.04	313	312	312	1807	1024	5893	2374	1807	6753	6545	76.14	74.64
70.03	314	323	319	1724	1046	5622	2600	1908	6842	6524	77.13	75.63
71.07	352	347	350	1786	1147	5823	2574	1991	6842	6587	78.17	76.67
72.05	581	615	598	1923	1962	6271	2977	2628	6842	6716	79.15	77.65
74.03	560	568	564	1923	1850	6271	2857	2525	6842	6716	81.13	79.63
75.04	685	688	687	1613	2252	5260	2751	2615	6638	6265	82.14	80.64
76.08	647	625	636	1765	2086	5755	2847	2635	6842	6565	83.18	81.68
78.06	560	566	563	1829	1847	5965	2400	2239	6872	6650	85.16	83.66
79.88	641	647	644	1829	2112	5965	2349	2288	6842	6629	86.98	85.48
82.06	605	615	610	1829	2001	5965	2364	2263	6872	6650	89.16	87.66
84.05	466	459	462	1899	1517	6192	2393	2110	6695	6587	91.15	89.65
86.09	498	498	498	1948	1635	6352	2311	2106	6420	6422	93.19	91.69
88.11	549	547	548	1875	1799	6114	2241	2121	6933	6738	95.21	93.71
89.95	682	701	691	1724	2268	5622	2203	2214	7222	6782	97.05	95.55
92.10	439	439	439	1744	1439	5688	2407	2076	6903	6587	99.20	97.70
96.05	551	556	554	1402	1816	4571	1921	1896	6047	5630	103.15	101.65

98.05	694	708	701	1500	2300	4891	2037	2093	7059	6402	105.15	103.65
100.08	507	532	519	1724	1704	5622	2210	2051	6367	6190	107.18	105.68
101.90	434	419	426	1579	1399	5149	2680	2214	7156	6565	109.00	107.50
104.01	375	381	378	1734	1240	5655	3059	2258	6710	6442	111.11	109.61
106.01	534	554	544	1852	1784	6039	2694	2390	6500	6402	113.11	111.61
108.12	743	743	743	2041	2436	6655	2335	2362	6710	6716	115.22	113.72
110.03	714	698	706	1887	2316	6153	2185	2224	7027	6816	117.13	115.63
112.03	549	549	549	2041	1803	6655	2476	2276	7647	7405	119.13	117.63
114.05	424	463	443	1948	1455	6352	2796	2253	8211	7697	121.15	119.65
116.38	549	556	553	1987	1813	6479	2626	2370	7959	7569	123.48	121.98
118.00	573	591	582	1961	1908	6394	2301	2186	8062	7611	125.10	123.60
120.06	673	694	684	1987	2243	6479	2042	2082	7959	7569	127.16	125.66

Vs & Vp Interval Velocities
see red triangle & blue squares
on Plate 1

COLUMN HEADER LEGEND

DEPTH: Reference point of the Interval Velocity Measurement

INTERVAL Vs and Vp VELOCITIES

- VsLeft (m/s) S-wave velocities determined from left strike; differemce in near and far detector arrival times
- VsRight (m/s) S-wave velocities determined from right strike; difference in near and far detector arrival times
- VsAvg (m/s) S-wave velocity average in meters/second
- Vp (m/s) P-wave Velocity in Meters/second
- Vs Avg (fps) S-wave velocity average in feet per second
- Vp (fps) P-wave velocity average in feet per second

DIRECT TRAVEL VELOCITIES:

- Vs Ave Near Shear wave velocity = inline distance from source to lower detector divided by travel time measurements at the lower detector
- Vs Ave Far Shear wave velocity = inline distance from source to upper detector divided by travel time measurements at the upper detector
- Vp Near P-wave velocity = inline distance from source to the lower detector divided by travel time measurement at the lower detector
- Vp Far P-wave velocity = inline distance from source to the upper detector divided by travel time measurement at the upper detector

OFF SET DEPTH MEASUREMENT POINT:

- Near Detector Depth reference for source to near detector velocity value; mid-point
- Far Detector Depth reference for source to far detector velocity value, mid-point

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

Appendix A:

P- and S-WAVE SUSPENSION VELOCITY SURVEY

APPENDIX A

P and S-WAVE SUSPENSION VELOCITY SURVEY

The Suspension logger is a highly specialized downhole tool that measures P- and S-wave velocities at discrete depths. The following presents a narrative on its operation and the data reduction procedures we use in analyzing the data. Also presented are the velocity profiles and tabulated velocity data acquired in Borehole B-1.

METHODOLOGY

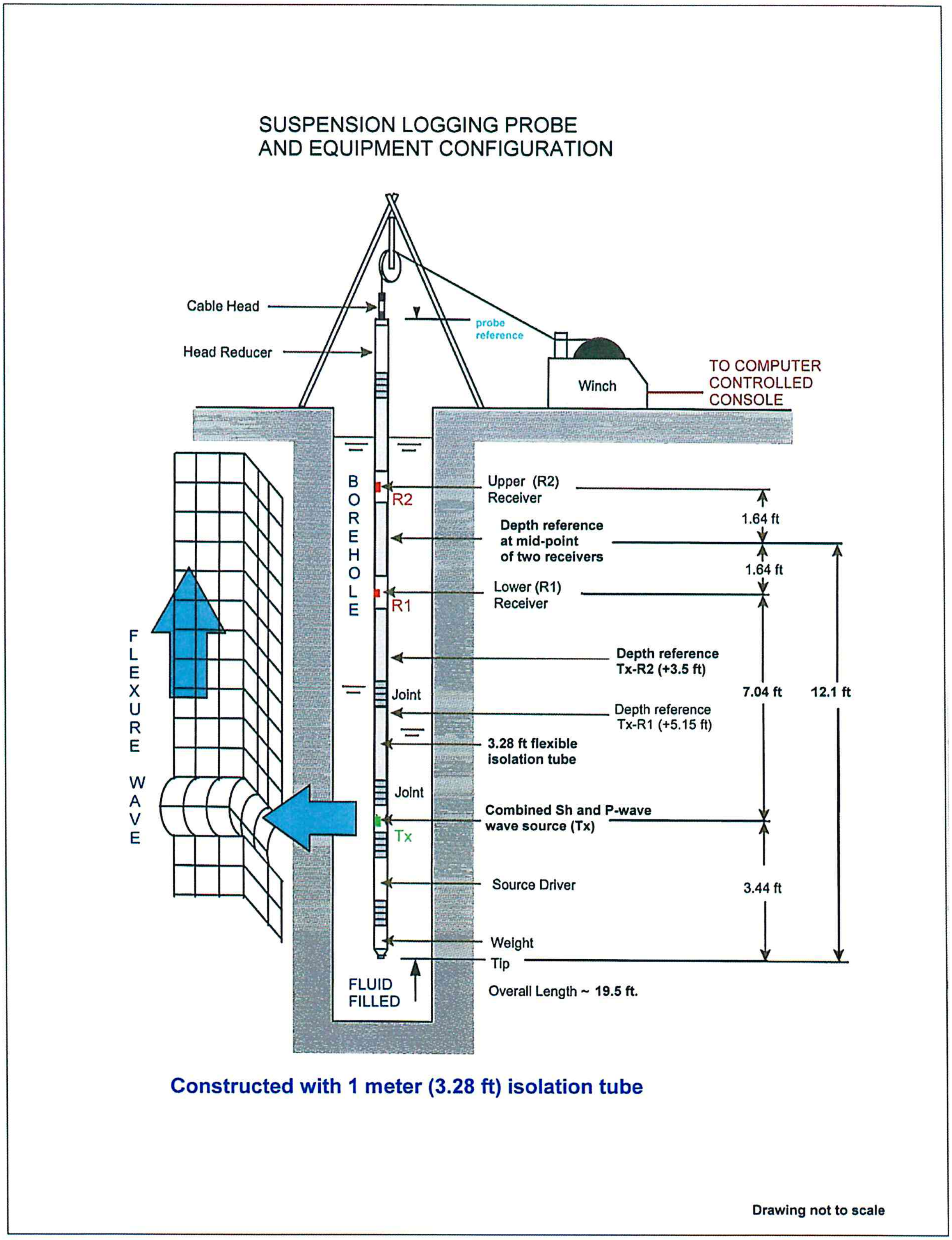
We measured downhole compressional (P-) and shear (S-) wave velocities using a Robertson Geologging, Ltd. digital suspension logging system. A schematic diagram depicting the probe configuration and equipment attachment is shown in Figure 1. The suspension logging tool is equipped with a dipole seismic energy source located near the base of the probe and a pair of geophones (detectors R-1 and R-2) located within the middle to the upper sections of the probe. The distance from the energy source to the first (near detector) geophone was 7.04, feet (2.14 meters) when assembled with a detachable 1-meter isolation tube. The in-line distance between the geophone pair was 3.28-feet (1.0 meter). Each geophone contains one horizontal and one vertical oriented element. The horizontal geophone elements preferentially record shear wave motion. The vertical geophone elements record first arriving P-wave energy.

Suspension seismic data are collected at discrete depths in the fluid-filled portion of the borehole. At each measurement depth, the energy source is activated via commands from the surface control console. This activation causes a metal solenoid to strike a plate (anvil) mounted inside the probe housing. This energy transmits through the fluid to the borehole wall which produces a seismic wave (“flexure”) in the adjacent formation. As this wave propagates radially into the formation a physical interaction between the seismic wave and the borehole wall creates tube waves together with a refracted compressional P-wave that travels up the borehole to the two recording geophones.

When assembled with a 1-meter isolation tube, the suspension logging tool measures approximately 20-ft in length (Figure 1). The measuring point of the tool is taken at the center of the pair of receiver geophones. This measuring point is approximately 12-ft from the probe tip. Therefore, the maximum depth of a suspension logging survey given a non-sloughing borehole will always be reported as 12feet less than the total depth of the borehole.

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

Figure 1. Suspension logger schematic diagram



SURVEY CONDITIONS AND DATA ACQUISITION PROCEDURES

We measured seismic suspension velocities at stationary 1.0 to 2.0-ft measurement intervals. The finer interval spacing being taken across alluvial layers in some cases. The survey began near the bottom of the borehole (PS- measuring point at approximately 120-ft bgs and proceeded upward to 10-ft bgs. At each measurement station, we cycled the energy source to fire 2 times in succession into each of the geophone elements. This cycling stacks the seismic energy resulting in an improved signal-to-noise ratio. We also recorded S-wave data using a 600 KHz low pass filter. This filtering reduces high frequency interference from the onset of earlier arriving P-wave energy on the S-wave channels. We recorded P-wave waveforms using a 20 KHz low pass filter.

DATA ANALYSIS

Suspension P- and S-wave velocities were calculated with the interpretation computer software programs ***PSLogger Application*** Version 1.121 and ***PSLOG Analysis*** Version 1.0.001 both published by ***Robertson Geologging, Ltd.*** (2009). Example suspension waveform records from Borehole B-1 at a depth of 79.88-ft below ground is presented in Figure 2. This suspension waveform records show six detector (geophone) traces. The upper four waveform traces are related to S-wave velocity arrival time measurements determined at the "far" (*srf* and *slf*) and "near" (*srn* and *sln*) horizontal detectors; the lower two waveform traces are related to P-wave velocity arrival time measurements determined at the far and near vertical detectors. The far and near detector labels refer to the relative in-line distances of the geophone detectors to the energy source.

Referring to the P- and S-wave suspension record in Figure 2, the red traces (cycle 1) are created by a right strikes or impacts of the dipole source (anvil) to the probe housing *(srf* and *srn*); the green traces (cycle 2) are created from left strikes (*slf* and *sln*) of the dipole source. By superimposing and pairing the respective left and right strike waveform traces, phase reversals associated with the arrival times of the S-wave energy can be identified. These arrival times are presented as open dots on the waveform plot. P-wave records are associated with the lower two waveform traces (blue color). With P-wave energy, the direction of the dipole strike can be in either direction but requires another recording cycle. P-wave arrival times are determined by noting the first breaks on the set of near and far detector traces. Interpreted arrival times are shown as open dots on the waveforms at a position corresponding to the onset of the first break (either up or down). Note that at a minimum, a complete suspension waveform record requires at least three recording cycles.

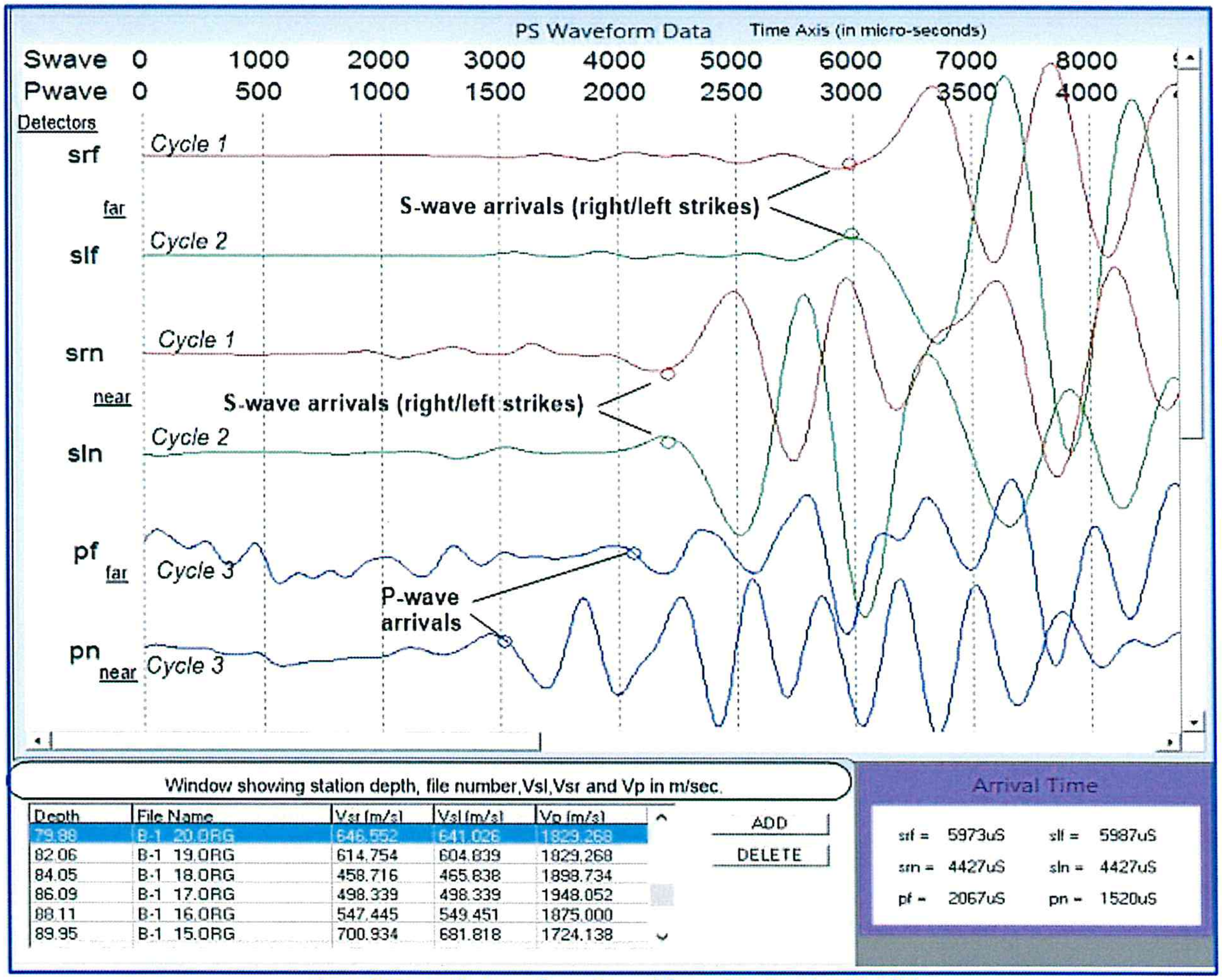


Figure 2 Example Waveform Record from B-1, Depth 79.88-ft bgs

All suspension waveform records were analyzed for P- and S-wave arrival times in this manner

P- and S-WAVE VELOCITY TABLE

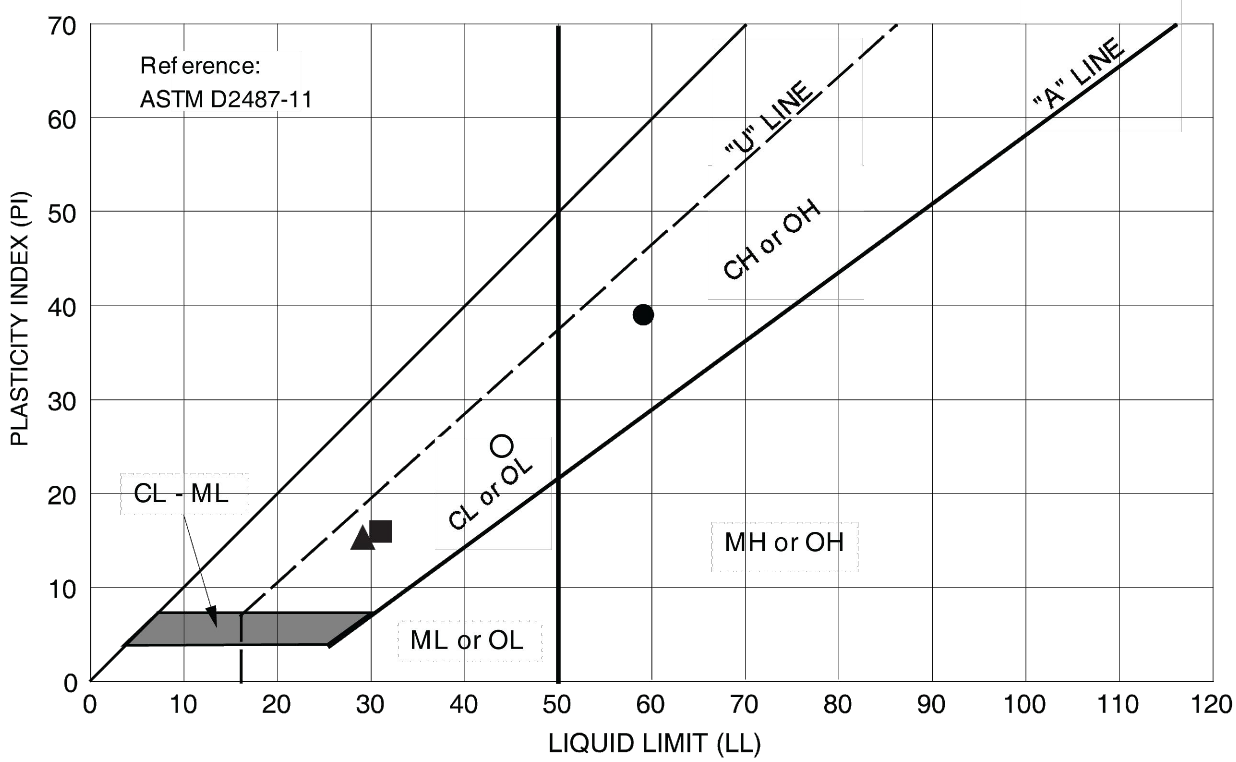
The suspension interpretation program (***PSLOG Analysis***) computes interval seismic P- and S-wave velocities in meters per second. Interval velocities are calculated by dividing the detector spacing (Far distance minus the near detector distance = 1 meter) by the difference in interpreted arrival times in microseconds at the two detectors. Note, that two separate interval S-wave velocities (created from the dipole source striking left then right) are calculated at each measurement depth. In the attached table at the end of this appendix, these are tabulated as ***Vs left*** and ***Vs right***. These two interval Vs velocities are then averaged (***Vs Ave***) in a separate column for each measurement station. We export these velocity data and arrival times to **EXCEL** (Microsoft Corporation) computer program to create a spreadsheet that lists the various interval velocities and measurement depths.

Within the spreadsheet we converted P- and S-wave interval velocities in meters/sec to feet per second. These two columns, ***VsAve (fps)*** and ***Vp (fps)*** appear shaded on the following spread sheet. For comparison purposes we also computed what we refer to as Direct Velocities for each wave type at the near and far detectors. These are calculated as the in-line distance between the dipole source and respective detectors divided by interpreted arrival times. The Direct Velocities and are labeled as ***Vs Ave Near, Vs Ave Far, Vp Near and Vp Far*** in the column headers. Note, these direct velocities have measuring points that are midway between the source and two respective detectors. The direct velocities are actually a few feet lower than the interval velocity measurement depth though these are presented along the same row as the interval velocity measurement depth.

The purpose of deriving direct velocities is to generally compare these to the interval velocities. If there were significant differences we would have reexamined interpretation of arrival times and produced different results. In this survey the Interval and Direct Velocities are comparable in general. Variations in Interval versus Direct velocity measurements are due to averaging direct velocities over a larger cross-section than the interval velocities, thin layer effects within the alluvial sediments and borehole diameter variations (see Borehole Diameter on Plate 1). The two latter effects are geometric as these can slightly alter the phase or scatter seismic signals causing differences observed in direct versus interval velocities along some sections within this borehole.

APPENDIX D

LABORATORY DATA



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 11 feet	CLAY with GRAVEL (CH), brown to dark brown	20.0	59	39	--
■	B-1 at 25.5 feet	SANDY CLAY with GRAVEL (CL), brown to yellow-brown	13.4	31	16	--
▲	B-2 at 85 feet	CLAYEY GRAVEL with SAND (GC), yellow-brown	12.2	29	15	--
○	B-4 at 6 feet	CLAY (CL), gray-brown	--	44	25	--

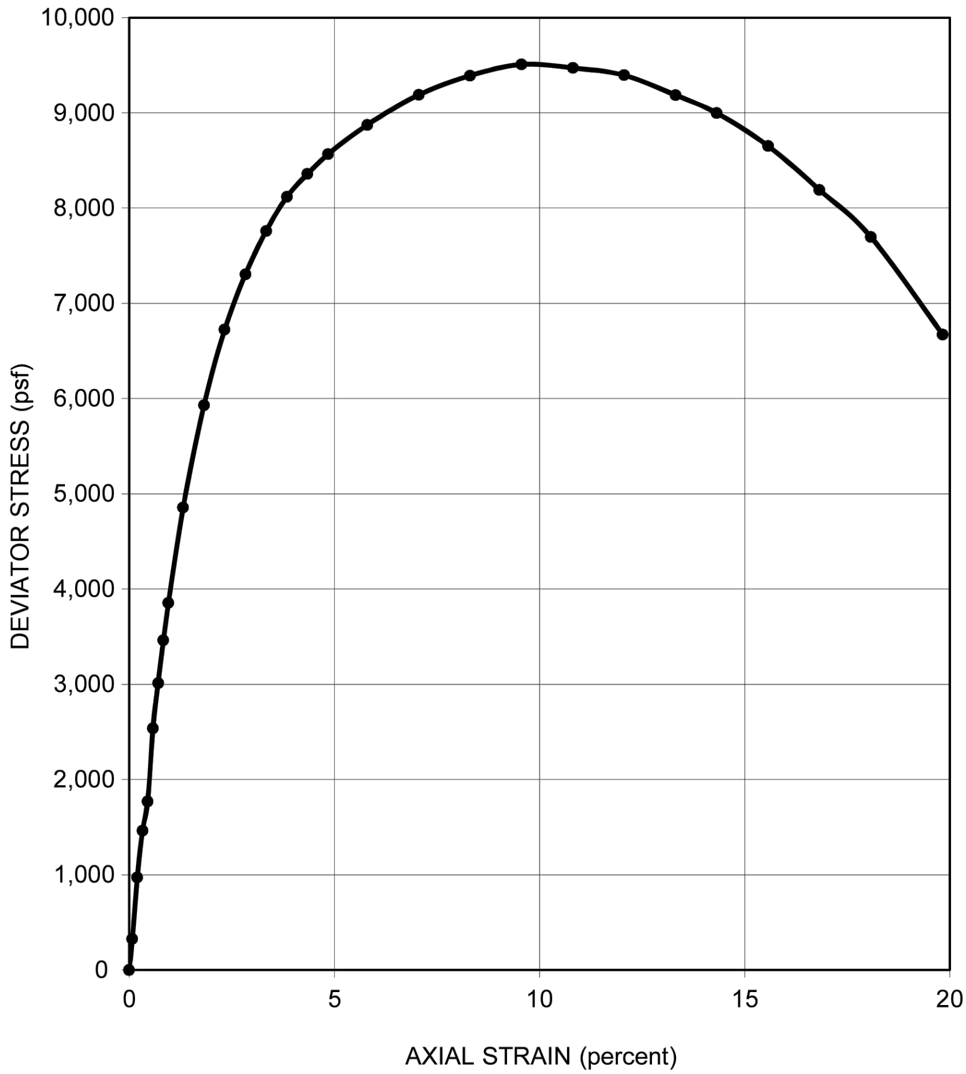
LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project
THE RISE
CUPERTINO
SANTA CLARA COUNTY CALIFORNIA

Figure Title
PLASTICITY CHART

Project No. 770633101
Date 11/07/2023
Drawn By AG
Checked By JF

Figure
D-1

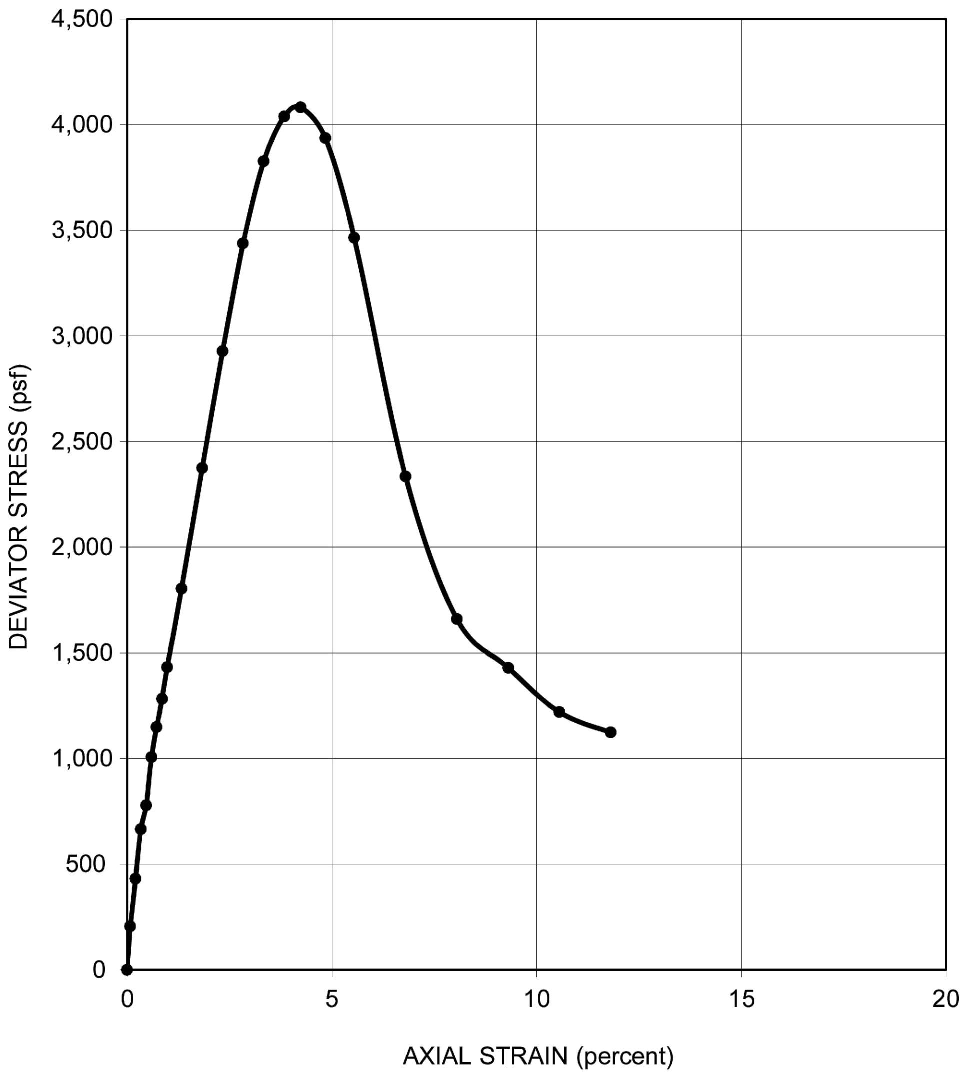


SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	4,750	psf
DIAMETER (in.)	2.39	HEIGHT (in.)	5.72	STRAIN AT FAILURE	9.6	%
MOISTURE CONTENT		20.0	%	CONFINING PRESSURE	600	psf
DRY DENSITY		111	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY with GRAVEL (CH), yellow-brown			SOURCE	B-1 at 10.5 feet	

LANGAN

Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project: THE RISE, CUPERTINO, SANTA CLARA COUNTY, CALIFORNIA	Figure Title: UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure D-2



SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	2,040	psf
DIAMETER (in.)	2.40	HEIGHT (in.) 5.7		STRAIN AT FAILURE	4.2	%
MOISTURE CONTENT		12.0	%	CONFINING PRESSURE	3,700	psf
DRY DENSITY		127	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAYEY SAND (SC), brown			SOURCE	B-1 at 31 feet	

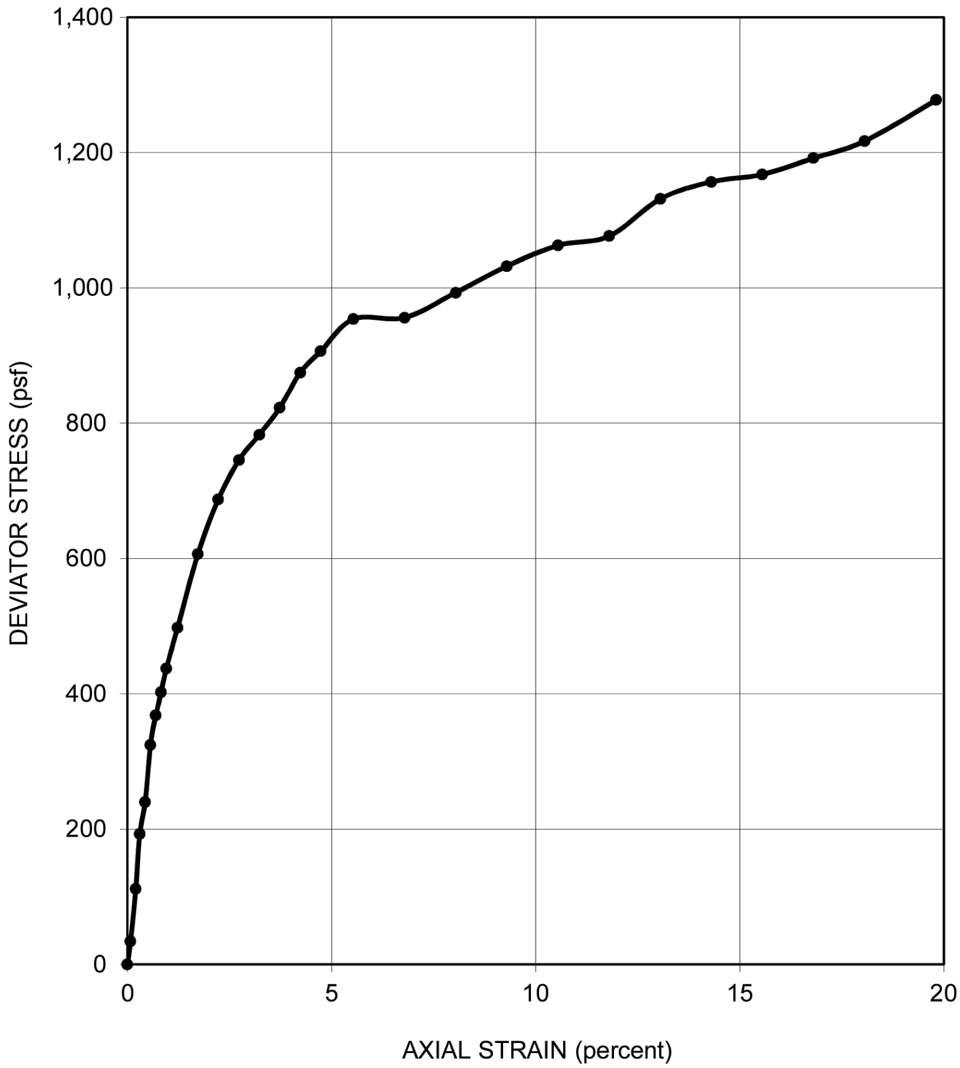
LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project
THE RISE
CUPERTINO
SANTA CLARA COUNTY CALIFORNIA

Figure Title
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Project No. 770633101
Date 11/07/2023
Drawn By AG
Checked By JF

Figure
D-3



SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	640	psf
DIAMETER (in.) 2.40	HEIGHT (in.) 5.52		STRAIN AT FAILURE	19.8	%
MOISTURE CONTENT	18.0	%	CONFINING PRESSURE	9,100	psf
DRY DENSITY	112	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), brown			SOURCE	B-1 at 75.5 feet

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project

THE RISE

CUPERTINO

SANTA CLARA COUNTY CALIFORNIA

Figure Title

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Project No. 770633101

Date 11/07/2023

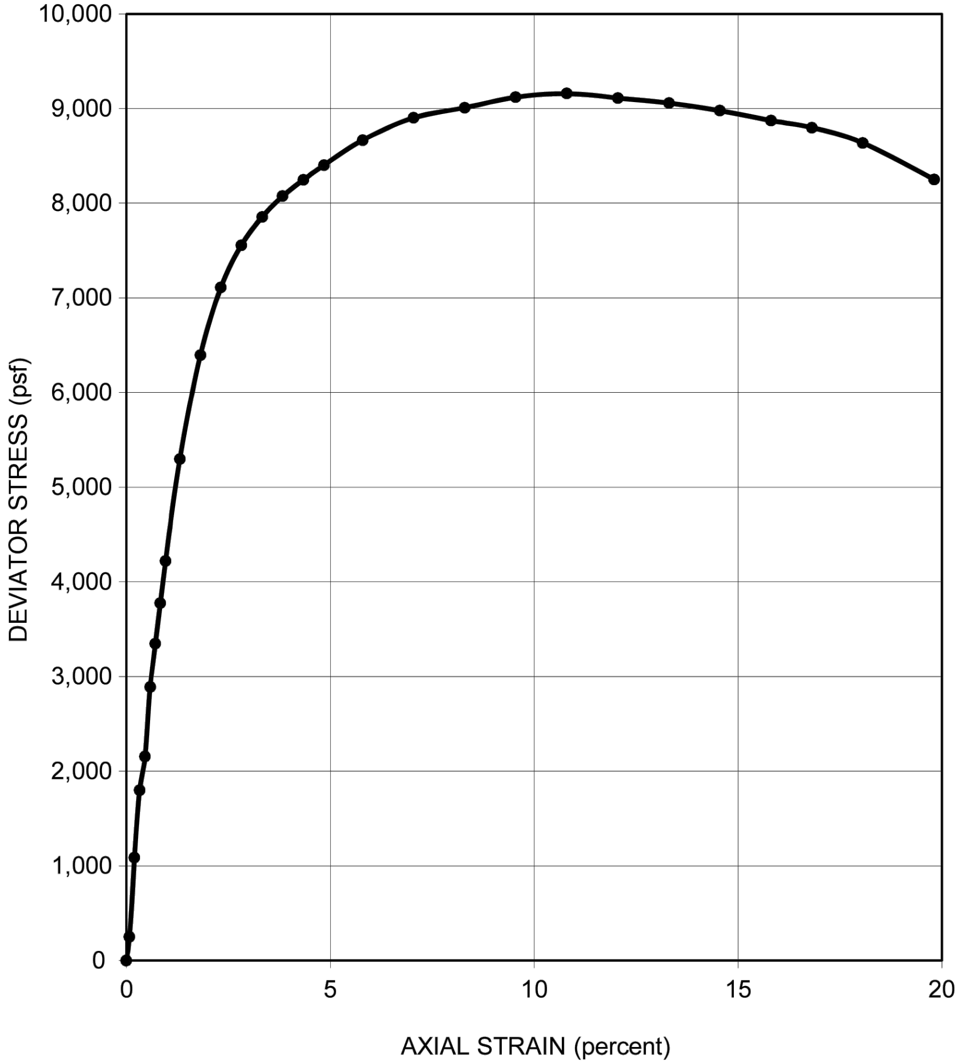
Drawn By AG

Checked By JF

Figure

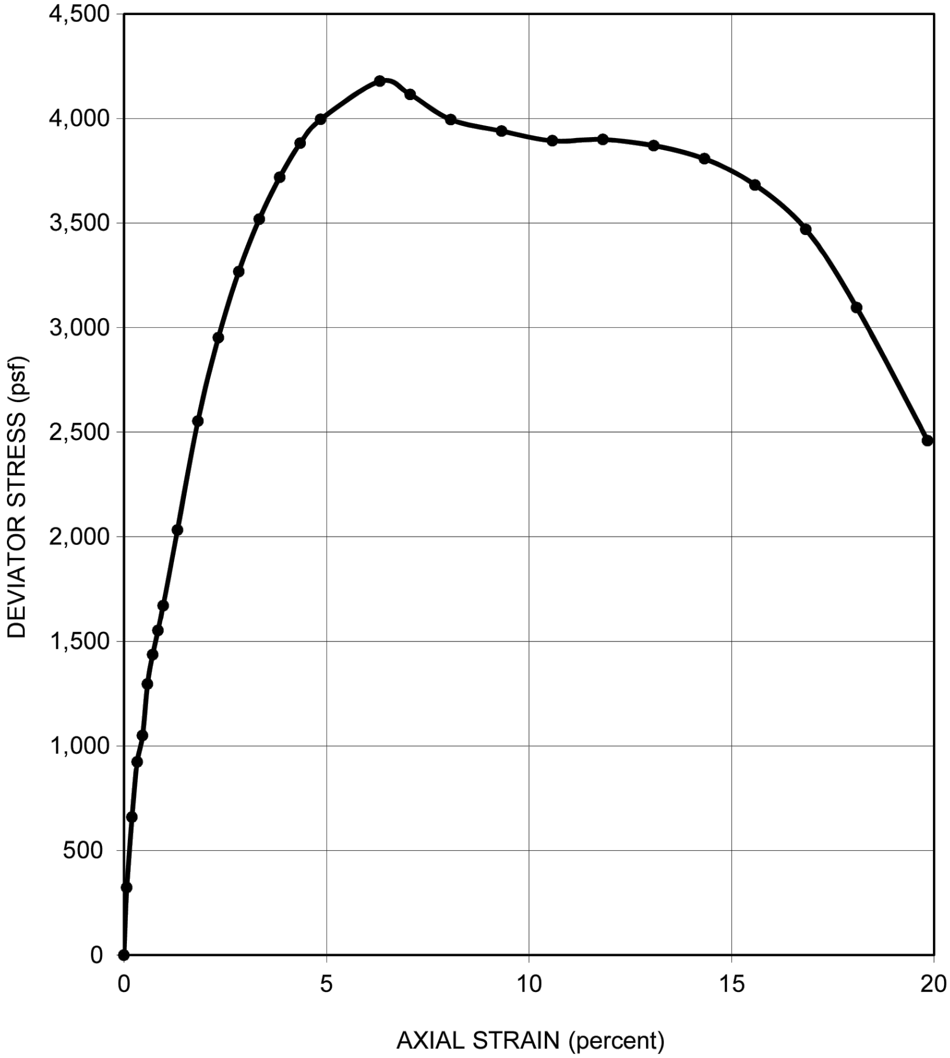
D-4

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 09:27 User: agekas Style Table: Langan.stb Layout: TXUU3



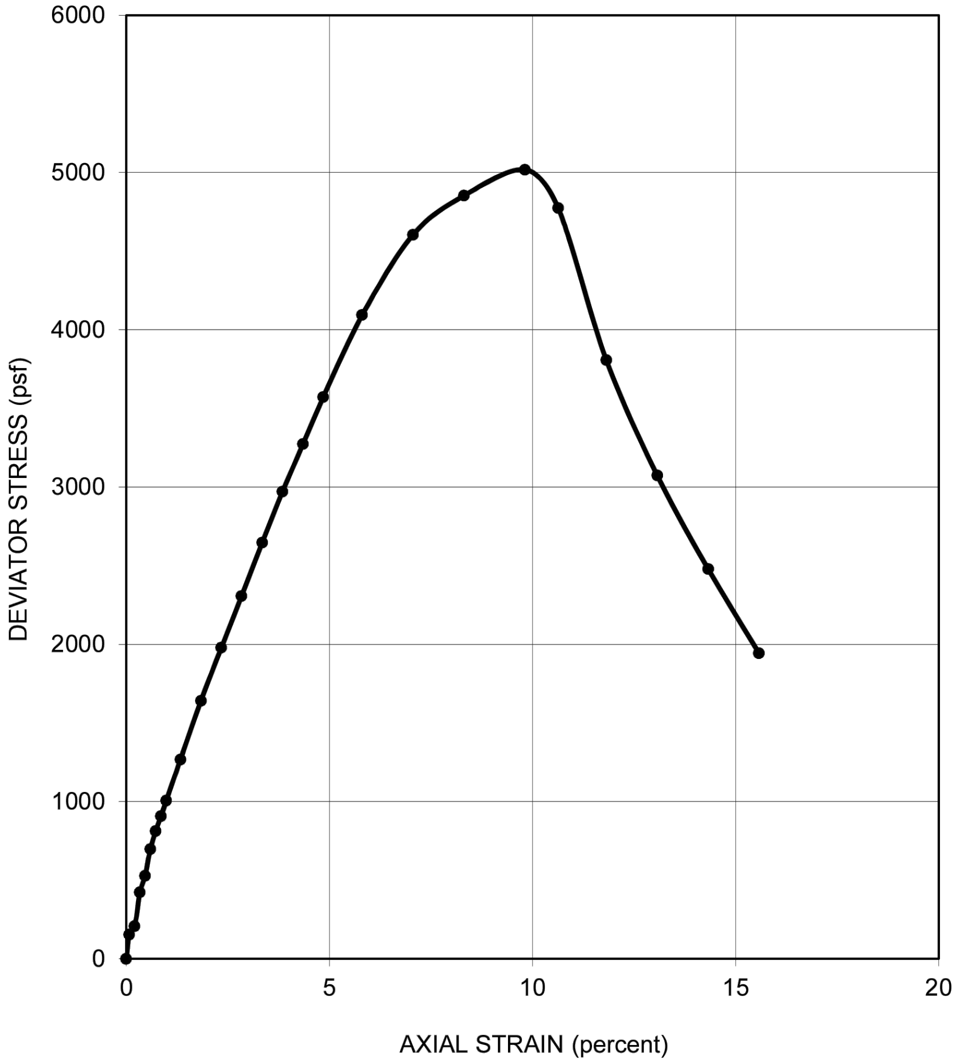
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	4,580	psf
DIAMETER (in.)	2.40	HEIGHT (in.) 5.61		STRAIN AT FAILURE	10.8	%
MOISTURE CONTENT		18.6	%	CONFINING PRESSURE	1,900	psf
DRY DENSITY		113	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY with SAND (CL), dark brown			SOURCE	B-2 at 16 feet	

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure D-5



SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	2,090	psf
DIAMETER (in.)	2.40	HEIGHT (in.)	5.72	STRAIN AT FAILURE	6.3	%
MOISTURE CONTENT		23.1	%	CONFINING PRESSURE	12,100	psf
DRY DENSITY		105	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CL), brown			SOURCE	B-2 at 100.5 feet	

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure D-6

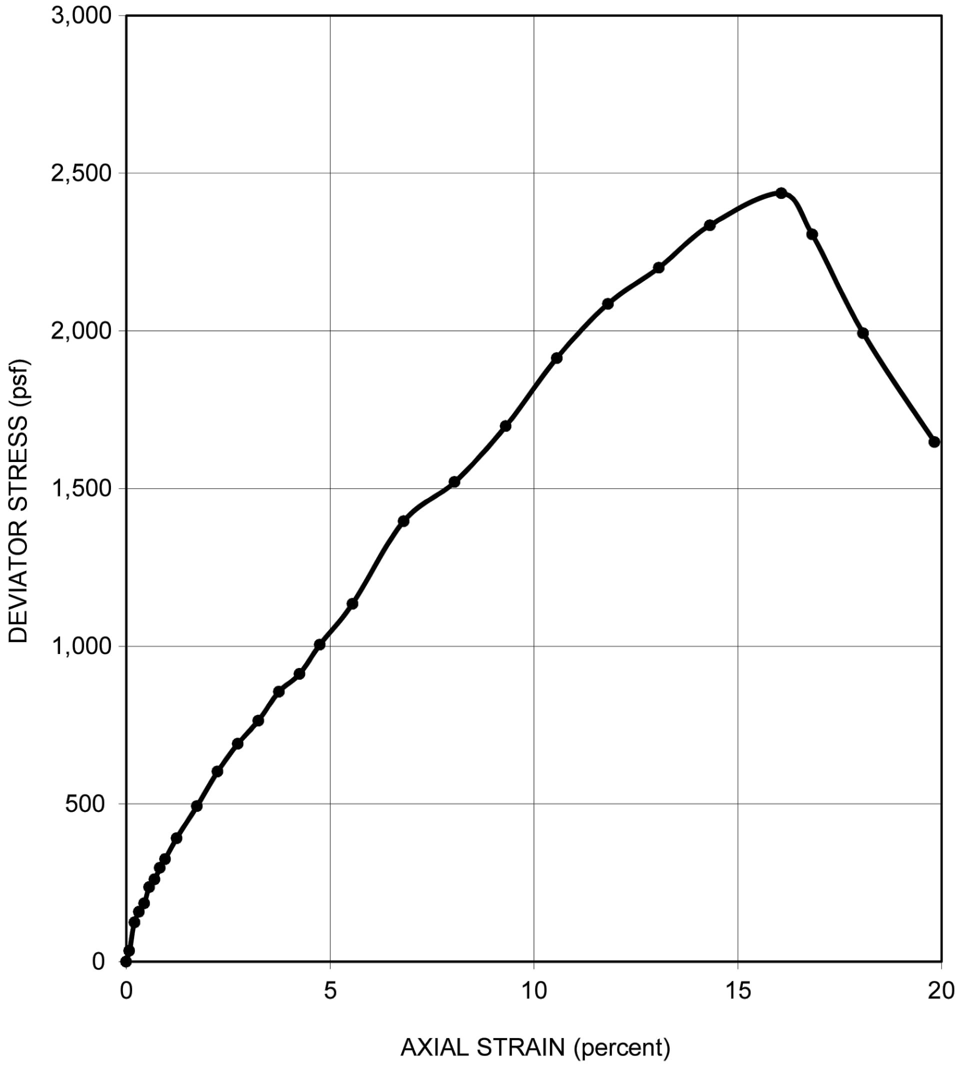


SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	2,510	psf
DIAMETER (in.) 2.42	HEIGHT (in.) 5.41		STRAIN AT FAILURE	9.8	%
MOISTURE CONTENT	21.4	%	CONFINING PRESSURE	2,300	psf
DRY DENSITY	104	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY with SAND (CL), brown		SOURCE	B-4 at 39.5 feet	

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No.	Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	770633101 Date 11/07/2023 Drawn By AG Checked By JF	D-7

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 10:21 User: agekas Style Table: Langan.stb Layout: TXUU6



SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	1,220	psf
DIAMETER (in.) 2.40	HEIGHT (in.) 5.42		STRAIN AT FAILURE	16.1	%
MOISTURE CONTENT	21.8	%	CONFINING PRESSURE	10,100	psf
DRY DENSITY	105	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown		SOURCE	B-4 at 84.5 feet	

LANGAN

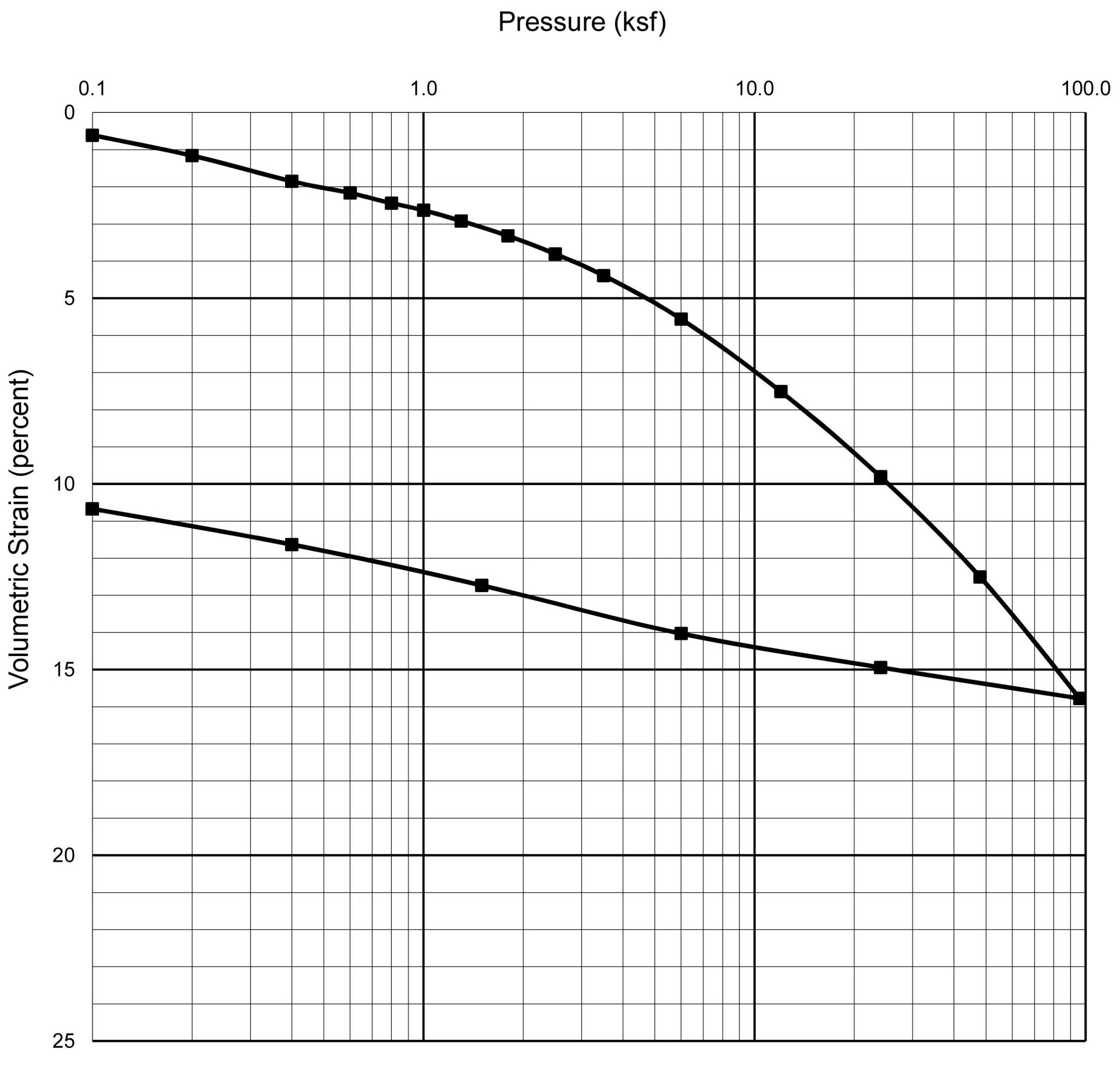
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project: **THE RISE**
CUPERTINO
SANTA CLARA COUNTY CALIFORNIA

Figure Title: **UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST**

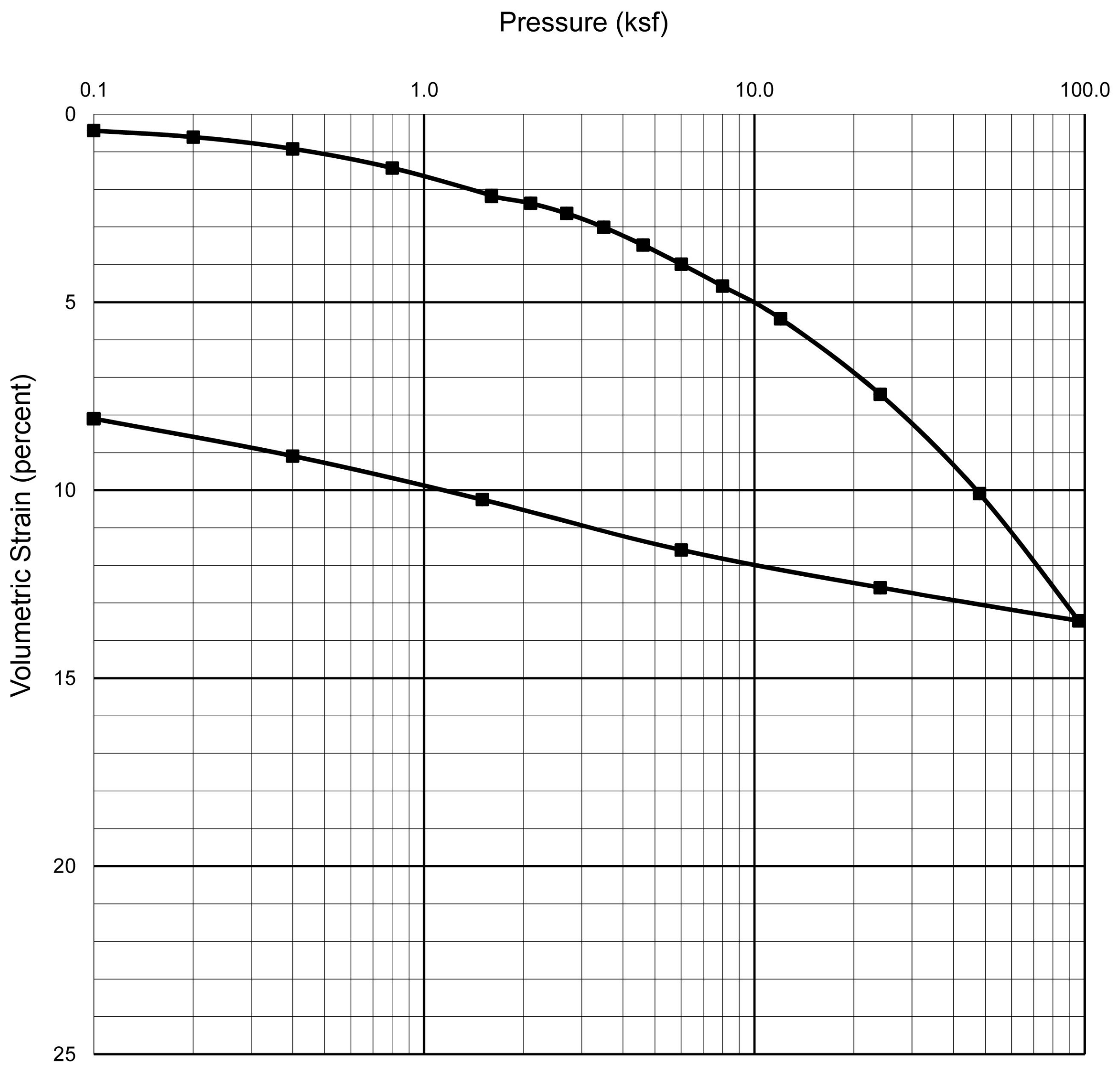
Project No. 770633101
Date 11/07/2023
Drawn By AG
Checked By JF

Figure **D-8**



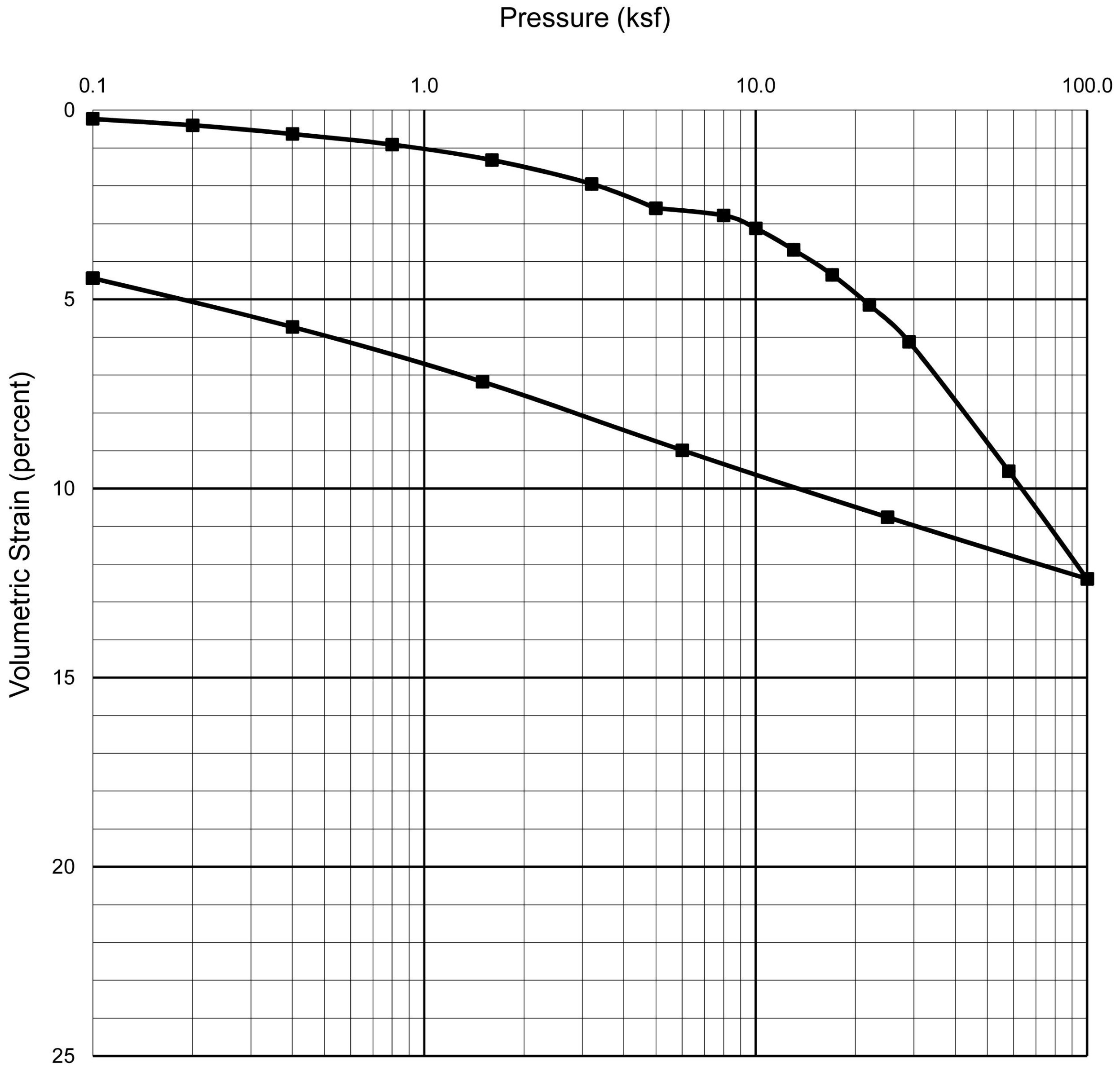
Sampler Type: Sprague & Henwood				Condition		Before Test			After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	17.7	%	w_f	12.6	%
Overburden Pressure, p_o	3,120	psf		Void Ratio	e_o	0.50		e_f	0.34	
Preconsol. Pressure, p_c	8,000	psf		Saturation	S_o	95	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.10			Dry Density	γ_d	112	pcf	γ_d	126	pcf
LL - -		PL - -		PI - -				G_s	2.70	(assumed)
Classification	SANDY CLAY with GRAVEL (CL), yellow-brown							Source	B-1 at 26 feet	

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title CONSOLIDATION TEST REPORT	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure D-9



Sampler Type: Sprague & Henwood			Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	17.2 %		w_f	14.7 %
Overburden Pressure, p_o	4,920 psf		Void Ratio	e_o	0.52		e_f	0.40
Preconsol. Pressure, p_c	10,700 psf		Saturation	S_o	89 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.10		Dry Density	γ_d	111 pcf		γ_d	121 pcf
LL - -	PL - -		PI - -				G_s 2.70	(assumed)
Classification SANDY CLAY (CL), brown							Source	B-2 at 41 feet

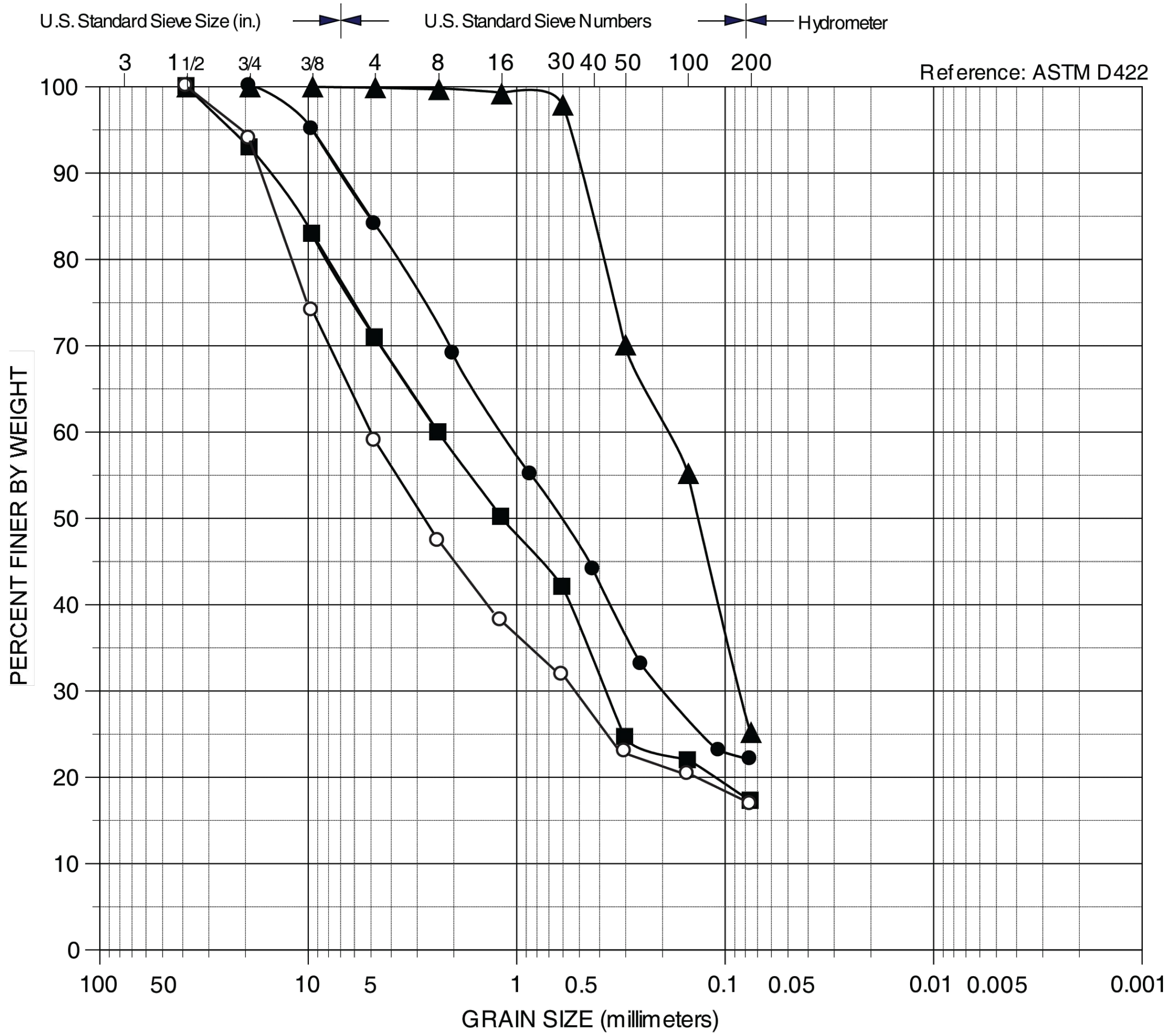
LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title CONSOLIDATION TEST REPORT	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure D-10



Sampler Type: Sprague & Henwood				Condition		Before Test			After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	20.7	%	w_f	19.6	%
Overburden Pressure, p_o	8,940	psf		Void Ratio	e_o	0.60		e_f	0.53	
Preconsol. Pressure, p_c	18,500	psf		Saturation	S_o	93	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.12			Dry Density	γ_d	105	pcf	γ_d	110	pcf
LL	- -	PL - -		PI - -				G_s	2.70	(assumed)
Classification	CLAY (CL), brown							Source	B-4 at 74.5 feet	

LANGAN Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title CONSOLIDATION TEST REPORT	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure D-11

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 11:01 User: agekas Style Table: Langan.stb Layout: CONSOL3

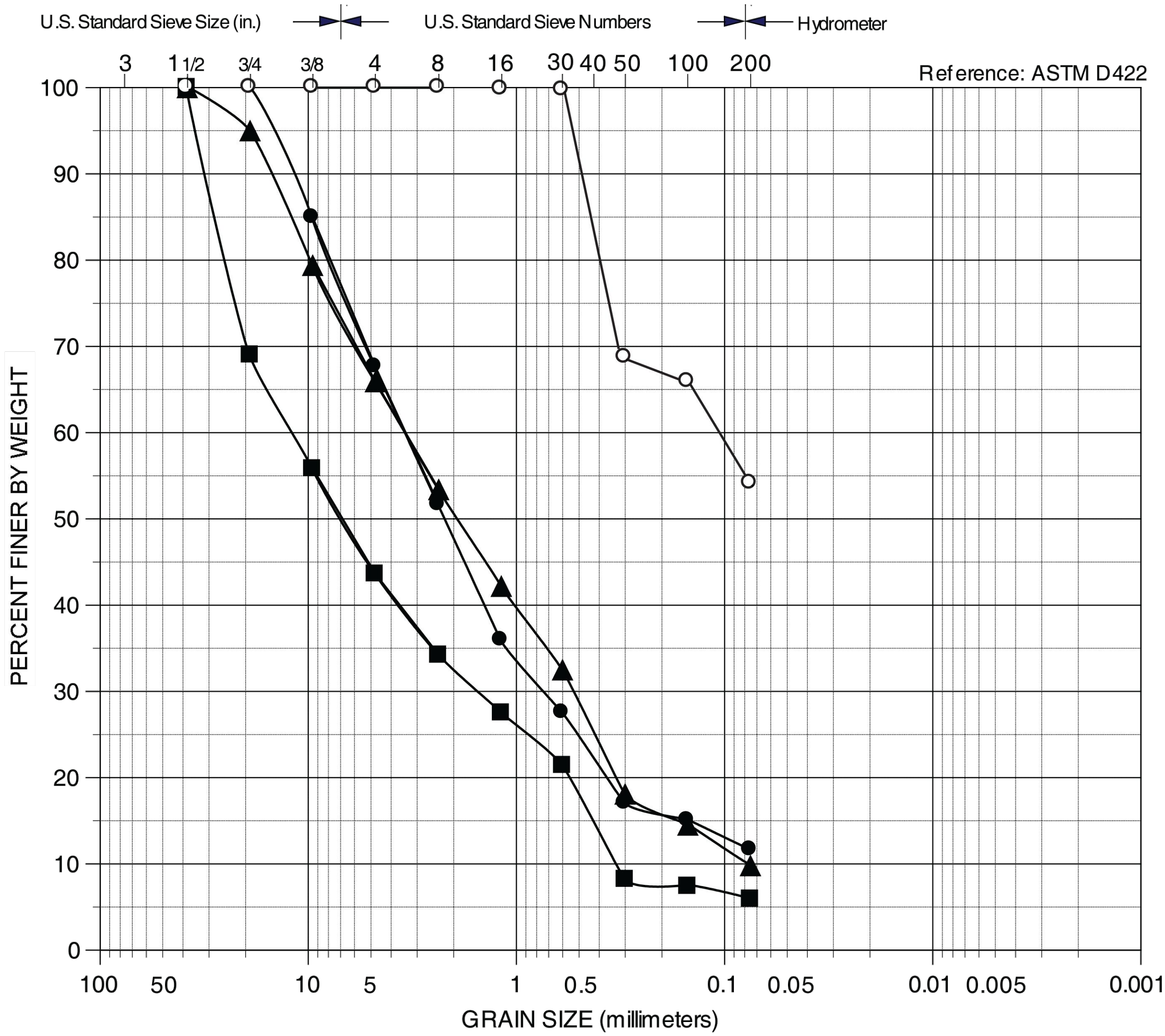


% Gravel		% Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Symbol	Sample Source	Classification
●	B-1 at 31 feet	CLAYEY SAND with GRAVEL (SC), brown
■	B-1 at 40.5 feet	CLAYEY SAND with GRAVEL (SC), brown
▲	B-2 at 45 feet	SILTY SAND (SM), yellow-brown
○	B-2 at 55 feet	CLAYEY SAND with GRAVEL (SC), brown

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title		Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	PARTICLE SIZE ANALYSIS	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	D-12



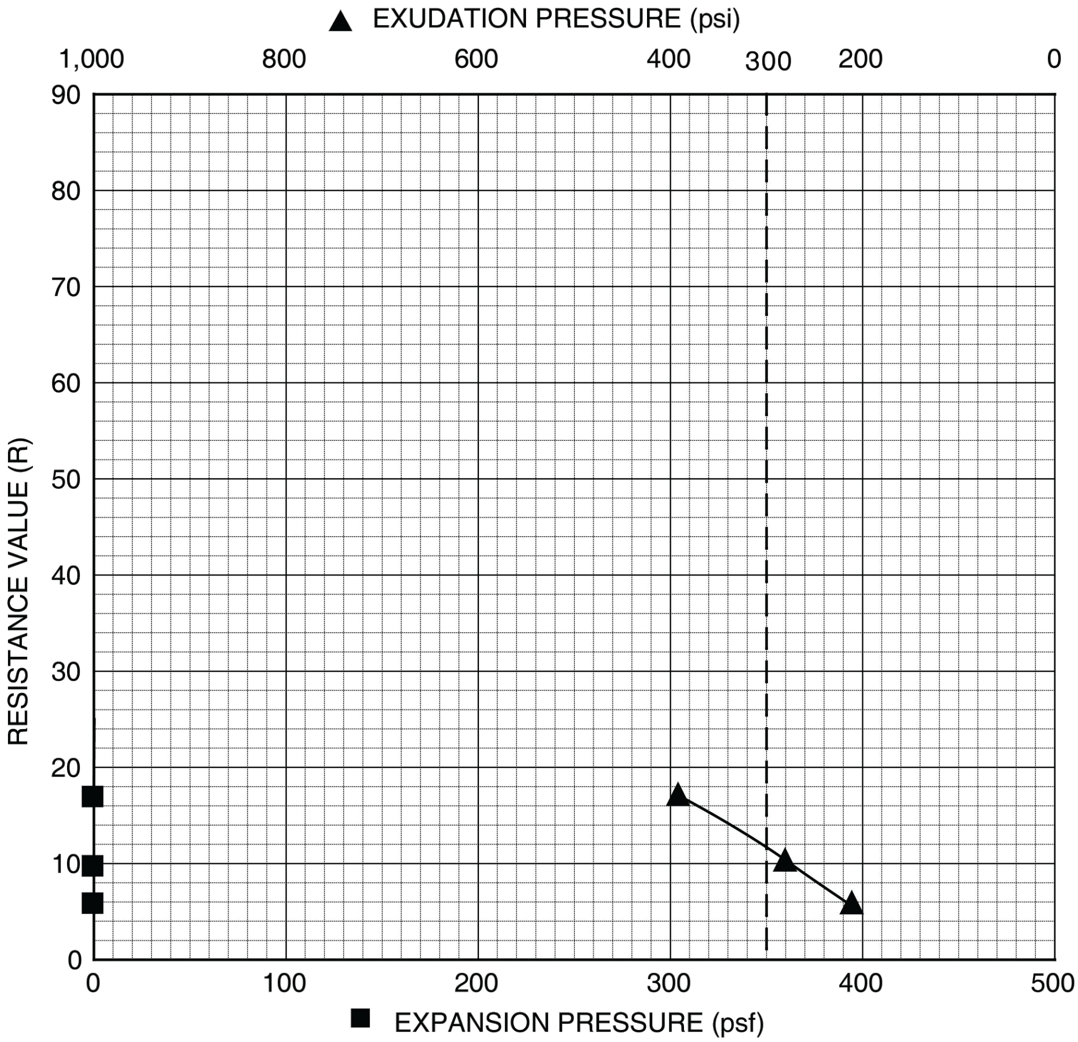
% Gravel		%Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Symbol	Sample Source	Classification
●	B-4 at 18.5 feet	SAND with CLAY and GRAVEL (SW-SC), brown
■	B-4 at 44 feet	GRAVEL with SILT and SAND (GP-GM), brown
▲	B-4 at 48.5 feet	SAND with SILT and GRAVEL (SP-SM), brown
○	B-5 at 23.5 feet	SANDY SILT (ML), light brown

LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project	Figure Title	Project No.	Figure
THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	PARTICLE SIZE ANALYSIS	770633101 Date: 11/07/2023 Drawn By: AG Checked By: JF	D-13

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 11:08 User: agekas Style Table: Langan.stb Layout: PARTICLE CHART2



Specimen ID:	A	B	C	D
Water Content (%)	15.3	14.0	13.2	--
Dry Density (pcf)	115.4	119.8	121.2	--
Exudation Pressure (psi)	205	281	390	--
Expansion Pressure (psf)	0.00	0.00	0.00	--
Resistance Value (R)	6	10	17	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-1 at 0 to 5 feet	CLAY with GRAVEL (CH), brown to dark brown	--	--	12

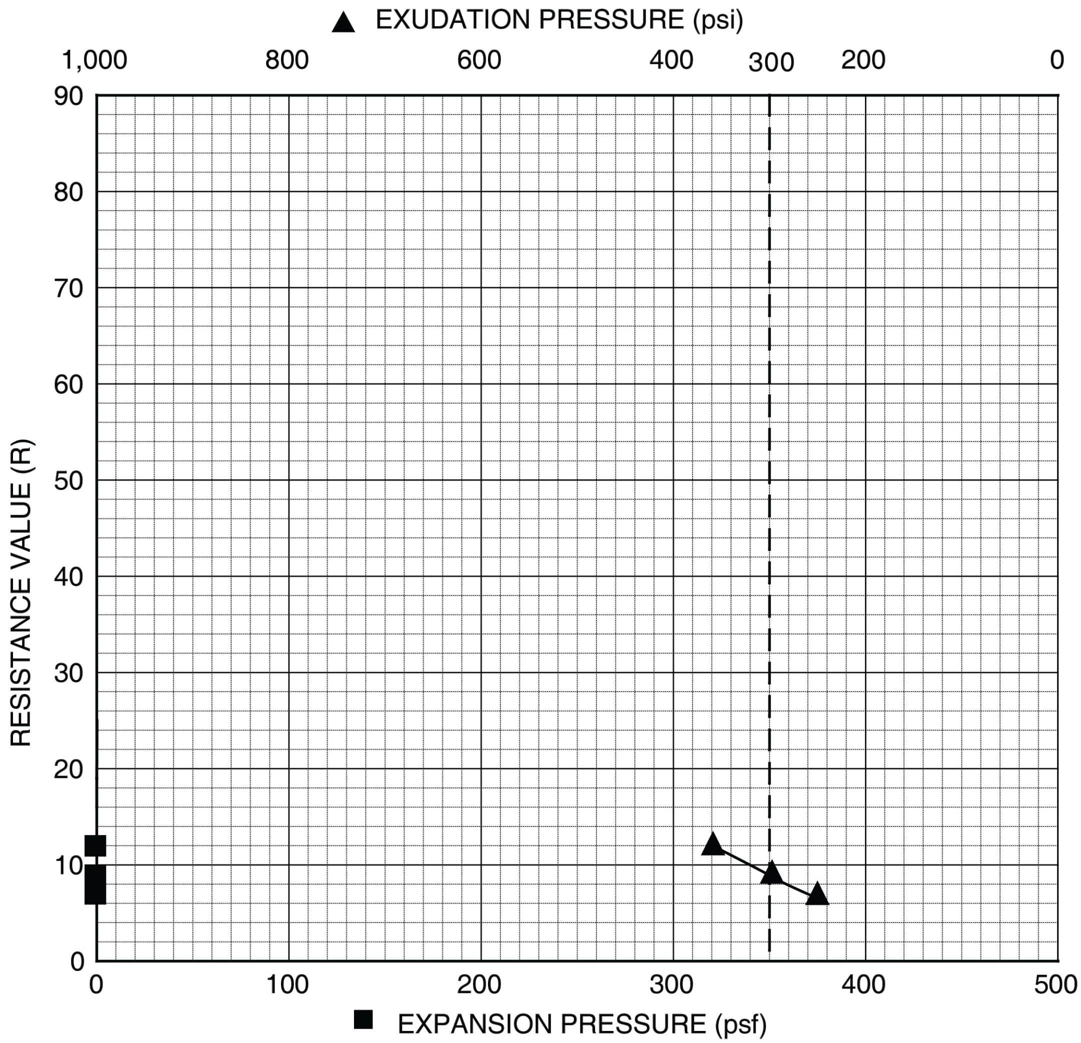
LANGAN
Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113
T: 408.283.3600 F: 408.283.3601 www.langan.com

Project
THE RISE
CUPERTINO
SANTA CLARA COUNTY CALIFORNIA

Figure Title
RESISTANCE VALUE TEST REPORT

Project No. 770633101
Date 11/07/2023
Drawn By AG
Checked By JF

Figure
D-14



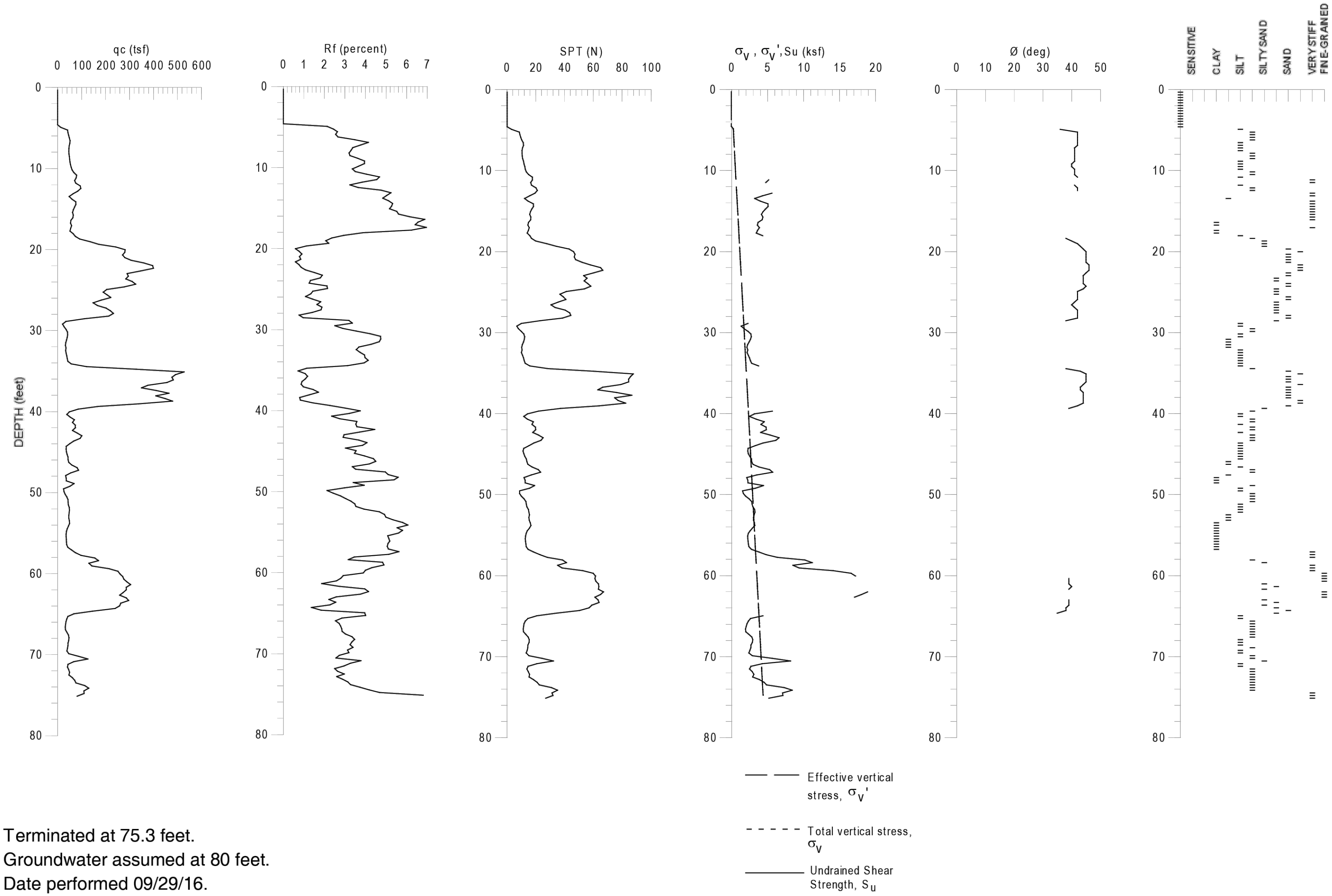
Specimen ID:	A	B	C	D
Water Content (%)	17.8	16.9	16.0	--
Dry Density (pcf)	108.4	113.1	113.9	--
Exudation Pressure (psi)	251	295	361	--
Expansion Pressure (psf)	0.00	0.00	0.00	--
Resistance Value (R)	7	9	12	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-4 at 0 to 5 feet	CLAY with SAND and GRAVEL (CL), brown	--	--	9

LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	RESISTANCE VALUE TEST REPORT	770633101 Date 11/07/2023 Drawn By AG Checked By JF	D-15

APPENDIX E

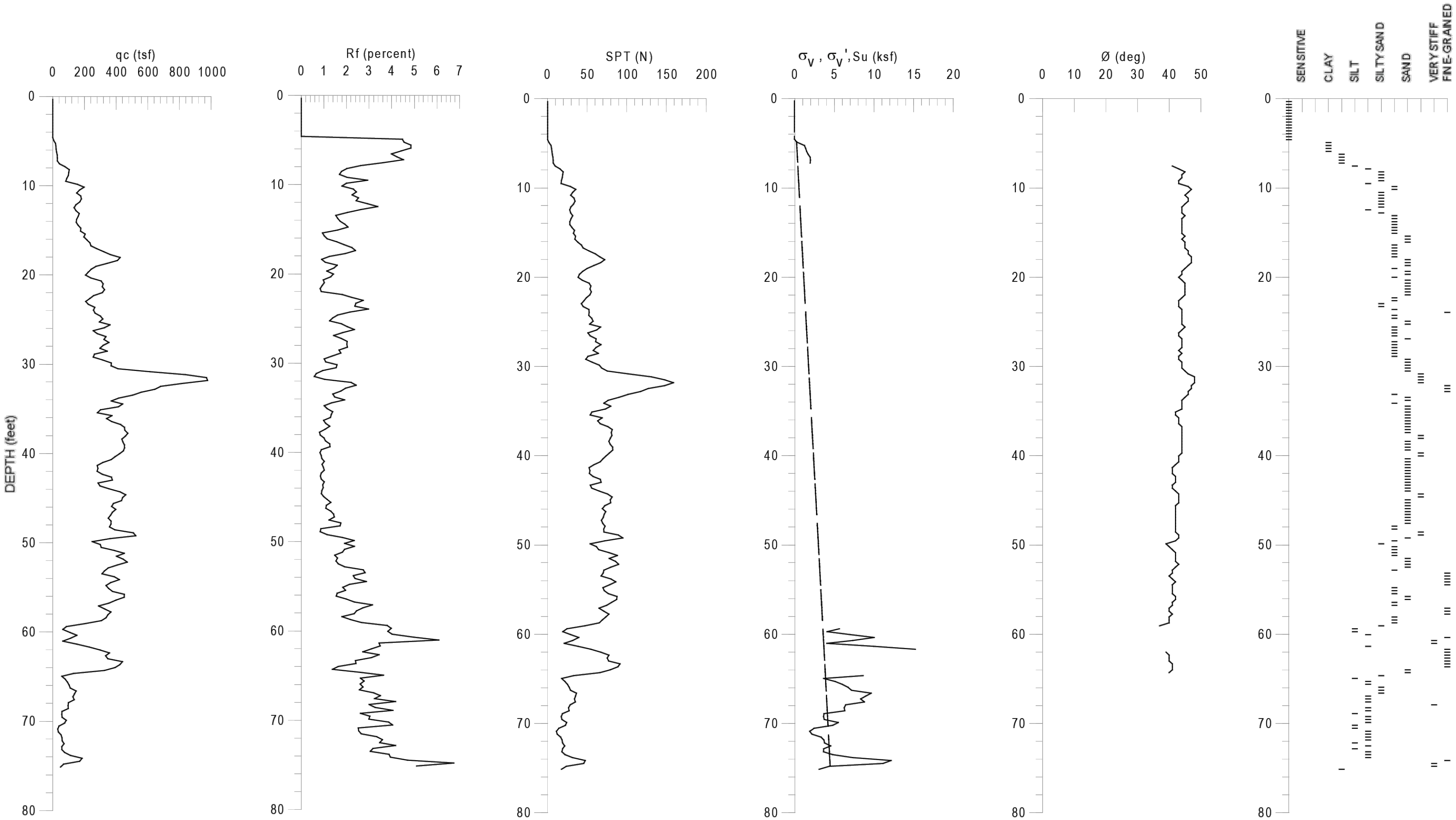
CONE PENETRATION TESTS



Terminated at 75.3 feet.
Groundwater assumed at 80 feet.
Date performed 09/29/16.
Ground surface elevation: 195.4 feet, NAVD 88 Datum.

LANGAN 135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title CONE PENETRATION TEST RESULTS CPT-1	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure E-1

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 11:39 User: agekas Style Table: Langan.stb Layout: CPT-1



Terminated at 75.3 feet.
Groundwater assumed at 80 feet.
Date performed 09/29/16.
Ground surface elevation: 194.2 feet, NAVD 88 Datum.

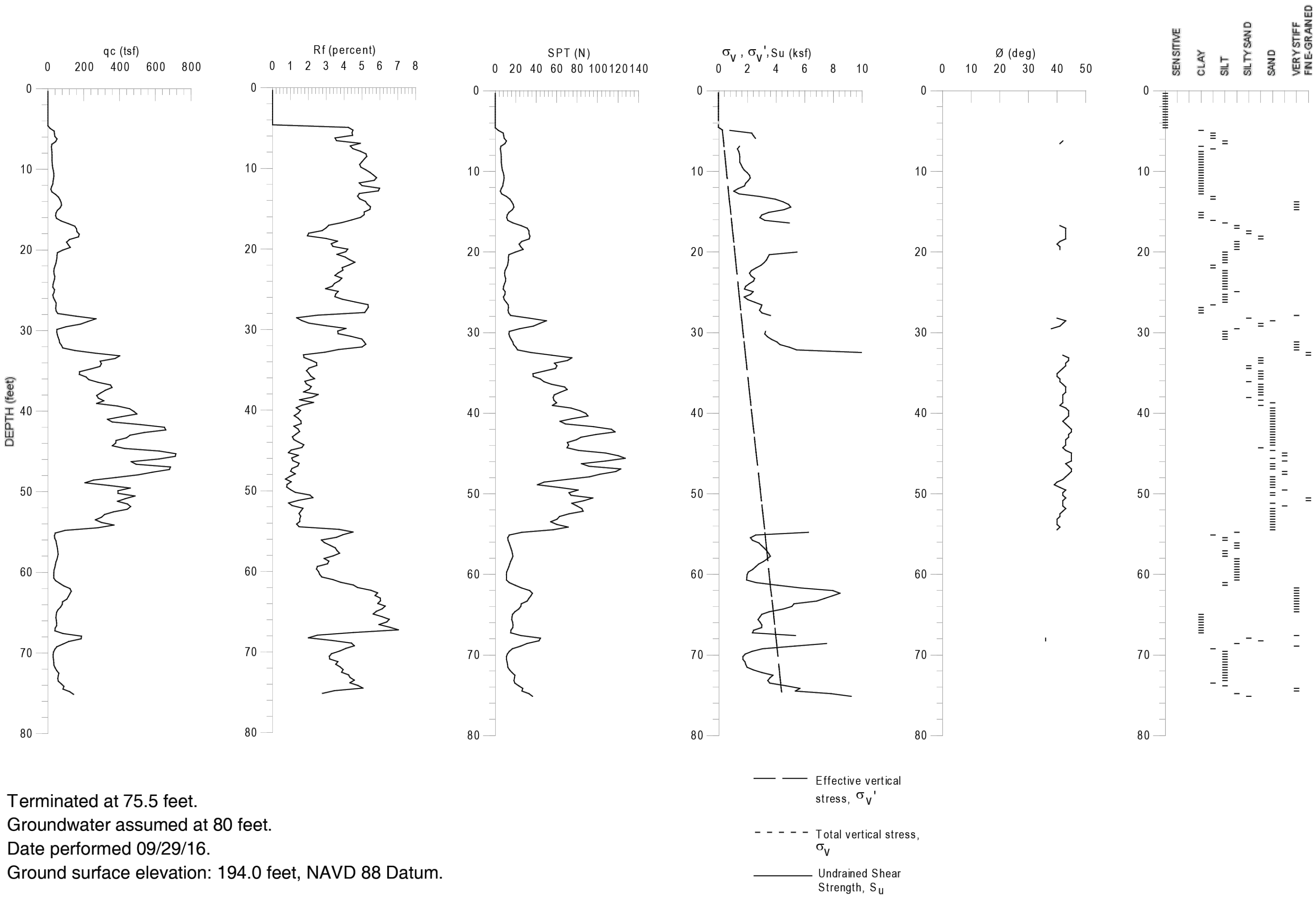
Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, S_u

LANGAN	Project	Figure Title	Project No.	Figure
135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	CONE PENETRATION TEST RESULTS CPT-2	770633101 Date 11/07/2023 Drawn By AG Checked By JF	E-2

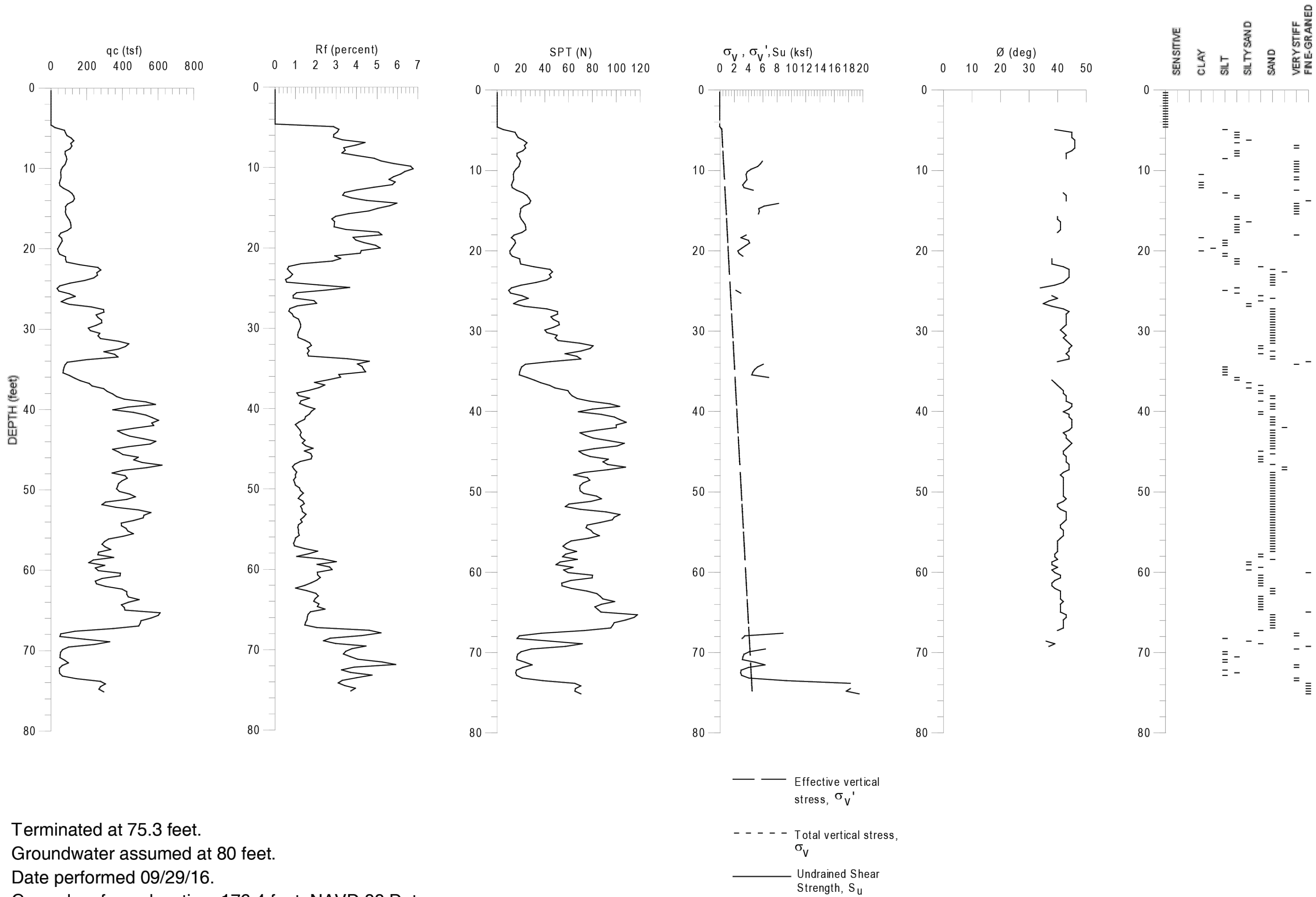
Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 11:43 User: agekas Style Table: Langan.stb Layout: CPT-2



Terminated at 75.5 feet.
Groundwater assumed at 80 feet.
Date performed 09/29/16.
Ground surface elevation: 194.0 feet, NAVD 88 Datum.

LANGAN	Project	Figure Title	Project No.	Figure
135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	CONE PENETRATION TEST RESULTS CPT-3	770633101 Date 11/07/2023 Drawn By AG Checked By JF	E-3

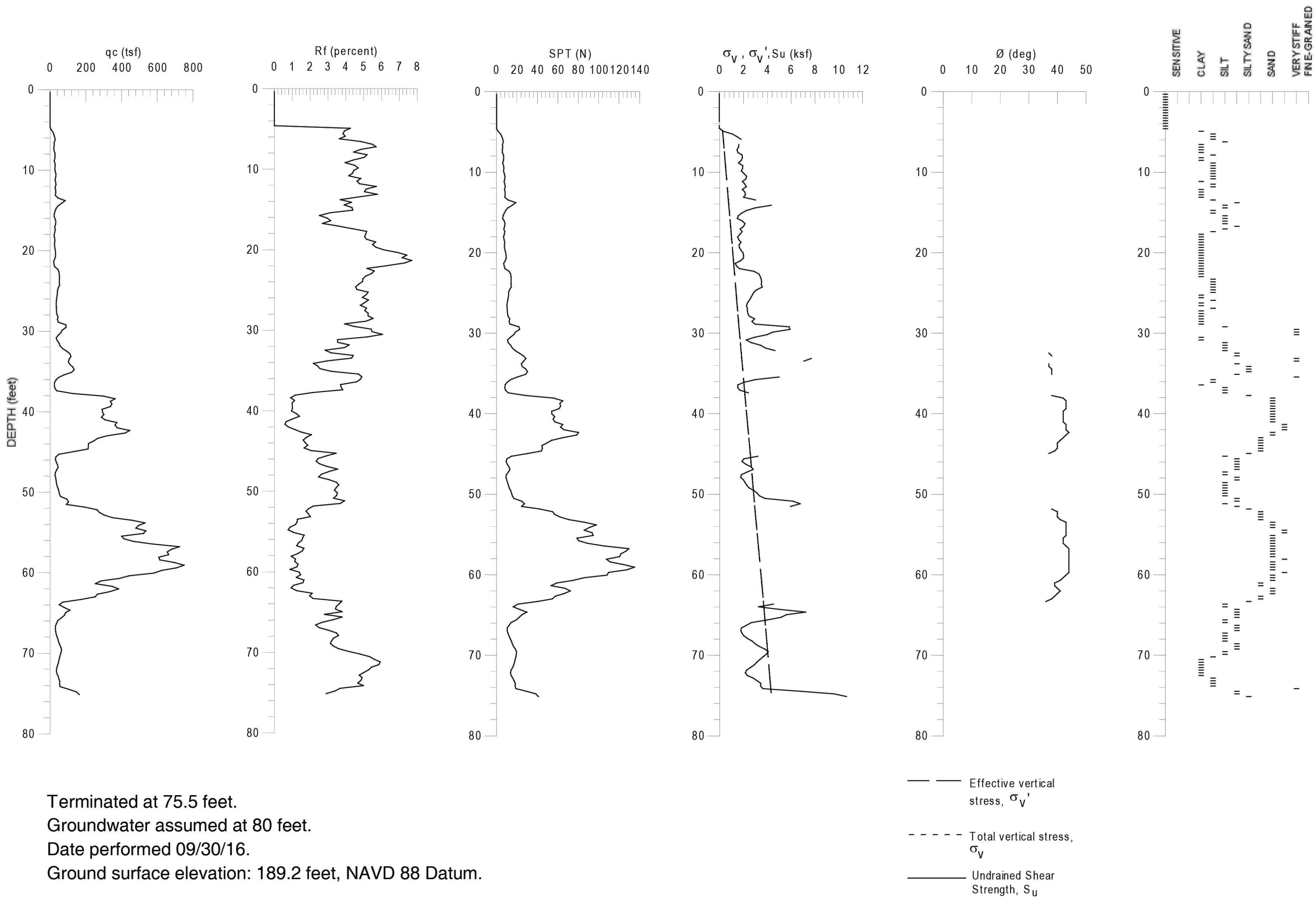
Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 11:48 User: agekas Style Table: Langan.stb Layout: CPT-3



Terminated at 75.3 feet.
Groundwater assumed at 80 feet.
Date performed 09/29/16.
Ground surface elevation: 176.4 feet, NAVD 88 Datum.

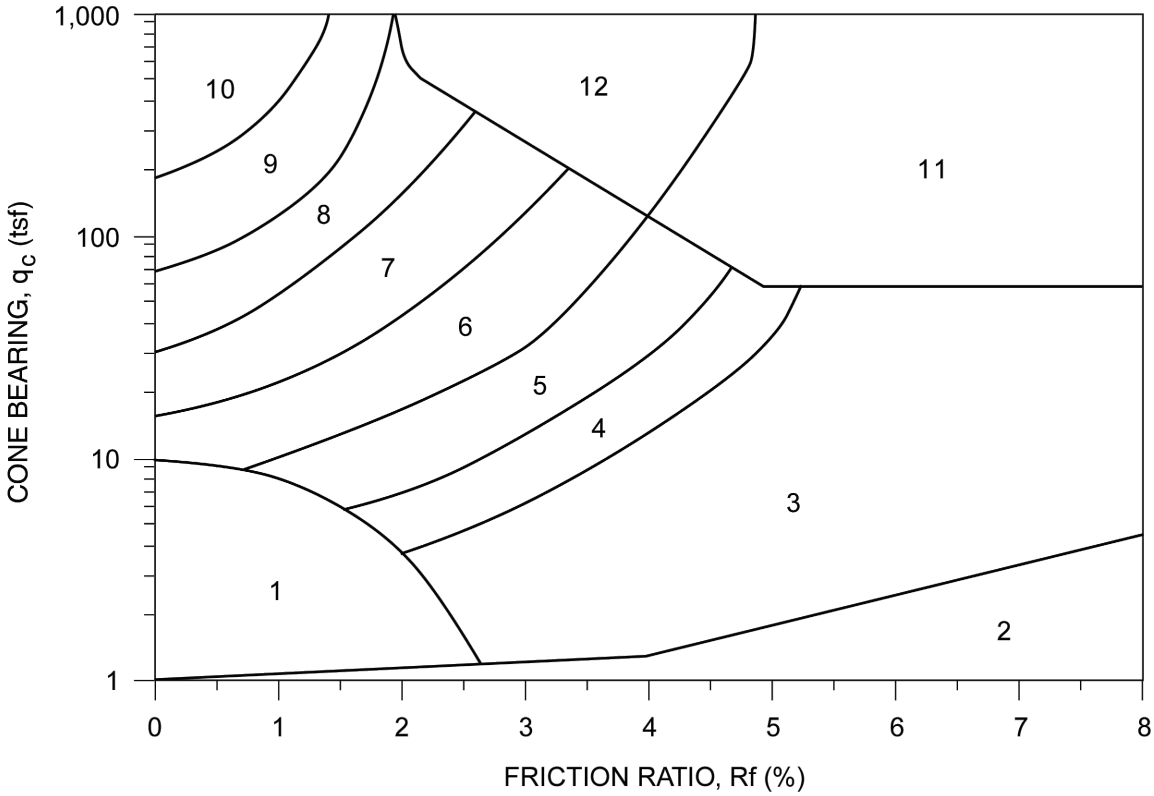
LANGAN 135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title CONE PENETRATION TEST RESULTS CPT-4	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure E-4

Filename: \\langan.com\data\SJO\data1\770633101\Cadd Data - 770633101\2D-DesignFiles\Geotechnical\770633101-B--GI0101_Lab.dwg Date: 11/15/2023 Time: 11:58 User: agekas Style Table: Langan.stb Layout: CPT-4



Terminated at 75.5 feet.
Groundwater assumed at 80 feet.
Date performed 09/30/16.
Ground surface elevation: 189.2 feet, NAVD 88 Datum.

LANGAN 135 Main Street, Suite 1500 San Francisco, CA 94105 T: 415.955.5200 F: 415.955.5201 www.langan.com	Project THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	Figure Title CONE PENETRATION TEST RESULTS CPT-5	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	Figure E-5



ZONE	q_c/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for $q_c \leq 9$ tsf)	Sensitive Fine-Grained
2	1	15 (10 for $q_c \leq 9$ tsf)	Organic Material
3	1	15 (10 for $q_c \leq 9$ tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
q_c = Tip Bearing
f_s = Sleeve Friction
$Rf = f_s/q_c \times 100$ = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud $q_c \leq 9$).
Estimated from local experience (fine-grained soils $q_c > 9$).

LANGAN	Project	Figure Title	Project No.	Figure
Langan Engineering and Environmental Services, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	THE RISE CUPERTINO SANTA CLARA COUNTY CALIFORNIA	CLASSIFICATION CHART FOR CONE PENETRATION TESTS	Project No. 770633101 Date 11/07/2023 Drawn By AG Checked By JF	E-6

APPENDIX F

SOIL CORROSIVITY EVALUATION AND RECOMMENDATIONS FOR CORROSION CONTROL

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

2 May, 2018

Revised
Job No. 1609167
Cust. No. 12242

Mr. Wilson Wong
Langan Treadwell Rollo
4030 Moorpark Avenue, Suite 210
San Jose, CA 95117

Subject: Project No.: 770633101.700.340
Project Name: **Vallco Town Center**
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Wong:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on September 21, 2016. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurements, samples 001 & 003 are classified as "corrosive" and sample 002 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from none detected to 32 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations range from none detected to 210 mg/kg and are determined to be sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The pH of the soils range from 7.56 to 7.95, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 350-mV which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please *call JDH Corrosion Consultants, Inc. at (925) 927-6630.*

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Langan Treadwell Rollo
Client's Project No.: 770633101.700.340
Client's Project Name: **Vallco Town Center**
Date Sampled: 14-Sep-16
Date Received: 21-Sep-16
Matrix: Soil
Authorization: Signed Chain of Custody

Revised
Date of Report: 2-May-2018

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1609167-001	B-3 @ 18.5'	350	7.56	-	1,200	-	32	210
1609167-002	B-4 @ 63.5'	350	7.77	-	3,900	-	N.D.	N.D.
1609167-003	B-5 @ 26'	350	7.95	-	1,700	-	21	21

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Sep-2016	27-Sep-2016	-	27-Sep-2016	-	27-Sep-2016	27-Sep-2016

Cheryl McMillen for

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX G

SITE-SPECIFIC GROUND MOTIONS FOR PREVIOUS DEVELOPMENT SCHEME

APPENDIX G
SITE-SPECIFIC RESPONSE SPECTRA FOR PREVIOUS DEVELOPMENT SCHEME

This appendix presents the details of our estimation of the level of ground shaking at the site during future earthquakes. To develop site-specific response spectra in accordance with 2016 California Building Code (CBC) criteria, and by reference ASCE 7-10, we performed probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis to develop smooth, site-specific horizontal spectra for two levels of shaking, namely:

- Risk Targeted Maximum Considered Earthquake (MCE_R), which corresponds to the lesser of two percent probability of exceedance in 50 years (2,475-year return period) or 84th percentile of the controlling deterministic event both considering the maximum direction as described in ASCE 7-10.

- Design Earthquake (DE) which corresponds to 2/3 of the MCE_R.

G1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are that:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data;

- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake;

- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

As part of the development of the site-specific spectra, we performed a PSHA to develop a site-specific response spectrum for 2 percent probability of exceedance in 50 years. The spectrum for this hazard level was developed using the computer code EZFRISK 8.06 (Risk Engineering

2019). The approach used in EZFRISK is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using Next Generation Attenuation West 2 (NGA – West2) relationships that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault.

G1.1 Probabilistic Model

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i v_i \iint P[Z > z \mid m,r] f_{M_i}(m) f_{R_i|M_i}(r;m)\,dr\,dm$$

where:

v_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i

P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z

f_{Mi} (m) and $f_{Ri|Mi}$ (r;m) = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

G1.2 Source Modeling and Characterization

The segmentation of faults, mean characteristic magnitudes, and recurrence rates were modeled using the data presented in the WGCEP (2008) and Cao et al. (2003) reports. We also included the combination of fault segments and their associated magnitudes and recurrence rates as described in the WGCEP (2008) in our seismic hazard model. Table G-1 presents the distance and direction from the site to the fault, mean characteristic magnitude, mean slip rate, and fault length for individual fault segments. We used the California fault database identified as "USGS 2014 Lower 48 v0.1" in EZFRISK 8.06. Each segment is characterized with multiple magnitudes, occurrence or slip rates and weights. This approach takes into account the epistemic uncertainty associated with the various seismic sources in our model.

TABLE G-1
Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Approx. Fault Length (km)
Monte Vista-Shannon	4.8	Southwest	6.50	0.4	45
N. San Andreas; SAN+SAP	10.6	Southwest	7.73	22	274
N. San Andreas; SAN+SAP+SAS	10.6	Southwest	7.87	21	336
N. San Andreas; SAO+SAN+SAP	10.6	Southwest	7.95	22	410
N. San Andreas; SAO+SAN+SAP+SAS	10.6	Southwest	8.05	22	472
N. San Andreas; SAP	10.6	Southwest	7.23	17	85
N. San Andreas; SAP+SAS	10.6	Southwest	7.48	17	147
N. San Andreas; SAS	17	South	7.12	17	62
Hayward-Rodgers Creek; HN+HS	20	Northeast	7.00	9	87
Hayward-Rodgers Creek; HS	20	Northeast	6.78	9	52
Hayward-Rodgers Creek; RC+HN+HS	20	Northeast	7.33	9	150
Calaveras; CC	22	Northeast	6.39	15	59
Calaveras; CC+CS	22	Northeast	6.50	15	78
Calaveras; CN	22	Northeast	6.87	6	45
Calaveras; CN+CC	22	Northeast	7.00	11	104
Calaveras; CN+CC+CS	22	Northeast	7.03	12	123
Zayante-Vergeles	27	South	7.00	0.1	58
San Gregorio Connected	33	West	7.50	5.5	176
Greenville Connected	46	East	7.00	2	50
Monterey Bay-Tularcitos	46	South	7.30	0.5	83
Mount Diablo Thrust	48	Northeast	6.70	2	25
Hayward-Rodgers Creek; HN	58	North	6.60	9	35
Hayward-Rodgers Creek; RC+HN	58	North	7.19	9	97
Calaveras; CS	61	Southeast	5.83	15	19
Great Valley 7	63	Northeast	6.90	1.5	45

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Approx. Fault Length (km)
Green Valley Connected	64	North	6.80	4.7	56
Ortigalita	65	East	7.10	1	70
N. San Andreas; SAN	71	Northwest	7.51	24	189
N. San Andreas; SAO+SAN	71	Northwest	8.00	24	326
Quien Sabe	73	Southeast	6.60	1	23
SAF - creeping segment	75	Southeast	6.70	34	125
Rinconada	76	Southeast	7.50	1	191
Great Valley 8	77	East	6.80	1.5	41
Great Valley 5, Pittsburg Kirby Hills	78	North	6.70	1	32
Hayward-Rodgers Creek; RC	92	Northwest	7.07	9	62
Great Valley 9	94	East	6.80	1.5	39
West Napa	95	North	6.70	1	30
Point Reyes	100	Northwest	6.90	0.3	47

G1.3 Attenuation Relationships

Pacific Earthquake Engineering Research Center (PEER) embarked on a project to enhance the Next Generation Attenuation for the Western United States, the NGA-West 2 project. We used the relationships by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). These attenuation relationships include the average shear wave velocity in the upper 100 feet. Furthermore, these relationships were developed using the same database and each relationship is considered equally credible. Therefore, the average of the relationships was used to develop the recommended spectra.

The NGA-West 2 relationships were developed for the orientation-independent geometric mean of the data. Geometric mean is defined as the square root of the product of the two recorded components.

As part of our field exploration, we performed down hole suspension logging to estimate the shear wave velocity of the soil beneath the proposed basement. On the basis of the shear wave velocity measurements, we estimate an average shear wave velocity of the upper 30 meters (100 ft), V_{S30}, of approximately 1,670 feet per second (510 meters per second) as such, the site is classified as a very dense profile, site class C. The NGA-West 2 flat files indicate $Z_{1.0}$ and $Z_{2.5}$ are 530 meters and 2.6 kilometers, respectively.

G2.0 PSHA RESULTS

Figures G-1 presents results of the PSHA for 2 percent probability of exceedance in 50 years, 2,475 return period, using the four relationships discussed above. The average of these relationships is also presented.

ASCE 7-10 specifies the development of MCE_R site-specific response spectra in the maximum direction. Shahi and Baker (2014) provide scaling factors that modify the geometric mean spectra to provide spectral values for the maximum response (maximum direction). We used the scaling factors presented in Table 1 of Shahi and Baker (2014) ratios $Sa_{RotD100}/Sa_{RotD50}$ to modify the average of the PSHA results. The maximum direction spectrum is also shown on Figure G-1.

Figure G-2 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, it can be seen that the Monte Vista Shannon and San Andreas faults dominate the hazard at the project site at different periods of interest.

G3.0 DETERMINISTIC ANALYSIS

We performed a deterministic analysis to develop the MCE_R spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. On the basis of the deaggregation results we developed deterministic spectra for both scenarios earthquakes:

- a moment magnitude 6.5 earthquake on the Monte Vista Shannon fault occurring 4.8 km from the site;
- a moment magnitude 8.0 earthquake on the San Andreas fault occurring 10.6 km from the site.

The deterministic MCE spectrum was defined as an envelope of both scenario earthquakes. This is consistent with the deaggregation results discussed in Section G2.0.

The same attenuation relationships as discussed in Section G1.3 were used in our deterministic analysis. Figures G-3 and G-4 presents the 84th percentile deterministic results for the San Andreas and Monte Vista scenarios, respectively. The average of the four relationships is also presented on those figures. Similarly, to the PSHA results, we developed the 84th percentile deterministic spectrum in the maximum direction using the Shahi and Baker (2014) ratios.

Figure G-5 presents the average of the 84th percentile deterministic results in the maximum direction for both scenarios as well as the recommended envelop of both scenarios.

G4.0 RECOMMENDED SPECTRA

The MCE_R as defined in ASCE 7-10 is the lesser of the maximum direction PSHA spectrum having a two percent probability of exceedance in 50 years (2,475-year return period) or the maximum direction 84th percentile deterministic spectrum of the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE_R spectrum. Furthermore, the MCE_R spectrum is defined as risk targeted response spectrum which corresponds to a targeted collapse probability of one percent in 50 years. According to USGS website the risk coefficients vary from 0.88 to 0.96. We used these risk coefficients to develop the Risk-Targeted PSHA response spectrum.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-10 to develop the site-specific spectra for MCE_R and DE. Chapter 21 of ASCE 7-10 requires the following checks:

- the deterministic spectrum used to develop the MCE_R shall not fall below the Deterministic Lower Limit spectrum as shown on Figure 21.2-1 of ASCE 7-10 for site class C;
- the DE spectrum shall not fall below 80 percent of general design spectrum (Section 21.3 of Chapter 21 ASCE 7-10).

Figure G-6 and Table G-2 present a comparison of the site-specific spectra for the PSHA 2,475-year return period (max. dir.), the 84th percentile deterministic (max. dir.), and the Deterministic Lower Limit spectra for Site Class C per ASCE 7-10. We included the risk coefficients as discussed above in the Risk-Targeted PSHA spectrum. The deterministic 84th percentile spectrum is greater than the Deterministic Lower Limit spectrum; hence the MCE_R is defined as the lesser of the 84th percentile deterministic and the PSHA 2,475-year return spectra. The recommended MCE_R spectrum is presented on Figure G-4 and in Table G-2.

TABLE G-2
Comparison of Site-specific and Code Spectra for Development of MCE_R Spectrum per ASCE 7-10
S_a (g) for 5 percent damping

Period (seconds)	Risk Targeted PSHA – 2,475-Year Return Period – Maximum Direction	Deterministic 84th percentile – Maximum Direction	ASCE 7-10 Deterministic Lower Limit Site Class C	Recommended MCE_R
0.01	0.995	0.806	0.600	0.806
0.10	2.053	1.608	1.500	1.608
0.20	2.531	1.997	1.500	1.997
0.30	2.383	1.912	1.500	1.912
0.40	2.131	1.717	1.500	1.717
0.50	1.900	1.568	1.500	1.568
0.60	1.688	1.412	1.300	1.412
0.75	1.450	1.230	1.040	1.230
1.00	1.176	1.012	0.780	1.012
1.50	0.801	0.736	0.520	0.736
2.00	0.601	0.578	0.390	0.578
3.00	0.411	0.427	0.260	0.411
4.00	0.319	0.343	0.195	0.319
5.00	0.258	0.280	0.156	0.258
6.00	0.205	0.223	0.130	0.205
7.00	0.171	0.185	0.111	0.171
8.00	0.143	0.153	0.098	0.143

Table G-3 presents the development of recommended DE spectrum following the procedures outlined in Chapter 21 of ASCE 7-10. The DE is defined as 2/3 of the MCE_R per ASCE 7-10; however, the recommended DE may not be below 80 percent of the general spectrum at any period (ASCE 7-10 Section 21.3). Figure G-6 and Table G-3 presents a comparison of 2/3 of the MCE_R spectrum and 80 percent of the general spectrum for Site Class C. As shown in Table G-3 and Figure G-6, 80 percent of the general spectrum is lower than 2/3 of the MCE_R spectrum. Therefore, we recommend that 2/3 of the MCE_R spectrum be used to develop the DE spectrum. The recommended DE spectrum is shown on Figure G-6.

TABLE G-3
Comparison of Site-specific and Code Spectra for Development of DE Spectrum per ASCE 7-10
S_a (g) for 5 percent damping

Period (seconds)	Recommended MCE_R	2/3 times MCE_R	80% of General Design Spectrum	Recommended DE
0.01	0.806	0.537	0.320	0.537
0.10	1.608	1.072	0.855	1.072
0.20	1.997	1.331	0.855	1.331
0.30	1.912	1.274	0.855	1.274
0.40	1.717	1.145	0.855	1.145
0.50	1.568	1.046	0.855	1.046
0.60	1.412	0.942	0.740	0.942
0.75	1.230	0.820	0.592	0.820
1.00	1.012	0.674	0.444	0.674
1.50	0.736	0.490	0.296	0.490
2.00	0.578	0.385	0.222	0.385
3.00	0.411	0.274	0.148	0.274
4.00	0.319	0.213	0.111	0.213
5.00	0.258	0.172	0.089	0.172
6.00	0.205	0.136	0.074	0.136
7.00	0.171	0.114	0.063	0.114
8.00	0.143	0.095	0.056	0.095

The recommended MCE_R and DE spectra in the maximum direction are presented on Figure G-7 along with a comparison of the general spectrum for site class C and digitized values of the recommended spectra are presented in Table G-4 for a damping ratio of 5 percent.

TABLE G-4
Recommended Spectra S_a (g) for 5 percent Damping

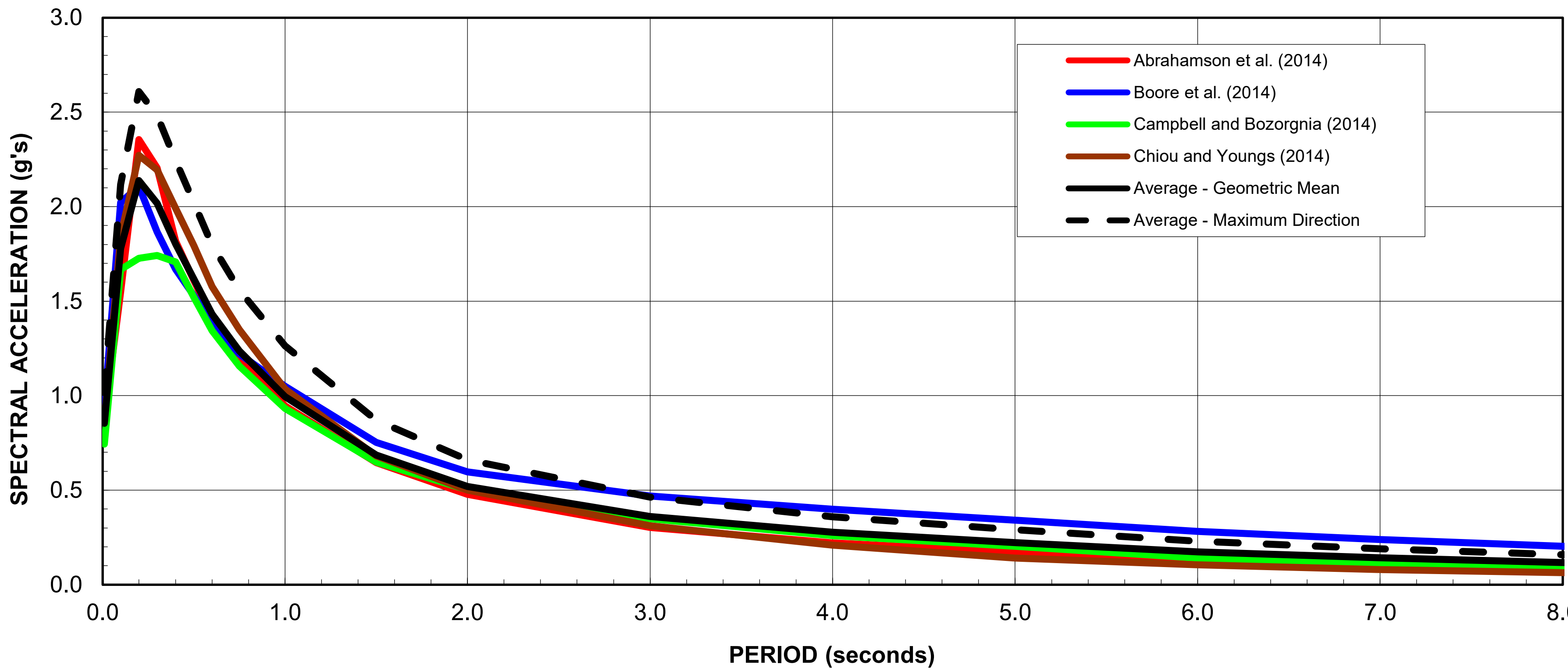
Period (seconds)	Recommended MCE_R	Recommended DE
0.01	0.806	0.537
0.10	1.608	1.072
0.20	1.997	1.331
0.30	1.912	1.274
0.40	1.717	1.145
0.50	1.568	1.046
0.60	1.412	0.942
0.75	1.230	0.820
1.00	1.012	0.674
1.50	0.736	0.490
2.00	0.578	0.385
3.00	0.411	0.274
4.00	0.319	0.213
5.00	0.258	0.172
6.00	0.205	0.136
7.00	0.171	0.114
8.00	0.143	0.095

Because site-specific procedure was used to determine the recommended MCE_R and DE response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-10 should be used as shown in Table G-5.

TABLE G-5
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.997
S_{M1}	1.156*
S_{DS}	1.331
S_{D1}	0.770*

* S_{M1} and S_{D1} are based on the site-specific response spectra and are governed by the spectral acceleration at a period of two seconds.



Damping Ratio = 5%

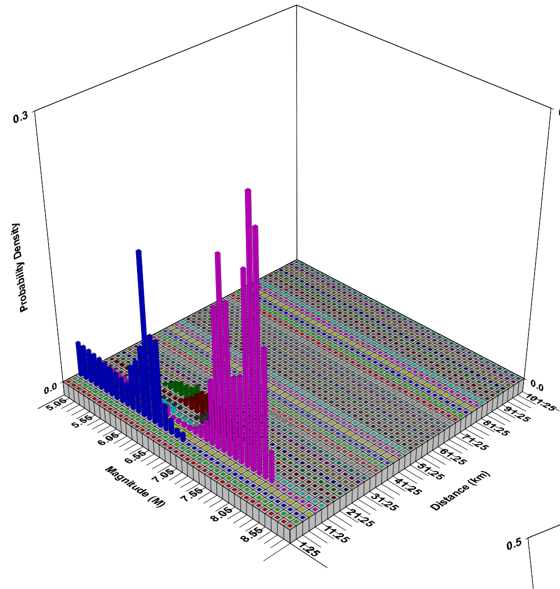
Notes: (1) Estimated V_{S30} = 510 m/s
(2) Maximum direction factors from Shahi and Baker (2014)

THE RISE
Cupertino, California

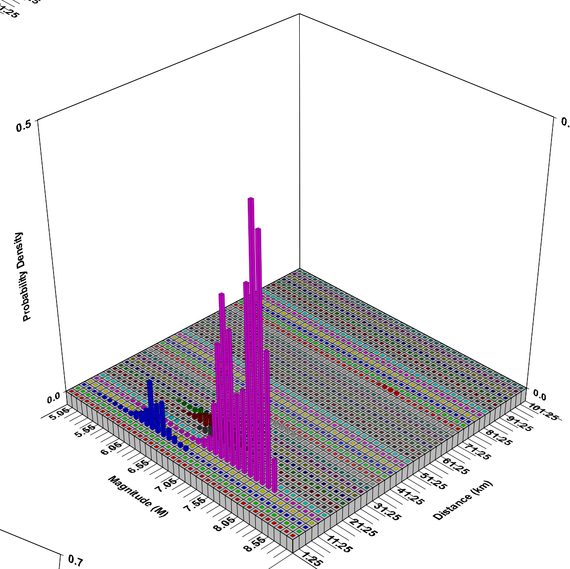
RESULTS OF PSHA, 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

Date 10/06/20	Project No. 770633101	Figure G-1

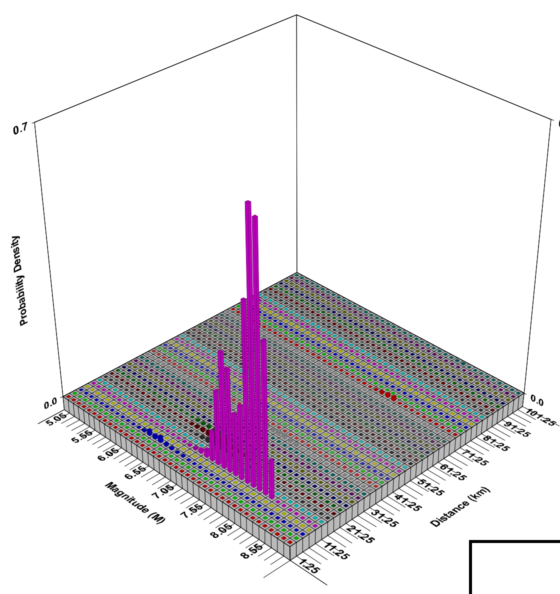
LANGAN



(a) PGA



(b) S_a, T = 1.0 seconds



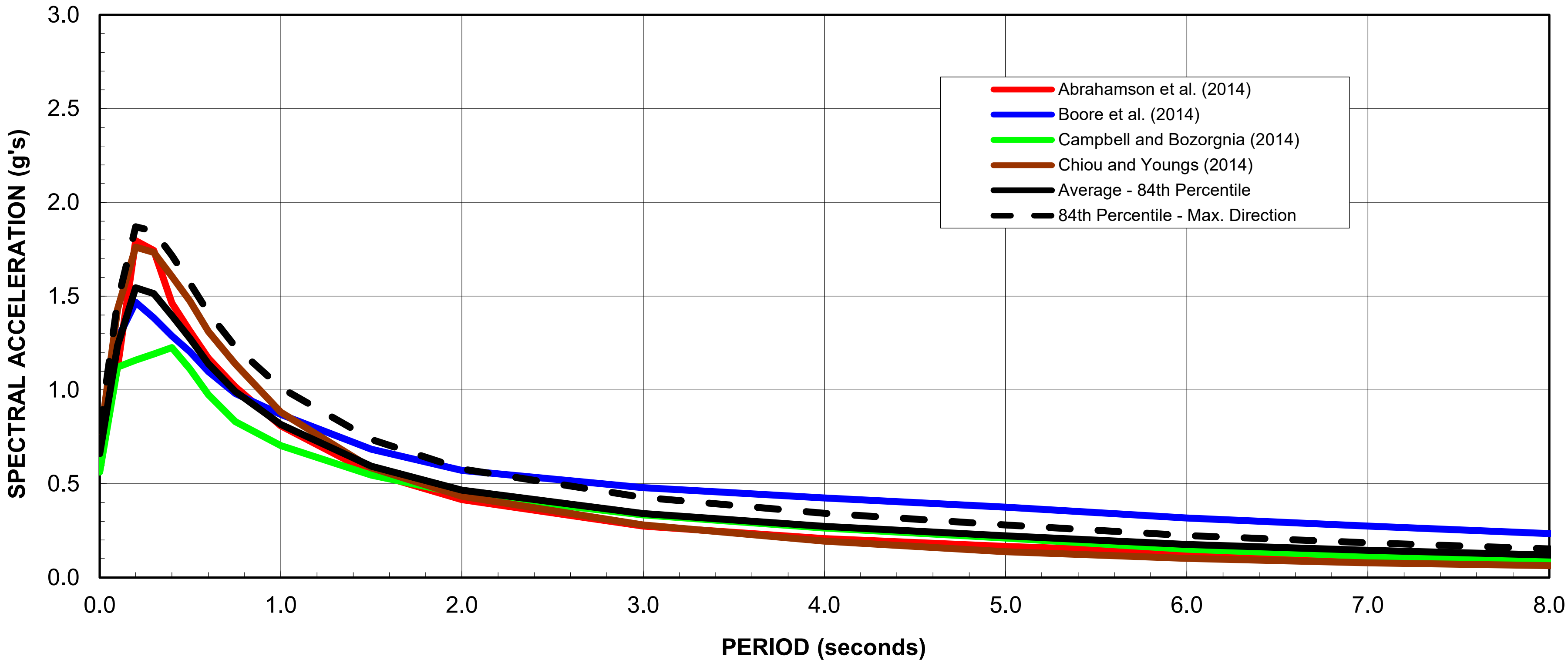
(c) S_a, T = 4.0 seconds

THE RISE
Cupertino, California

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS FOR - MAGNITUDE DISTANCE DEAGGREGATION PLOTS

Date 10/06/20	Project No. 770633101	Figure G-2

LANGAN



Damping Ratio = 5%

Notes: (1) Estimated V_{S30} = 510 m/s

(2) Deterministic results correspond to a Moment Magnitude 8.05 occuring on the San Andreas fault about 10.6 km from the site.

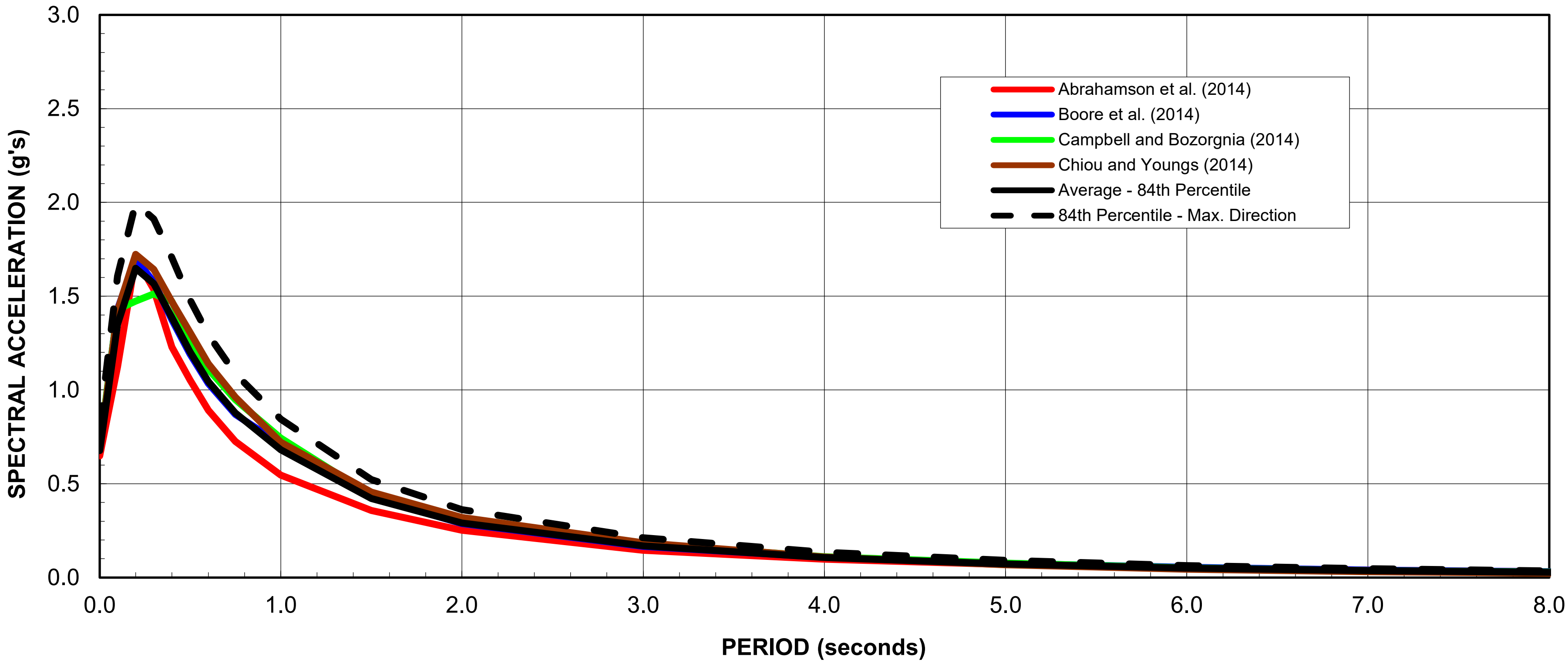
(3) Maximum direction factors from Shahi and Baker (2014)

THE RISE
Cupertino, California

RESULTS OF 84th PERCENTILE DETERMINISTIC ANALYSIS FOR SAN ANDREAS FAULT

Date 10/06/20	Project No. 770633101	Figure G-3

LANGAN



Damping Ratio = 5%

Notes: (1) Estimated V_{S30} = 510 m/s

(2) Deterministic results correspond to a Moment Magnitude 6.5 occuring on the Monte Andreas fault about 4.8 km from the site.

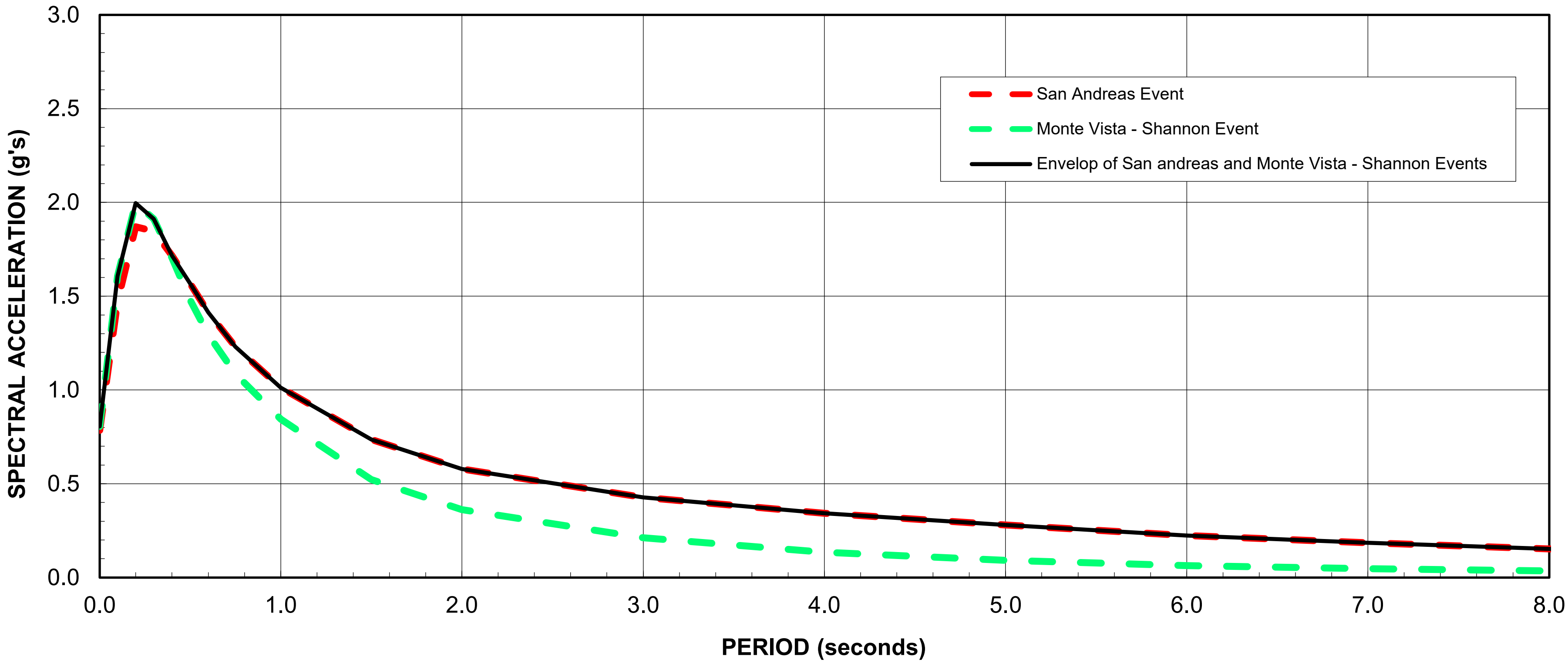
(3) Maximum direction factors from Shahi and Baker (2014)

THE RISE
Cupertino, California

RESULTS OF 84th PERCENTILE DETERMINISTIC ANALYSIS FOR MONTE VISTA SHANNON FAULT

Date 10/06/20	Project No. 770633101	Figure G-4

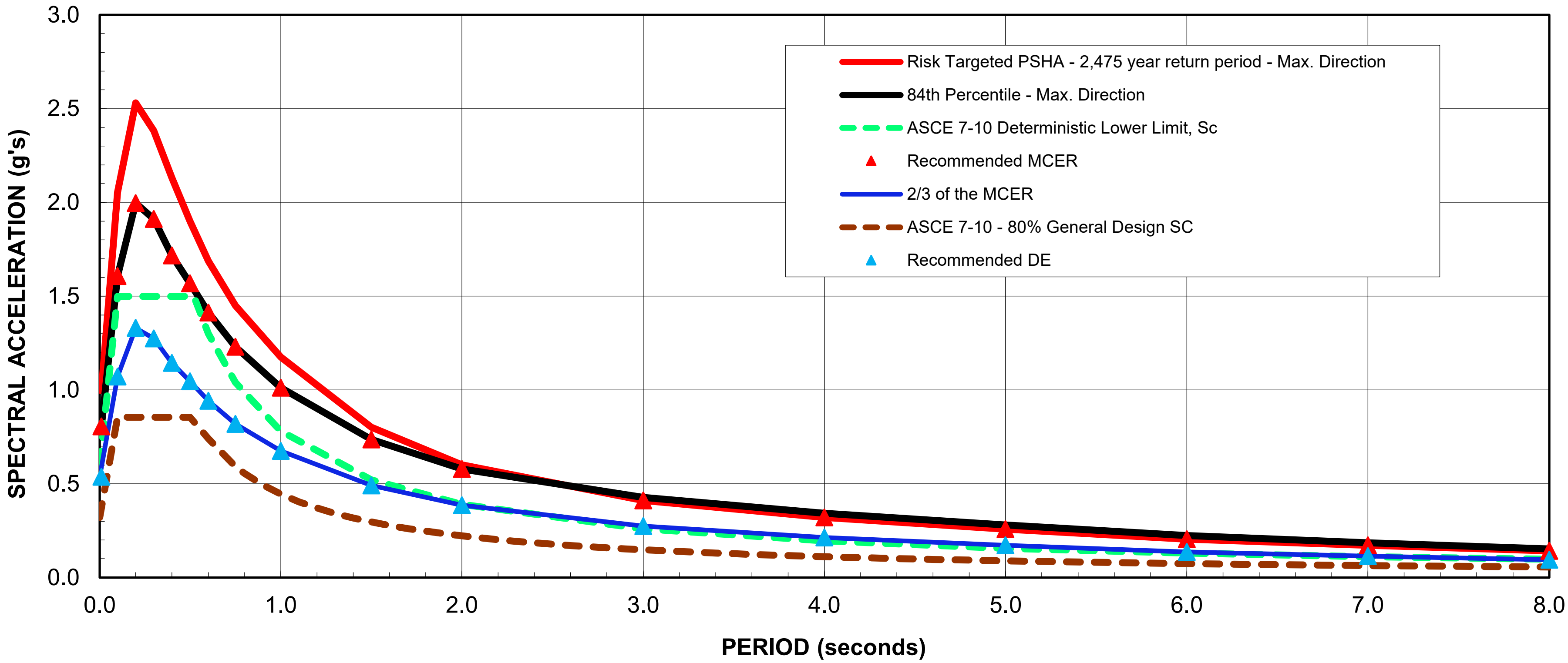
LANGAN



Damping Ratio = 5%

Notes: (1) Estimated V_{S30} = 510 m/s
(2) Deterministic results corresponds to the San Andreas event (M_W = 8.05 and D = 10.6 km) and the Monte Vista-Shannon event ((M_W = 6.5 and D = 4.8 km).
(3) Maximum direction factors from Shahi and Baker (2014)

THE RISE Cupertino, California		
COMPARISON OF 84th PERCENTILE DETERMINISTIC SPECTRA FOR SAN ANDREAS AND MONTE VISTA SHANNON FAULTS		
Date 10/06/20	Project No. 770633101	Figure G-5
LANGAN		

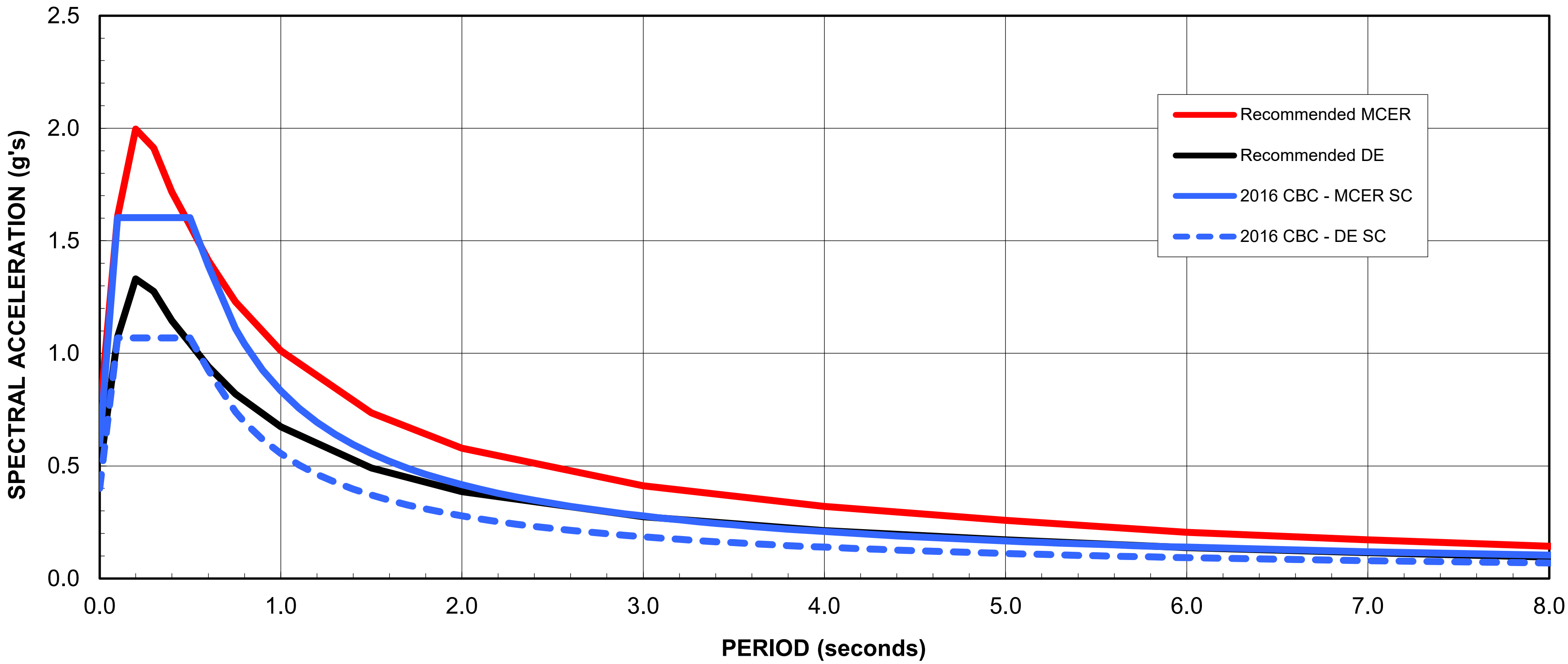


Damping Ratio = 5%

Notes: (1) Estimated V_{S30} = 510 m/s
(2) Deterministic results corresponds to an envelop of the San Andreas event (M_W = 8.05 and D = 10.6 km) and the Monte Vista-Shannon event (M_W = 6.5 and D = 4.8 km).
(3) Maximum direction factors from Shahi and Baker (2014)

THE RISE Cupertino, California		
COMPARISON OF DETERMINISTIC, PROBABILISTIC AND CODE SPECTRA		
Date 10/06/20	Project No. 770633101	Figure G-6

LANGAN



Damping Ratio = 5%

THE RISE
Cupertino, California

COMPARISON OF RECOMMENDED MCE_R AND DE SPECTRA WITH CODE

Date 10/06/20	Project No. 770633101	Figure G-7

LANGAN

DISTRIBUTION

2 copies: Mr. Nandy Kumar
Vallco Property Owner, LLC
10123 North Wolfe Road
Cupertino, California 95014

QUALITY CONTROL REVIEWER

Wilson Wong, GE #3103
Senior Project Engineer