PRELIMINARY GEOTECHNICAL INVESTIGATION
The Oaks
21255 Stevens Creek Boulevard
Cupertino, California

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TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 SCOPE OF SERVICES 1

3.0 FIELD EXPLORATION AND LABORATORY TESTING 2

- **3.1 Borings** 2
- **3.2 Laboratory Testing** 3
- **3.3 Soil Corrosivity Testing** 3

4.0 SUBSURFACE CONDITIONS 4

5.0 SEISMIC CONSIDERATIONS 4

- **5.1 Regional Seismicity** 4
- **5.2 Seismic Hazards** 6
  - **5.2.1 Liquefaction** 7
  - **5.2.2 Seismic Densification** 7
  - **5.2.3 Lateral Spreading** 7
  - **5.2.4 Tsunami** 8
  - **5.2.5 Surface Faulting** 8

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS 8

- **6.1 Foundations** 8
- **6.2 Basement Wall Design** 9
- **6.3 Corrosion Potential** 11
- **6.4 Site Preparation** 11
  - **6.4.1 Slab-On-Grade Floors** 12
  - **6.4.2 At-Grade Improvements** 13
- **6.5 Temporary Shoring** 14
- **6.6 Excavation and Monitoring** 16
- **6.7 2013 CBC Mapped Values** 17
- **6.8 Additional Subsurface Investigation** 17

7.0 LIMITATIONS 18

REFERENCES

FIGURES

APPENDIX A – Logs of Borings

APPENDIX B – Laboratory Data

APPENDIX C – Corrosivity Analysis with Brief Evaluation

DISTRIBUTION

LIST OF FIGURES

Figure 1 Site Location Map

Figure 2 Site Plan

Figure 3 Map of Major Faults and Earthquake Epicenters in the San Francisco Bay Area

Figure 4 Modified Mercalli Intensity Scale

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1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation performed by Langan Treadwell Rollo (Langan) for the proposed development at The Oaks which is located at 21255 Stevens Creek Boulevard in Cupertino, California. The site is northwest of the intersection of Stevens Creek Boulevard and Mary Avenue, across from DeAnza College. It is bound on the north and east side by Mary Avenue, Stevens Creek Boulevard to the south and an on-ramp onto Highway 85 to the west, as shown on Figure 1. It is currently The Oaks shopping center and is occupied by several one-story buildings and surrounding paved parking lots and landscaping. The elevations vary from 290 feet at the east end of the property to 300 feet at the west end of the property (Kier and Wright, 2003).

We understand that the proposed development is not yet planned, but that a mid- to high-rise building with one to three basement levels is being considered. Detailed structural loading information is not available at this point.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposal dated 4 September 2014. The purpose of our investigation was to obtain subsurface data, evaluate the potential geologic hazards at the site and provide preliminary conclusions and recommendations for the geotechnical aspects of the project. We used the results from our field exploration of the site to perform our engineering analysis and developed preliminary conclusions and recommendations for the following:

- soil and groundwater conditions
- site seismicity and seismic hazards, if any
- probable foundation type(s) for the proposed building

- preliminary design parameters for the recommended foundation type(s),
- subgrade preparation for slab-on-grade floors
- probable temporary shoring type(s) for the basement option
- site grading and excavation, including criteria for fill quality and compaction
- 2013 California Building Code (CBC) soil profile type and mapped values S_S and S_1 and coefficients F_a and F_v

3.0 FIELD EXPLORATION AND LABORATORY TESTING

As part of our field exploration, we drilled three borings in the parking lot around the Oaks shopping center. The approximate locations of the borings are presented on the Site Plan, Figure 2. Prior to performing the field exploration, Underground Service Alert (USA) was contacted and a private utility locator was retained to check the boring locations for existing utilities. Details of each aspect of the field exploration and laboratory testing are discussed in the remainder of this section.

3.1 Borings

Borings, designated B-1, B-2, and B-3 were drilled on 2 to 3 October 2014 using a truck-mounted, drill rig operated by Gregg Drilling (Gregg). The borings were drilled with a hollow stem auger to about 45 feet below ground surface (bgs). Our engineer logged the borings and obtained samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-3. The soil encountered in the boring was classified in accordance with the Classification Chart presented on Figure A-4.

Soil samples were obtained using two different types of samplers. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel 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) split-barrel 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 being sampled 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 soil and the SPT sampler was used to evaluate the penetration resistance of sandy soil.

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The 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. 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 1.2, respectively, to account for sampler type and hammer energy, and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts.

Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD.

The soil cuttings from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and eventually transported off-site for proper disposal.

3.2 Laboratory Testing

Soil samples recovered from our field exploration program were re-examined in the office and representative samples were selected for laboratory testing. Soil samples were tested to measure moisture content, unit weight, shear strength, compressibility and corrosivity. Results of the laboratory tests are included on the boring logs and in Appendix B.

3.3 Soil Corrosivity Testing

To evaluate the corrosivity of the soil near the foundation subgrade, we performed corrosivity tests on samples obtained from the upper 6.5 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 are presented in Appendix C.

4.0 SUBSURFACE CONDITIONS

The site is in Cupertino, which is underlain by alluvial sediment deposited from the Santa Cruz Mountains. These alluvial fan deposits are typically coarse grained with large amounts of gravel deposits.

The surface material encountered in the borings consists of 3.5 to 6 inches of asphalt concrete (AC) and aggregate base (AB). Beneath the pavement section, the upper 2.5 to 6.5 feet consists of very dense sand with clay and gravel and hard sandy clay with varying amounts of gravel. Laboratory test results indicate the near surface clay layer has low expansion potential[1] with plasticity index of 9.

Below these depths are medium dense to very dense sand and gravel layers with varying amounts of silt and clay interbedded with 3.5 to 7 feet thick layers of very stiff to hard sandy clay, sandy clay with gravel, and clay with gravel to the maximum explored depth of 46.5 feet.

During the investigation, groundwater was not encountered while drilling the three borings. The California Geological Survey, as part of their Seismic Hazards Zone Report (Cupertino Quadrangle) reported the historic high groundwater level in this area as approximately 50 feet bgs.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The major active faults in the area are the San Andreas, Monte Vista-Shannon, San Gregorio, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 3. For each of the active faults within 50 kilometers (km) of the site, the distance from the site

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

and estimated mean characteristic Moment magnitude[2] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	3	Southwest	6.50
N. San Andreas - Peninsula	8	Southwest	7.23
N. San Andreas (1906 event)	8	Southwest	8.05
N. San Andreas - Santa Cruz	17	South	7.12
Total Hayward	23	Northeast	7.00
Total Hayward-Rodgers Creek	23	Northeast	7.33
Total Calaveras	25	East	7.03
Zayante-Vergeles	27	Southeast	7.00
San Gregorio Connected	30	West	7.50
Monterey Bay-Tularcitos	45	South	7.30
Greenville Connected	49	East	7.00
Mount Diablo Thrust	49	Northeast	6.70

Figure 3 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 4) 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), a 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, approximately 59 km from the site.

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[2] 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.

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 a 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 felt in the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 82 kilometers north of the site, with a M_w of 6.0.

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2

WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6

5.2 Seismic Hazards

During a major earthquake, strong to violent ground shaking is expected to occur at the project site. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[3] lateral spreading,[4] cyclic densification,[5] landsliding, or can cause a tsunami. Each of these conditions has been evaluated based on our literature review and field exploration and analysis and are discussed in this section.

5.2.1 Liquefaction

The site is outside the zone designated with the potential for liquefaction, as identified by the California Geologic Survey (formerly the California Division of Mines and Geology) in a map titled, "State of California Seismic Hazard Zones, Cupertino Quadrangle, Official Map" prepared by the California Geologic Survey (23 September 2002). Specifically, the map shows the site outside the area "where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693 (c) would be required." Furthermore, because the cohesionless layers are generally dense to very dense and the groundwater is deeper than 50 feet, we judge the liquefaction potential as low.

5.2.2 Seismic Densification

Cyclic densification refers to seismically-induced differential compaction of non-saturated granular material (sand and gravel above the groundwater table) caused by earthquake vibrations. The borings indicate the sand deposits above the groundwater level are sufficiently dense and/or clayey. Therefore, we judge that seismic densification is unlikely.

5.2.3 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes.

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[3] 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.

[4] 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.

[5] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

The project site is near a free face on the west side of the site at the entrance ramp for Highway 85 North; however, because the soil is not potentially liquefiable we conclude lateral spreading is not likely to occur.

5.2.4 Tsunami

Recent published maps (California Emergency Agency, 2009) indicate the site is not within the tsunami inundation zone; therefore, we conclude the potential risk by inundation from tsunami to be low for the site.

5.2.5 Surface Faulting

We evaluated the risk of surface faulting at the site associated with active or potentially active fault traces. Historically, ground surface displacements closely follow the trace of geologically young faults. Based on our study, we conclude 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. Therefore, we judge the risk of surface faulting at the site is very low. However, in a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

We understand the design team is evaluating 1, 2, or 3 basement levels. On the basis of our subsurface investigation we conclude any of these three options are feasible from a geotechnical standpoint.

Our preliminary conclusions and recommendations regarding geotechnical issues are discussed in the remainder of this section.

6.1 Foundations

The soil at the bottom of the proposed basement level options consists primarily of very stiff sandy clay and very dense sand and gravel. Therefore, we conclude that the building can be supported on a shallow foundation system consisting of either isolated spread footings or a mat foundation bearing on native soil. We preliminarily recommend allowable dead plus live load bearing pressures ranging from 4,000 to 5,000 pounds per square foot (psf) with a one third increase for total loads. We estimate total settlements will be less than one inch.

All footings should be embedded at least 18 inches below lowest adjacent grade. Continuous footings should be at least 18 inches wide; isolated spread footings should be at least 24 inches in plan dimension.

Lateral forces can be resisted by a combination of friction along the base of the foundation, and passive resistance against the vertical faces of the foundation. Frictional resistance should be computed using a base friction coefficient of 0.30. Depending on the number of basement levels and if waterproofing is needed, the allowable friction factor should be reduced and will depend on the type of waterproofing used at the base of the foundation; typically a base friction coefficient of 0.15 to 0.20 can be assumed for waterproofing. However, friction factors will depend on type of waterproofing membrane selected and should be provided by the manufacturer. To calculate the passive resistance against the vertical faces of the footings, we recommend an equivalent fluid weight of 350 pounds per cubic foot (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 about 1.5.

6.2 Basement Wall Design

We recommend all basement walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. 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 a seismic pressure 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 Design Earthquake ground motion level to compute the seismic pressure increment. Basement walls should be designed for the more critical loading condition of static or seismic conditions using the equivalent fluid weights and pressures presented in Table 3.

TABLE 3

Basement Wall Design Earth Pressures (Drained Conditions)

Condition	**Equivalent Fluid Weights for Static Conditions**		**Seismic Conditions**[1]
	Unrestrained Walls (Active)	**Restrained Walls (At rest)**	**Total Pressure – Active Plus Seismic Pressure Increment**
Above Groundwater[2]	30 pcf	50 pcf	70 pcf

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. Assumes basement will be above ground water table.

Where traffic will pass within 10 feet of basement walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls. If the basement walls are designed to resist lateral forces such as wind or earthquake loading they should be checked using passive pressures. To calculate the passive resistance against the below-grade walls, we preliminarily recommend an equivalent fluid weight of 350 pcf. This value includes a factor of safety of about 1.5 and assumes several inches of movement would be required to mobilize full passive resistance. The walls should be checked by the structural engineer for this condition, if passive resistance of walls is needed.

The lateral earth pressures given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. If the walls are not drained, they should be designed for an equivalent fluid weight of 80 pcf to account for hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend to a the base of the wall and drain into the surrounding soil near the base. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

If backfill is required behind basement walls, the walls should be braced or hand-compaction equipment used to prevent unwanted surcharges on the walls.

6.3 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 4 and Appendix C, which includes a brief evaluation of the corrosivity.

TABLE 4

Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfates (mg/kg)	Resistivity (ohms-cm)	Redox (mV)	Chlorides (mg/kg)
B-1	6	7.00	N.D.	2,100	330	N.D.
B-2	3.5	7.44	69	2,800	310	16

N.D. = None Detected

Based upon resistivity measurements, the soil samples tested are classified as "moderately 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 not 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.

6.4 Site Preparation

Existing pavements, old building foundations, abandoned utilities and other obstructions should be removed from areas to receive improvements. We anticipate the excavation for this project can be made using conventional earth-moving equipment except where old foundations and other obstructions are encountered. These may require hoe rams or jackhammers to remove. Any portions of existing buried foundations that could interfere with the proposed foundations or basement walls should be removed.

Where utilities to be removed extend off site, they should be capped or plugged with grout at the property line. It may be feasible to abandon utilities in-place, outside the proposed building footprint provided they will not interfere with future utilities, or building foundations or walls. If utilities are abandoned in-place, they should be completely filled with flowable cement grout over their entire length within the property limits. Existing utility lines, where encountered, should be addressed on a case-by-case basis.

6.4.1 Slab-On-Grade Floors

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[6]. If the subgrade is disturbed during excavation for footings 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 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 ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 5.

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[6] 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 laboratory compaction procedure.

TABLE 5
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
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be dry at the time concrete is cast. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

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 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not 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.

6.4.2 At-Grade Improvements

Other areas that will receive improvements (e.g. sidewalks and exterior slabs and concrete flatwork) should be stripped of existing improvements. The surface exposed by stripping should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. If soft or loose soil is encountered, the unsuitable material should be removed and be replaced with suitable fill material that is properly compacted and moisture conditioned. The exposed ground surface should be kept moist during subgrade preparation.

New fill should be free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, have low corrosion potential[7] and be approved by the geotechnical engineer. All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The subgrade surface should be rolled to a dense, non-yielding surface. If the compacted subgrade is disturbed during utility trench or foundation excavations, the subgrade should be re-rolled to provide a smooth, firm surface for concrete slab support.

We recommend new sidewalks and concrete flatwork (in non-vehicular traffic area) be underlain by at least four inches of Class 2 aggregate base material that has been compacted to at least 90 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 the geotechnical engineer for approval at least three business days prior to use at the site.

6.5 Temporary Shoring

Construction of 1- to 3- basement levels will require an excavation ranging from about 15 to 35 feet or more below the adjacent street grades. During excavation for the proposed basement levels, shoring will be required to laterally restrain the sides of the excavation and limit the movement of adjacent improvements, such as public streets and sidewalks.

We considered a sheet pile shoring system or vibrated soldier piles to shore the excavation. However, because the soil underlying the site consists of dense to very dense sand and gravel layers, we conclude that it would be very difficult to penetrate these layers. We also considered a shotcrete wall with soil nails, but we conclude that the excavation would not stand vertically for sufficient time until a shotcrete wall could be installed, because of the presence of cohesionless sand and gravel. Therefore, we preliminary conclude, the excavation

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[7] 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.

should be retained using a soldier-pile-and-lagging shoring system with tiebacks or internal bracing. Internal braces may be required if there are obstructions precluding the use of tiebacks or if extending them beyond property lines is not permitted; encroachment permits will be required to install tiebacks beneath city streets and sidewalks, Caltrans right of way and adjacent properties. A soldier-pile-and-lagging system consists of concrete encased steel H-beams placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds.

Tied-back soldier piles and lagging shoring should be designed to resist the lateral earth pressures presented on Figure 5. In developing the pressures shown on Figure 5, we assumed that groundwater will be below the bottom of the excavation and not impose loads on the shoring system. Traffic or surcharge loads should be added to the active pressures. If traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf should be applied to the upper 10 feet of the walls. If cranes are planned outside of the proposed excavations, their foundations should be designed so as not to surcharge the shoring, or the shoring should be designed for the imposed surcharge.

Tiebacks will be installed in hard clay and dense to very dense sand and gravel. Allowable capacities of the tiebacks will depend on the installation method, hole diameter, grout pressure, and workmanship. For estimating purposes, we recommend using the skin friction values for pressure grouted tiebacks given on Figure 5. These values include a safety factor of 1.5. Tiebacks should be spaced no closer than four hole diameters or four feet, whichever is greater.

As shown on Figure 5, tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point 0.2xH feet away from the bottom of the excavation at an angle 60 degrees from horizontal, where H is the wall height in feet. The stressing and bond lengths should each be at least 15 feet, respectively.

Auger-type drilling equipment (hollow stem or flight) should not be used to install tiebacks because of the potential for caving and the potential for overdrilling. On similar projects, specialty contractors have used a Klemm rig (double cased hole) with success. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Determining the length of tiebacks required to resist the pressures presented on Figure 5 should be the contractor's responsibility. The computed bond length should be confirmed by a proof-testing program under the observation of an engineer experienced in this type of work.

Tiebacks should be proof- and performance-tested to at least 1.25 times the design load. The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested. If any tiebacks fail to meet the testing requirements, additional tiebacks should be added to compensate for the deficiency, as required by the shoring designer.

Passive resistance below the bottom of the excavation may be computed using an equivalent fluid weight of 350 pcf. These values include a factor of safety of 1.5. Passive pressures can be assumed to act on an area of three pile widths provided the piles are spaced at least three diameters apart (center to center spacing).

6.6 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. However, remnants of buried foundations, building slabs or basements may be encountered, which may require the use of jack hammers or hoe-rams to break apart and remove.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor’s skill in installing the shoring system. Movements for a properly designed and constructed shoring system should be the order of 1 to 2 inches. Considering the size and depth of the excavation and the presence of adjacent streets and other improvements, we judge a monitoring program should be established to evaluate the potential movement.

If sand with low fines content is encountered within the zone of excavation, installation of lagging in the sand can be difficult. Lagging boards should be placed with every three feet of excavation. If caving occurs, the lagging should be placed with every foot of excavation. Voids that result from caving soil behind wood lagging should be grouted before proceeding to the next row of lagging. To restrict potential wall movement tiebacks or internal bracing should be installed. The shoring system and adjacent improvements should be monitored for movements throughout the excavation until the street-level slab is cast.

Movements associated with a soldier pile and lagging system may adversely impact adjacent improvements. Therefore, a stiffer shoring system may be required, such as a soil-cement mix wall or concrete diaphragm walls where movements may adversely impact adjacent improvements. These systems are stiffer than a conventional soldier pile and lagging system and consequently, will deflect less. Recommendations for these types of shoring systems can be provided, if it is determined that the deflections from a tied-back soldier-pile-and-lagging system are not tolerable.

6.7 2013 CBC Mapped Values

For seismic design in accordance with the provisions of 2013 California Building Code (CBC) we recommend the following:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.993g and 0.738g, respectively.
- Site Class D
- Site Coefficients F_a and F_v of 1.0 and 1.5
- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.993g and 1.107g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.328g and 0.738g, respectively.

6.8 Additional Subsurface Investigation

This report presents the results of our preliminary geotechnical investigation. The number of borings performed does not provide adequate site coverage to prepare detailed design level recommendations concerning differential settlements due to static and seismic loads, or evaluate potential variations of near surface soil characteristics beneath proposed structures. This investigation was performed to assess the general engineering characteristics of soil conditions present at the site, and to provide insight into the anticipated geotechnical issues that may affect the potential development options and design of the improvements being considered for the site. Once more detailed development plans become available; a more detailed geotechnical investigation including additional field exploration should be performed to develop design level recommendations for use in the design of proposed improvements.

7.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of engineering studies and our interpretations of the existing geotechnical conditions. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan Treadwell Rollo should be notified so that supplemental recommendations can be developed.

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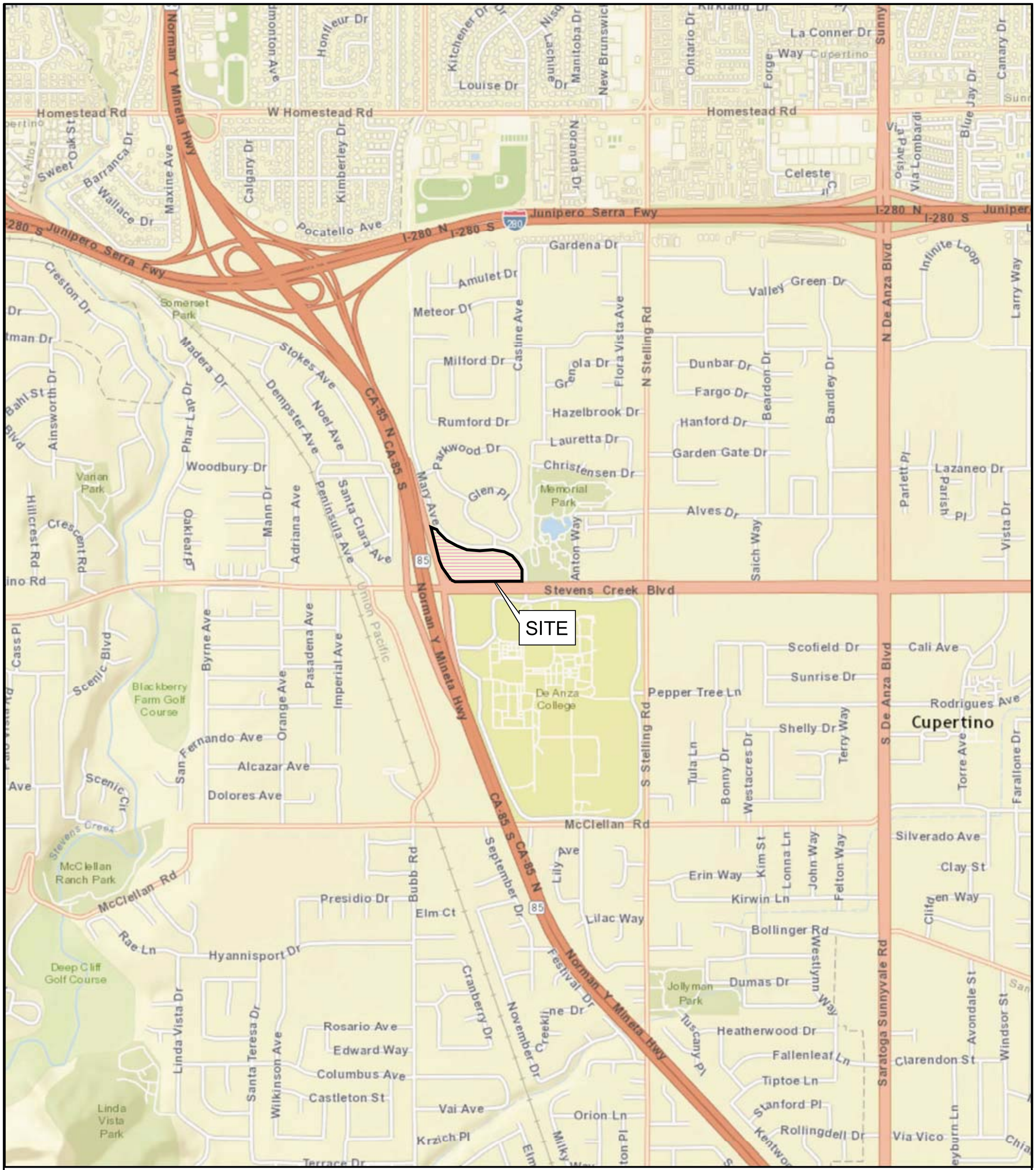
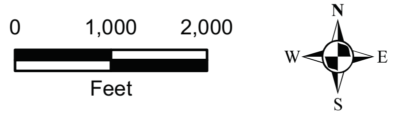
FIGURES



NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.



THE OAKS
Cupertino, California

LANGAN TREADWELL ROLLO

SITE LOCATION MAP

Date 10/09/14	Project No. 770619001	Figure 1

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EXPLANATION

B-1 ⊕ Approximate location of boring by Langan Treadwell Rollo, October 2014

— - - — Approximate site boundary

0 150 Feet

Approximate scale

Reference: Esri, Digital Globe, GeoEye, i-cubed, USDA, USGS AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and GIS User Community.

THE OAKS
Cupertino, California

SITE PLAN

LANGAN TREADWELL ROLLO

Date 10/09/14	Project No. 770619001	Figure 2

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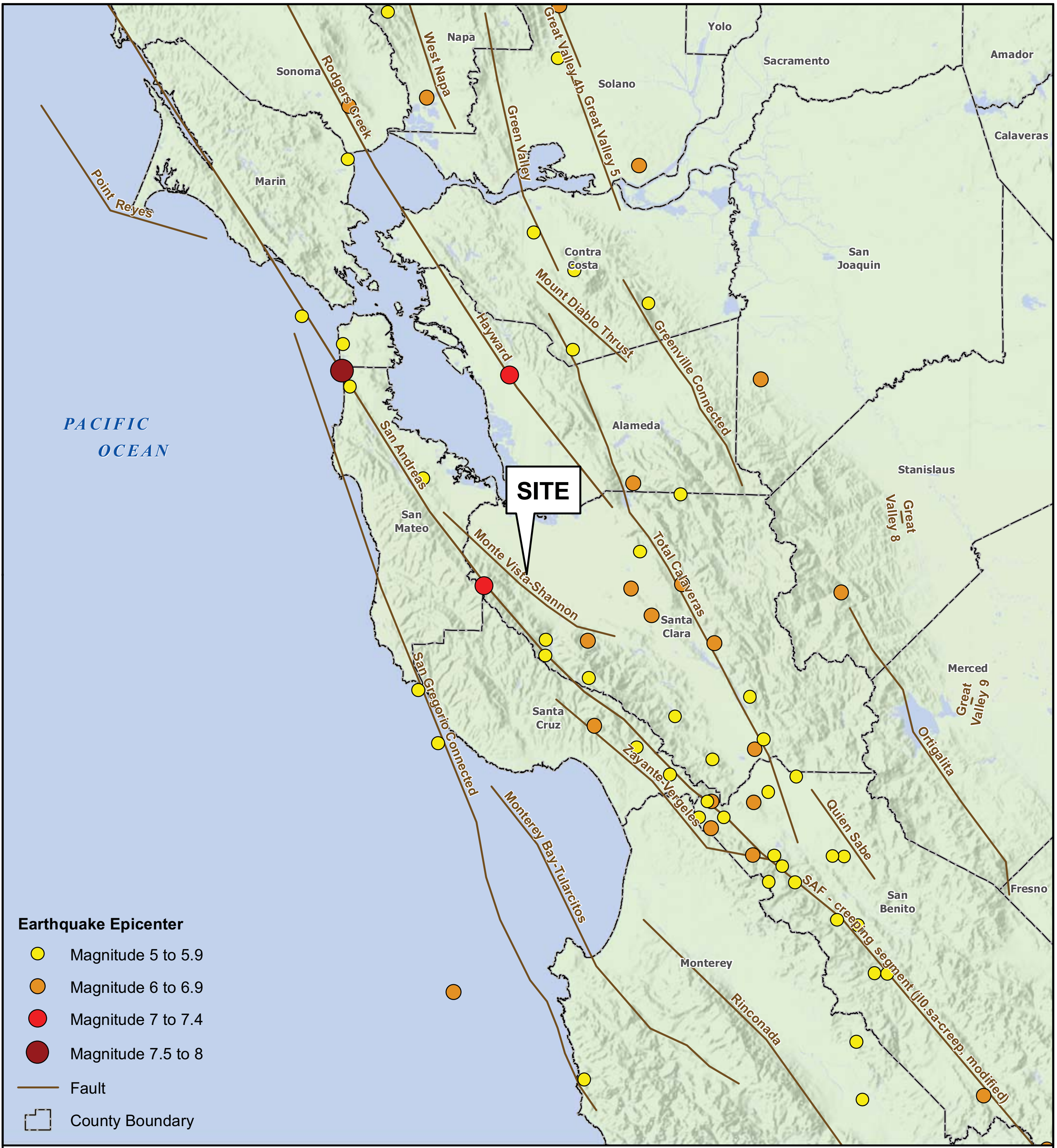


Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 08/26/2014.
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.

0 5 10 20

Miles

N

THE OAKS
Cupertino, California

LANGAN TREADWELL ROLLO

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 10/9/2014	Project No. 770619001	Figure 3

\\langan.com\data\SJ\data0\770619001\ArcGIS\ArcMap_Documents\Fault Map.mxd User: cyoung

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.

THE OAKS
Cupertino, California

MODIFIED MERCALLI INTENSITY SCALE

LANGAN TREADWELL ROLLO

Date 10/09/14 | Project No. 770619001 | Figure 4

APPENDIX A

LOG OF BORINGS

PROJECT: **THE OAKS**
Cupertino, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 10/2/14 Date finished: 10/2/14
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: M. Lattin

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 290.5 feet[2]	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						1.5 inches asphalt concrete (AC)						
1					CL	2 inches aggregate base (AB)						
	BULK	☒				SANDY CLAY with GRAVEL (CL) red-brown, dry, fine gravel up to 3/4 inch in diameter						
2–3	S&H	■	24 36 29	46								
3					CL	SANDY CLAY with GRAVEL (CL) red-brown to light brown, hard, dry, fine- to medium-grained sand, fine- to coarse subangular gravel up to 2 1/2 inches in diameter					5.1	109
4	BULK	☒										
6	S&H	■	31 49 38	61		increase in clay content						
7–8	S&H	■	29 60 50/ 4"	77/ 10"	CL	SANDY CLAY (CL) brown, hard, dry, fine-grained sand						
9						SAND with CLAY and GRAVEL (SP-SC) brown, very dense, dry, fine- to coarse-grained sand, fine- to coarse subangular to angular gravel up to 3 inches in diameter, trace fragmented cobbles						
10–11	S&H	■	12 40 50/ 3"	63/ 9"							3.7	109
13					SP- SC	increase in gravel content and angularity						
16	SPT	●	26 40 21	73								
17	S&H	■	14 21 46	47	SP	SAND (SP) brown, very dense, dry, fine-grained, trace fine angular gravel						
18						CLAY with GRAVEL (CL) brown, hard, dry, fine- to coarse angular gravel up to 1 inch in diameter, some fine- to coarse-grained sand						
20–21	S&H	■	34 70/ 6"	49/ 6"	CL	increase in gravel size; up to 2.5 inches in diameter to fragmented cobbles						
24						decrease in gravel size up to 1 inch in diameter						
25–26	S&H	■	47 65/ 6"	46/ 6"	SP	SAND with GRAVEL (SP) brown and yellow, very dense, dry, fine- to coarse-grained sand, fine- to coarse subangular gravel up to 1 inch in diameter					5.3	107
27												
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 770619001
Figure: A-1a

PROJECT: **THE OAKS**
Cupertino, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	SPT	◩	21 24 31	66		SAND with GRAVEL (SP) (continued)						
32					SP							
33												
34						SILTY SAND (SM) brown, medium dense, moist						
35												
36	SPT	◩	5 5 6	13								
37					SM							
38												
39												
40						CLAYEY SAND with GRAVEL (SC) red-brown, medium dense, moist, fine- to coarse-grained sand, fine gravel up to 3/4 inch in diameter						
41	S&H	■	9 12 15	19							12.1	112
42												
43					SC							
44												
45												
46	S&H	■	10 14 31	32	SW	SAND with GRAVEL (SW) brown, dense, moist, fine- to coarse-grained sand, fine rounded to subrounded gravel up to 1/2 inch in diameter						
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 46.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations are based on "Topographic Boundary and Utility Survey" by Kier and Wright, dated March 2003.

LANGAN TREADWELL ROLLO

Project No.: 770619001

Figure: A-1b

PROJECT: **THE OAKS**
Cupertino, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 10/2/14
Date finished: 10/2/14
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: M. Lattin

LABORATORY TEST DATA

Ground Surface Elevation: 296.0 feet[2]

DEPTH (feet)	SAMPLES 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
						2 inches asphalt concrete (AC)						
1						4 inches aggregate base (AB)						
2	BULK	☒			SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, very dense, dry, fine- to coarse-grained sand, fine- to coarse gravel up to 1 inch in diameter, trace cobbles						
3	S&H	■	19 29 44	51								
4	S&H BULK	■ ☒	50/ 2"	35/ 2"		decrease in gravel content						
5						SANDY CLAY (CL) brown, hard, dry, fine- to coarse-grained sand, trace fine gravel						
6	SPT	◩	10 20 25	54	CL							
7												
8	SPT	◩	12 16 22	46	SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, dense, dry, fine- to coarse-grained sand, fine gravel						
9												
10						SAND with GRAVEL (SP) brown, very dense, dry, fine to coarse-grained sand, fine- to coarse subrounded to angular gravel up to 3 inches in diameter, trace fragmented cobbles						
11	S&H	■	30 36 44	56							3.0	115
12												
13												
14					SP							
15												
16	S&H	■	20 28 20	34		dense, increase in gravel content, subrounded to subangular gravel					3.6	111
17												
18												
19						SANDY CLAY with GRAVEL (CL) brown, hard, moist, fine- to coarse-grained sand, fine angular and fragmented gravel up to 1/2 inch in diameter						
20					CL							
21	S&H	■	8 20 34	38								
22					GP-GC	GRAVEL with CLAY and SAND (SP-SC) brown, dense, moist, fine gravel up 1/3 inch diameter, fine- to coarse sand						
23												
24						SANDY CLAY (CL) brown, very stiff, moist, fine-grained sand						
25												
26	S&H	■	8 12 12	17	CL						20.7	94
27												
28												
29					ML	SANDY SILT (ML) brown, very stiff, moist						
30												

TEST GEOTECH LOG 770619001.GPJ TR.GDT 11/12/14

LANGAN TREADWELL ROLLO

Project No.: 770619001
Figure: A-2a

PROJECT: **THE OAKS**
Cupertino, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SANDY SILT (ML) (continued)						
31 32 33	S&H	▨	5 10 14	17	SC	CLAYEY SAND (SC) brown, medium dense, moist, fine- to coarse-grained sand, some fine gravel up to 1/4 inch in diameter					13.5	110
34 35					SP	SAND with GRAVEL (SP) brown, medium dense, moist, fine- to coarse-grained sand, fine angular gravel up to 1/2 inch in diameter						
36 37 38	S&H	▨	13 19 18	26	SM	SILTY SAND (SM) brown, medium dense, moist, some fine subrounded gravel						
39 40 41 42 43	S&H	▨	21 28 31	41	SP	SAND with GRAVEL (SP) brown, dense, moist, fine- to coarse-grained sand, fine- to coarse subrounded gravel up to 1 1/2 inches in diameter						
44 45 46	S&H	▨	15 20 22	29	SP	SAND (SP) brown, medium dense, moist, fine- to coarse-grained sand, some fine rounded gravel up to 3/4 inch in diameter					4.2	109
47 48 49 50 51 52 53 54 55 56 57 58 59 60												

Boring terminated at a depth of 46.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations are based on "Topographic Boundary and Utility Survey" by Kier and Wright, dated March 2003.

LANGAN TREADWELL ROLLO

Project No.: 770619001

Figure: A-2b

PROJECT: **THE OAKS**
Cupertino, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 10/3/14
Date finished: 10/3/14
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: M. Lattin

Ground Surface Elevation: 300.5 feet^2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches asphalt concrete (AC)						
						4 inches aggregate base (AB)						
1 2	BULK	☒			CL	SANDY CLAY (CL) red-brown, hard, dry, fine-grained sand LL = 23, PI = 9, see Figure B-1						
3	S&H	■	26 27 46	51		trace fine gravel up to 3/4 inch in diameter						
4	BULK	☒				SAND with CLAY and GRAVEL (SP-SC) brown, dense, dry, fine- to coarse-grained sand, fine- to coarse subangular to angular gravel up to 1 3/4 inch in diameter						
5 6	S&H	■	19 25 33	41							6.9	105
7												
8 9	S&H	■	20 34 35	48	SP- SC							
10 11	S&H	■	18 26 34	44		dark brown, coarse gravel and fragmented cobbles up to 3 inches in diameter					5.6	109
12												
13 14						SAND with SILT and GRAVEL (SP-SM) brown, very dense, dry, fine- to coarse-grained subrounded to angular gravel up to 2 3/4 inches in diameter, fine- to coarse-grained sand Sieve Analysis, see Figure B-2						
15 16	S&H	■	29 44 43	61						8.7	3.8	
17 18 19					SP- SM							
20	S&H	■	28 60/ 6"	42/ 6"		increase in sand content						
21 22	SPT	◩	17 15 14	35	SM	SILTY SAND (SM) brown, dense, dry						
23 24						SAND with GRAVEL (SP) brown, very dense, moist, fine- to coarse sand, fine- to coarse subrounded gravel up to 2 inches in diameter, with interbedded thin lenses of silt						
25 26	S&H	■	29 44 48	64	SP							
27 28 29 30												

LANGAN TREADWELL ROLLO

Project No.: 770619001
Figure: A-3a

PROJECT: **THE OAKS**
Cupertino, California

Log of Boring B-3

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	■	48 62 34	67		SAND with GRAVEL (SP) (continued) rounded to subrounded fine gravel up to 1/2 inch in diameter					9.1	108
32					SP							
33												
34												
35						SANDY CLAY (CL) brown, very stiff, moist						
36	SPT	◪	5 9 12	25	CL							
37												
38												
39						SAND with GRAVEL (SP) brown, very dense, moist, fine- to coarse-grained sand, fine- to coarse subangular gravel up to 1 inch in diameter						
40												
41	SPT	◪	11 23 40	76	SP							
42												
43												
44						SANDY CLAY (CL) brown, very stiff, moist						
45					CL							
46	SPT	◪	4 6 11	20		LL = 28, PI = 11, see Figure B-1						
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 46.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations are based on "Topographic Boundary and Utility Survey" by Kier and Wright, dated March 2003.

LANGAN TREADWELL ROLLO

Project No.: 770619001

Figure: A-3b

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	**Grain Size in Millimeters**
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

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

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

THE OAKS
Cupertino, California

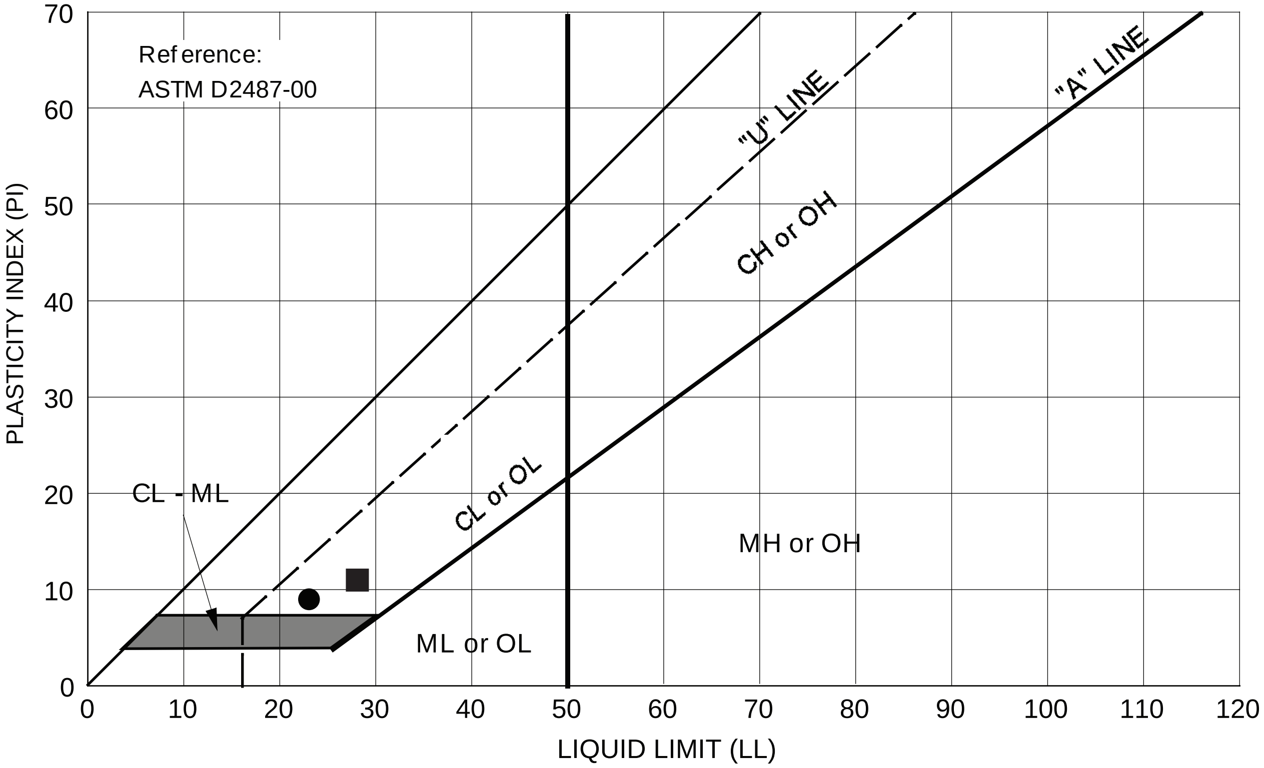
CLASSIFICATION CHART

LANGAN TREADWELL ROLLO

Date 10/09/14 | Project No. 770619001 | Figure A-4

APPENDIX B

LABORATORY DATA



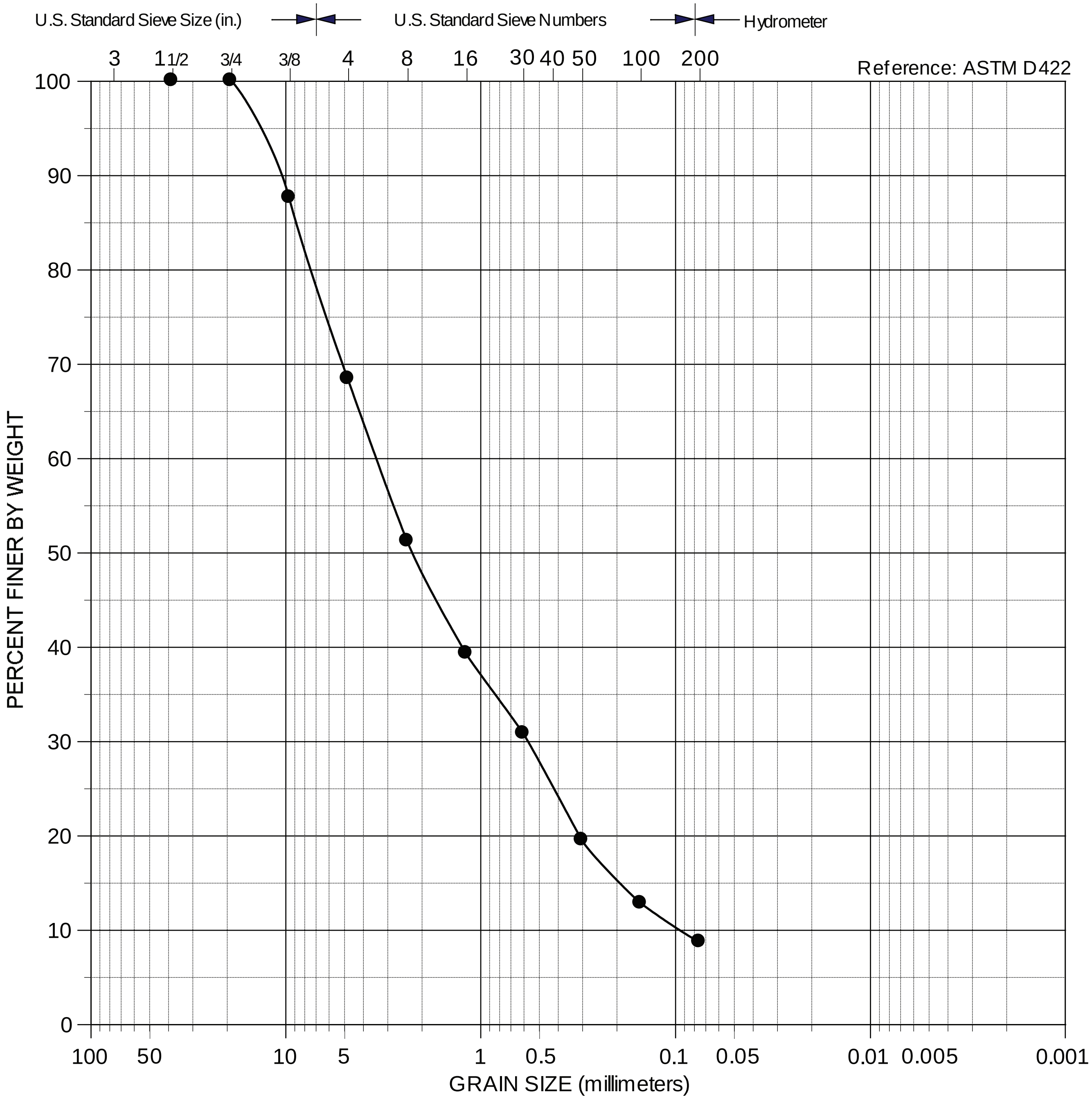
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-3 at 1-2.5 feet	SANDY CLAY (CL), red-brown	--	23	9	--
■	B-3 at 45 feet	SANDY CLAY (CL), brown	--	28	11	--

THE OAKS
Cupertino, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 10/28/14	Project No. 770619001	Figure B-1



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

Symbol	Sample Source	Classification
●	B-3 at 16 feet	SAND with SILT and GRAVEL (SP-SM), brown

THE OAKS
Cupertino, California

LANGAN TREADWELL ROLLO

PARTICLE SIZE ANALYSIS

Date 10/28/14	Project No. 770619001	Figure B-2

APPENDIX C

CORROSIVITY ANALYSES WITH BRIEF EVALUATION

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

16 October, 2014

Job No.1410096
Cust. No.12242

Mr. Matt Lattin
Langan Treadwell Rollo
4030 Moorpark Avenue, Suite 210
San Jose, CA 95117

Subject: Project No.: 770619001-700-318
Project Name: The Oaks, Cupertino, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Lattin:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on October 13, 2014. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are 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 16 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 69 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.00 to 7.44, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 310 to 330-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.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

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.: 770619001-700-318
Client's Project Name: The Oaks, Cupertino, CA
Date Sampled: 2-Oct-14
Date Received: 13-Oct-14
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 16-Oct-2014

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)*
1410096-001	B-2-2	310	7.44	-	2,800	-	16	69
1410096-002	B-1-4	330	7.00	-	2,100	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	15-Oct-2014	15-Oct-2014	-	14-Oct-2014	-	15-Oct-2014	15-Oct-2014

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

1410096

Chain of Custody

12242

Page 1 of 1

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775
CERCC analytica

Job No.	CU#		Client Project I.D.
770619001	700	318	The Oaks

Schedule:
Analyte:

Date Sampled: 10/2/14

Date Due: Normal turnara

Full Name: Matt Lattih

email ~~Fax~~ mlattih@langan.com

Phone: 916-844-8148

Cell: 925-955-5266

☒

Company: Langan Treadwell Rollo

Sample Source: Cupertino, CA

ASTM w/Brief Evaluation

ANALYSIS

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation
	B-2-2	10/2/14		S	bag			1	x	x	x	x	x		x
	B-1-4	10/2/14		S	bag			1	x	x	x	x	x		x

GSO

MATRIX
- DW - Drinking Water
- GW - Ground Water
- SW - Surface Water
- WW - Waste Water
- Water
- SL - Sludge
- S - Soil
- Product

ABBREVIATIONS
- HB - Hosebib
- PV - Petcock Valve
- PT - Pressure Tank
- PH - Pump House
- RR - Restroom
- GL - Glass
- PL - Plastic
- ST - Sterile

SAMPLE RECEIPT
- Total No. of Containers
- Rec'd Good Cond/Cold
- Conforms to Record
- Temp. a t Lab -°C
- Sampler

		Date	Time
Relinquished By:		10/10/14	10 am
Received By:	Lim MDurker	10/13/14	1445
Relinquished By:			
Received By:			
Relinquished By:			
Received By:			

Comments:

THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES

8/6/2009

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