Geotechnical Investigation

Skylonda Fire Station No. 58
17290 Skyline Boulevard

Woodside
San Mateo County, California

Prepared for County of San Mateo
Department of Public Works

April 10, 2015
#2014-128G

Rutherford + Chekene
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April 10, 2015

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2014-128G

Subject: **GEOTECHNICAL INVESTIGATION**
SKYLONDA FIRE STATION NO. 58
17290 SKYLINE BOULEVARD
WOODSIDE, CALIFORNIA
#PC008, RESOLUTION NO. 073246

Dear Ms. Yee:

We are pleased to transmit herewith our report covering the subject geotechnical investigation. The scope of our services was described in our proposal dated November 12, 2014.

This report contains a summary of geotechnical recommendations developed for the design of the facility, as well as the results of our field exploration, laboratory testing, and engineering analyses that form the basis of our recommendations.

We understand that this report will be part of the bridging documents package that prospective design-build teams will use to prepare their bids. We anticipate that recommendations contained in this report will be incorporated into all contract documents prepared by the selected design-build team and that we would be given the opportunity to review those contract documents for conformance with our recommendations. We also anticipate that supplementary geotechnical recommendations aimed at addressing design issues arising during the design-build phase will be provided by the geotechnical engineer for the design-build team.

We greatly appreciate the opportunity to be of service to you on this project. If you have questions regarding this report, please contact us.

Sincerely,

RUTHERFORD + CHEKENE

John C. Burton, GE #177
Geotechnical Engineer

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INTRODUCTION

General

This report summarizes the results of the findings of the geotechnical investigation performed for the Skylonda Fire Station No. 58, at 17290 Skyline Boulevard in Woodside, San Mateo County, California. The location of the site is shown in Figure 1 - Site Vicinity Map.

The overall geotechnical investigation program consists of the following two phases:

1. Gathering of geotechnical data through field exploration and laboratory testing.

2. Interpretation and analysis of the geotechnical data for the sole purpose of developing recommendations for design.

Site Description

The project site is located on the existing Skylonda Fire Station No. 58 property, along the southwest side of Skyline Boulevard and north of its intersection with La Honda Road. The fire station adjoins the property of Alice's Restaurant on the southeast, and is bounded by Linwood Way on the northwest. Skyline Boulevard forms the northeasterly edge of the property, and the southwesterly boundary is along Blakewood Way and the adjacent reservoir.

The existing fire station consists of three buildings placed roughly in a line along the Skyline Boulevard side of the property: the apparatus building, the office building, and the barracks building. The apparatus building is a metal structure, while the office and barracks buildings are older wooden structures. Access to the site is currently via a driveway that enters from the parking area for Alice's Restaurant and runs along the southwest side of the barracks and office buildings, to a wide and flat paved area in front of the apparatus building. An access driveway continues to a second entrance onto Linwood Way.

Site Elevations

We have based the site elevations in this report on a site plan with topographic map background, prepared by BKF Engineers of Redwood City, dated February 12, 2015 and provided to us by the County of San Mateo.

Project Description

As we understand from our site meeting on October 28, 2014, the project as currently proposed will consist of constructing a new building to house the office and barracks functions, then demolishing the existing office and barracks buildings, and constructing a new access driveway directly onto Skyline Boulevard, approximately in the area now occupied by the office building. The new office/barracks building will be located southwest of the existing apparatus building, which will remain. The new building is anticipated to be a two-level structure, either with its main level at the existing driveway elevation and a lower level stepping down the slope to the southwest, or with its main level at the existing driveway elevation with a second level above. The sanitary sewer leach field that currently serves the facility is located under the paved driveway. It will be upgraded to current code requirements, in the existing location, and overlain by permeable paving.

Preliminary Geotechnical Investigation – BAGG Engineers (2013)

A preliminary geotechnical and geologic evaluation report[1] was prepared in 2013 by BAGG Engineers. Their evaluation was based on literature research and site reconnaissance; site-specific investigations or laboratory testing was not performed at that time. The BAGG report addresses the regional and site geology and seismicity, as well as geologic hazards at the site. The BAGG report indicated that the site conditions are generally favorable for the proposed project, with no major geologic hazards specific to the site, such as liquefaction, fault rupture, lateral spreading, slope instability, flooding, or expansive soil. Our findings from the present investigation concur with their preliminary findings, so these aspects are not duplicated here.

Previous Geotechnical Investigation – Cleary Consultants (1996)

A geotechnical investigation was performed on the site and a report[2] was prepared in 1996 by Cleary Consultants, Inc. Their investigation was performed for a new barracks/office building planned in a location similar to the currently-proposed project. The investigation included six borings, laboratory testing of samples, engineering analysis, and geotechnical recommendations. The Cleary report was not available until very late in the current investigation, but its subsurface information has been incorporated in this report and augments the basis for our recommendations. The locations of Cleary's 1996 borings and subsurface profiles are shown on Fig. 2 – Site and Boring Location Plan, and boring logs, laboratory test data and subsurface profiles from the Cleary report are reproduced and included as Appendix F.

Summary of Field Exploration and Laboratory Testing

We performed field exploration and laboratory test programs to gather subsurface information and laboratory test data for use in subsequent engineering analysis of the various components of the project.

The field exploration program involved the drilling and sampling of five exploratory borings. Details regarding this exploration program are contained in Section 4. The subsurface information gathered is presented in Appendix B.

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1 *Preliminary Geotechnical & Geologic Report, Skylonda Fire Station No. 58, 17290 Skyline Boulevard, San Mateo County, California,* by BAGG Engineers, dated November 27, 2013 (BAGG Job No. MWAAR-01-00).

2 *Geotechnical Investigation, New Barracks and Office Building, Skylonda Fire Station, 17290 Skyline Boulevard, Woodside, San Mateo County, California,* by Cleary Consultants, Inc., dated March 29, 1996 (Cleary Project No. 869.1).

The laboratory testing program consists of index, strength and corrosivity tests. Details regarding the laboratory test program are also contained in Section 4. The results of the index and strength tests are presented in Appendix C, and the results of the corrosivity tests are presented in Appendix E.

Limitations

1. This report has been prepared for the exclusive use of the County of San Mateo Department of Public Works and its consultants for specific application to the Skylonda Fire Station No. 58 project as described herein. In the event that there are any changes in ownership, nature, location or design of the project, the information contained in this report shall not be considered valid unless the project changes are reviewed by Rutherford + Chekene.

2. Any conclusions contained in this report are based in part upon the data obtained from exploratory borings and laboratory testing performed as part of this and previous investigations. The nature and extent of variations between the borings may not become evident until construction. If variations are discovered, it will be necessary to re-evaluate any conclusions contained in this report.

3. Simplified interpretations of geotechnical data have been made to facilitate the geotechnical analysis performed for this project. Such interpretations, while adequate for the analysis performed, are inadequate for estimating quantities for the purposes of developing construction costs or submitting bids for this project. These interpretations should therefore not be used for purposes other than the stated intended purpose.

4. This report should not be part of the contract documents for the proposed project described herein. Instead, the report should serve as a guide for preparing design drawings and specifications that are part of the contract documents.

5. We cannot be responsible for the impacts of any changes in geotechnical or geologic standards, practices, or regulations subsequent to the performance of our services if we are not consulted subsequent to the changes.

6. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for use of segregated portions of this report without consultation with our office.

7. The opinions set forth in this report are not based upon an examination of the location or condition of utility lines or other subsurface structures on the property. Those performing the construction must assume any risks arising from the locations or conditions of such lines.

8. Rutherford + Chekene assumes no responsibility for the management of contaminated or hazardous materials that may be found on the site.

a. Rutherford + Chekene has not performed investigations to determine the presence of contaminated or hazardous materials. The Owner must provide the results of any such investigations to the Contractor.

b. The Construction Contractor is responsible for ensuring that personnel within the work area are protected from hazardous materials. If hazardous materials are discovered, the Contractor must immediately notify the Owner and cease work until conditions can be maintained in accordance with all applicable regulations.

SECTION 2
SITE CONDITIONS AND GEOLOGIC HAZARDS

GEOLOGY AND SEISMICITY

Regional Geology

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates. As the Farallon plate subducted under the North American plate, the rising Pacific plate collided with the North America plate, creating the subsequent right-lateral-strike-slip shearing along the San Andreas Fault zone. Regional geologic mapping[3] identifies the site vicinity to be within the Sky Londa Assemblage and underlain by Lambert shale, of Oligocene to lower Miocene age.

Site Geology

The youngest deposit on the site consists of fill placed during grading and construction of the existing fire station. Fill is present on the southwest side of the apparatus yard, which was likely created by cutting into the hill toward Skyline Blvd. and placing the excavated materials as fill. The wedge of fill formed in this process meets the original grade on the slope above Blakewood Drive. Boring RC-2, located near the outer edge of the fill, encountered 9 feet of fill. Other borings (RC-1, 3 & 4), located farther back from the top of the fill slope, encountered between 3 and 6 feet of fill. Borings by Cleary Consultants (1996) encountered similar thicknesses of fill, in the range of 4.6 to 6.3 feet. The fill wedge is expected to taper out near the middle of the yard.

Beneath the fill and in undisturbed areas of the site, native colluvial soil occurs over the bedrock. Colluvium is absent in places, and variable in thickness where it occurs. In our borings, it varied from 1.5 to 5 feet in thickness in three borings and was absent in two borings. Similarly, in the Cleary borings, it ranged from zero (in one boring) to 5.7 feet thick. The colluvium consists generally of dark brown stiff sandy clay.

The predominant formation at the site is the Lambert shale bedrock. Although the Lambert Shale formation overall is referred to as shale, the rocks within the formation present on the site are claystones, siltstones, and sandstones. In general, these rocks are thin-bedded with low hardness, and are friable and deeply to moderately weathered. These materials are exposed in the open cut face behind the east wall of the apparatus building, where they were excavated to create the building pad.

Faulting and Seismicity

Major Active Faults: The San Andreas Fault Zone lies approximately 2 km east-northeast of the site. The Fault Zone splits from a very linear trace in Central California approximately 95 km southwest of the San Francisco Peninsula. The Hayward–Calaveras fault system trends up the east side of the San Francisco Bay, while the San Andreas fault proper follows the Peninsula on

[3] Brabb, E.E., Graymer, R.W. and Jones, D.L., *Geology of the Onshore Part of San Mateo County, California: A Digital Database,* USGS Open-File Report 98-137, 1998.

the west side of the Bay. The Hayward fault is about 32 km northeast of the site and the Calaveras fault is about 40 km east-northeast of the site. A third strike-slip fault zone, the San Gregorio, is about 13 km west-southwest of the site. It crosses the westernmost part of the Peninsula at Año Nuevo and Pillar Point and then trends offshore toward the Golden Gate where it merges with the San Andreas fault before the main trace trends north through Bolinas and Tomales Bays.

Monte Vista Fault and the Foothills Thrust System: The thrust and reverse faulting that has been mapped along the northeastern foot of the Santa Cruz Mountains are geologic structures, subsidiary to the San Andreas Fault Zone, and can be attributed to the compressional tectonic environment. At the southern end of the Peninsula, the northeast flank of the Santa Cruz Mountains marks the start, and widest expression, of the northwest trending Foothills Thrust System. At the northern end of the Peninsula, the Foothills Thrust System appears to die out to the north in a narrow band of two or three surface traces of the Serra Fault Zone. No trace of the thrust system has been mapped.

The Monte Vista fault is a potentially active fault mapped approximately 4.8 km southeast of the site. Several sub-parallel, generally southwest-dipping faults including the Monte Vista fault (Dibblee, 1966; Sorg and McLaughlin, 1975; William Cotton and Associates, 1978) trend along the northeast flank of the Santa Cruz Mountains from the vicinity of Los Gatos/Highway 17 northwest to just northwest of Page Mill Road in Palo Alto. These faults expose older rocks in their southwest walls suggestive of thrusting or reverse-slip. The fault geometry is compatible with uplift of the Santa Cruz Mountains relative to the Santa Clara Valley.

The Foothills Thrust System is believed to place Franciscan Complex bedrock over alluvial deposits in the Santa Clara Valley. The age of the youngest alluvial deposits juxtaposed with Franciscan Complex rocks is estimated at approximately 20,000 years old (Late Pleistocene; CDMG, 1980). Mapping of the fault zone characteristically shows Santa Clara Formation gravels cut by the faulting, indicating an age of younger than 1 million years.

The Pilarcitos fault, considered inactive, is mapped about 0.8 km northeast of the site.

Seismicity: The site lies in the seismically active San Francisco Bay region and is subject to frequent ground shaking. Significant earthquake scenarios associated with faults nearest the site were presented in Table 1 of the BAGG preliminary report, so are not repeated here.

The site does not lie within a known active fault zone. No other faults were identified on the site during our investigation.

A number of historical earthquakes have affected the area, including the 1906 San Francisco earthquake and the 1989 Loma Prieta earthquake. During a major earthquake on any one of the nearby active faults, the site may experience strong ground shaking.

The U.S. Geological Survey's 2007 Working Group on California Earthquake Probabilities (2008) has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region during the next 30 years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek and the Northern segment of the San Andreas faults. These probabilities are 31 and 21 percent, respectively (USGS, 2008).

SUBSURFACE CONDITIONS AND GEOLOGIC HAZARDS

Soil Conditions

The project site is underlain by bedrock of the Lambert shale formation, covered by varying amounts of colluvial soil and artificial fill. These earth materials fall under the following three categories:

1. Fill: The fills placed to create the southwest portion of the apparatus yard were likely derived from the excavation of the apparatus building pad. The fill materials consist primarily of moist, soft to stiff, sandy clay of medium plasticity with variable amounts of gravel. We have no records indicating that the fill was compacted as engineered fill. While the overall behavior of the fill appears to have been good, because of the lack of documentation and its variable consistency, new structures should not be supported on the existing fill.

2. Colluvium[4]: The natural colluvial soils consist of a variable thickness of dark brown stiff sandy clay of medium plasticity. In some places, colluvium is not present over bedrock. Where present, undisturbed and firm colluvium is a suitable bearing material to support new structures.

3. Bedrock: The Lambert Shale formation bedrock at the site consists primarily of claystone, siltstone, and sandstone. In general, these rocks are thin-bedded with low hardness, and are friable and deeply to moderately weathered. The Lambert formation forms the primary foundation stratum for new structures, which can be supported either on drilled piers extending into the rock, or on spread footings bearing on rock.

Groundwater Conditions

A continuous groundwater body was not encountered in the borings. However, perched groundwater was encountered in two of the borings (RC-2 and RC-4) located near the middle of the planned building. In both cases, the perched groundwater was encountered within the bedrock. In boring RC-4, perched groundwater occurred at a depth of 16.5 feet (approximate elevation 1467.0), while at boring RC-2, located about 40 feet to the southwest, i.e. in a downslope direction, the perched groundwater was encountered at a depth of 19 feet (approximate elevation 1463.2). The observed water surface gradient of 4.3 feet vertical in 40 feet horizontal, or about 11%, between the two borings suggests that this perched groundwater occurs in a more permeable (more heavily fractured and less clayey) zone of rock and is flowing is a direction roughly parallel to the original ground surface slope.

Groundwater was similarly encountered in the Cleary investigation, during March 1996. Our interpretation of the Cleary logs suggests that the groundwater surface measured in 1996 was

[4] Colluvium: Unconsolidated sediments that have been deposited by the action of gravity and slope processes.

about three feet higher than we measured in December 2014 in the planned building area. Cleary also observed and mapped three seeps (groundwater slowly seeping from the ground surface, similar to a spring) in the toe of the slope along Blakewood Way. The location of the mapped seeps is shown on Fig. 2 – Site and Boring Location Plan, and the relationship between the interpreted groundwater level in 1996 and the recent measurements is shown in Fig. 3 – Subsurface Profile A-A. These conditions would be consistent with a sloped groundwater surface parallel to, but higher than, the surface measured in our recent borings. No evidence of seeps was observed along Blakewood Way during a site visit on March 30, 2015 (similar time of year to when seeps were mapped by Cleary). A lower groundwater surface this year is also consistent with the drought conditions that have prevailed over the last couple of years.

The groundwater encountered in our investigation, as well as by Cleary in 1996, is below the planned basement level, so is unlikely to affect the basement construction itself. However, drilled piers are likely to extend to elevations where perched groundwater could be encountered during pier installation. Subdrainage and waterproofing of the basement level should also be provided in anticipation that perched groundwater could occur at higher elevations and build up beneath the basement floor slab and behind the basement wall.

Geologic Hazards

Geologic hazards at the site were evaluated by the recent BAGG preliminary study, including faulting and fault-related ground surface rupture; liquefaction; lateral spreading; slope instability; flooding; tsunami and seiches; and expansive soils. The potential for these hazards at the site was deemed to be low to nil. In the course of the present investigation, we have not discovered any evidence contrary to their conclusions, therefore we concur with BAGG's findings and do not repeat them here.

MITIGATION OF THE POTENTIAL IMPACTS OF HAZARDS

Mitigation of Potential Geologic Hazards

The following subsections of this report discuss mitigation of the two geologic hazards that were considered to have a high likelihood of occurrence: strong ground shaking and soil corrosivity.

Ground Shaking

The primary approach to mitigating the potential impacts of ground shaking on the proposed facility is to design the new building in accordance with the current seismic design code. We have therefore developed recommendations for seismic design parameters in accordance with the 2013 California Building Code (CBC). Criteria for the seismic design of new project elements are presented in a subsequent section of this report under the subheading "Seismic Design Criteria."

Soil Corrosivity

We recommend that adequate cover should be provided on reinforcement for foundations, and buried utility lines should be corrosion-protected according to the recommendations of a qualified Corrosion Engineer.

SECTION 3
DESIGN RECOMMENDATIONS

DESIGN RECOMMENDATIONS

Seismic Design Criteria

The primary approach to mitigating the potential impacts of ground shaking on the proposed improvements is to design them in accordance with current seismic design codes. We have therefore developed recommendations for seismic design parameters in accordance with the 2013 California Building Code (CBC), as presented below.

Latitude and Longitude: The project site has the following coordinates:

Latitude: 37.38746 degrees North

Longitude: 122.26685 degrees West

Site Class/Soil Profile Type: C – Very Dense Soil and Soft Rock

Seismic Design Parameters for Site Class C: The seismic design parameters in the table below for Soil Profile S_C are applicable. The parameters can also be obtained from the United States Geological Survey (USGS) website: (http://earthquake.usgs.gov/designmaps/us/application.php), "US Seismic Design Maps."

Table 1
2013 CBC Seismic Design Parameters Based on Mapped Spectral Accelerations

Site Class	C	
Mapped Spectral Response Acceleration Parameters	S_S (From 0.2 sec Mapped Spectral Accelerations)	2.474
	S_1 (From 1.0 sec. Mapped Spectral Accelerations)	1.094
Site Coefficients	F_a (From Table 1613.3.3(1) of 2013 CBC)	1.0
	F_v (From Table 1613.3.3(2) of 2013 CBC)	1.3
Adjusted MCE Spectral Acceleration Parameters	$S_{MS} = F_aS_S$	2.474
	$S_{M1} = F_vS_1$	1.423
Design Spectral Acceleration Parameters	$S_{DS} = 2/3S_{MS}$	1.649
	$S_{D1} = 2/3S_{M1}$	0.949

Foundations - General

New structures and improvements on the site may be supported using two types of foundations. All major structures and large retaining walls should be supported on drilled piers founded in the Lambert Shale formation bedrock. The overlying stiff and undisturbed colluvial soils, where they occur, may also be included for the purposes of computing pier lengths. Minor retaining walls and other sitework may be supported on shallow spread footings.

Drilled Piers

All major structures and large retaining walls should be supported on drilled cast-in-place concrete piers designed and constructed according to the recommendations presented below. Drilled piers should be designed to resist axial compressive and uplift loads through friction between the shaft walls and the surrounding Lambert Shale formation bedrock and overlying firm undisturbed colluvial soil, where it occurs. Skin friction contributions within the existing fill materials should be neglected. The end-bearing capacity of the drilled piers should also be neglected because the end-bearing contribution is likely to be mobilized only at unacceptably large settlements.

Size and Spacing: We recommend using drilled piers with a minimum diameter of 18 inches. Drilled piers should have a minimum center-to-center spacing of three times the pier diameter.

Axial Compressive Loads: The average values of allowable skin friction for the drilled piers given in Table 2 can be used for design.

Table 2
Allowable Skin Friction for Drilled Piers
Under Axial Compressive Loading

Load Case	**Average Allowable Skin Friction (psf)**
Dead + Live Loads	600
Dead + Live + Seismic	800

Ultimate Axial Compressive Loads: If it is necessary to obtain ultimate values, multiply the allowable values given in Table 2 by two.

Axial Uplift Loads: The allowable uplift capacity for drilled piers may be taken as 3/4 of the allowable axial compressive capacity for the loading condition under consideration.

Settlement: The settlement of drilled piers designed and constructed in accordance with these recommendations is expected to be less than one-quarter inch.

Lateral Resistance: The pier length required to resist lateral forces may be determined by the code pole formula (2013 CBC, Section 1807.3), using a lateral soil resistance value of 375 psf/foot, beginning at the top of the native soil or rock (neglect lateral bearing within existing fill materials).

Reinforcing: Piers should be reinforced for their full length. Reinforcing should be determined by the structural engineer according to the requirements of the structure.

Drilling Conditions: The ground conditions for drilling and casting piers are expected to be generally favorable. However, perched groundwater may be encountered, which would require dewatering of the holes before casting, or placement of concrete by the tremie method if dewatering is not effective. The Lambert Shale formation bedrock is expected to be drillable using conventional truck-mounted auger drilling equipment with a kelly bar system capable of exerting a substantial crowd force, together with an auger fitted with rock-drilling teeth (rock auger).

Observation: The drilled pier installation process should be observed by the Geotechnical Engineer on a continuous basis, to verify the subsurface conditions assumed in developing the pier design recommendations, and to confirm that proper pier installation procedures have been followed.

Spread Footings

Minor structures and low retaining walls (site walls) may be supported on conventional shallow spread footings, bearing in firm native colluvial soils or Lambert Shale formation bedrock (not existing fill). To avoid the potential for differential settlement to occur between portions of a structure supported on different foundation systems, i.e. drilled piers and spread footings, the two systems should not be used in combination to support a single structure. Where a spread footing supported structure, such as a site wall, abuts a drilled pier supported structure, an isolation joint should be provided to accommodate differential settlement due to the expected difference in foundation behavior.

Spread footings should be designed in accordance with the bearing pressures presented in Table 3. The footings should have a minimum width of 18 inches and should be embedded at least 18 inches below the lowest adjacent grade.

Table 3
Allowable Bearing Pressures for Footings

Loading Condition	Bearing Pressure (psf)	Immediate Total Settlement (in.)	Differential Settlement (in.)
Dead + Live Loads	2,500	0.5	0.5
Dead + Live + Seismic Loads	3,500	---	---

Lateral loads applied to a footing may be resisted by: 1) friction at the base of the footing; and 2) passive pressure against the side of the footing perpendicular to the applied force. These components of resistance may be assumed to act together at the limit state, and so may be added to estimate the total resistance available.

The horizontal frictional resistance, F_{base}, at the interface of soil and a footing may be taken at:

$$F_{base} = 0.30 \text{ x Applied Bearing Pressure (psf)}$$

A passive pressure beginning at zero at surrounding grade, increasing with depth as a 270 pounds per cubic foot equivalent fluid pressure, may be assumed to act against the side of the footing.

Construction of Footings

To assure that the recommended bearing pressure and passive and frictional resistances are developed from all footings, they should be cast directly against firm native earth materials.

The following measures are recommended to minimize the potential detrimental impacts of footings excavations on foundation performance:

1. Footing excavations should be thoroughly cleaned of all loose materials immediately prior to concrete placement. Usually, the effort to clean the excavations is hampered by the presence of reinforcing bars in the excavations, making this a less-preferred approach than the option described below for creating acceptable bearing conditions.

2. The bottom of the foundation excavations may be covered with a thin lean concrete layer after suitable bearing conditions have been established. This lean concrete layer would ensure that the bearing conditions are maintained, provide a firm surface for placing the footing reinforcement, and ensure adequate concrete cover on the bottom reinforcing bars. Also, any loose materials that accumulate in the excavation can be easily removed using air-blowing techniques. We recommend that the Contractor utilize this approach if footings are to be installed during the rainy season.

We should be given the opportunity to observe the bearing conditions prior to the placement of reinforcement and immediately before concrete placement. Remedial work should be performed, if necessary, until the bearing conditions are deemed to be satisfactory by the Geotechnical Engineer. The responsibility to maintain suitable bearing conditions and control sloughing of the sides of the excavation should remain with the Contractor.

Where materials exposed in footing excavations are disturbed (as determined by the Geotechnical Engineer) by the excavation operations, a reasonably smooth surface should be prepared for foundation placement by removal of loose materials as directed by the Geotechnical Engineer.

Retaining Walls

Retaining walls should be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by earthquakes and/or surcharge loads, as described below. The design lateral earth pressures recommended below do not include contributions from hydrostatic pressures. Thus, a subdrain system should be provided behind retaining walls.

Retaining walls should be designed to resist lateral earth pressures from: 1) the static case and surcharge-induced pressures, if any; and 2) the dynamic case and surcharge-induced pressures, if any. The recommended design lateral earth pressures are as follows:

1. *Static Loading:* Use the following static equivalent fluid pressures for cantilever or top-restrained walls, with the slope gradient applicable to the surface slope of the retained soil. For slope gradients between the values given, determine the applicable design pressures by linear interpolation.

Retained Soil Slope (horizontal: vertical)	**Cantilever Wall** (pcf)	**Top-Restrained Wall** (pcf)
Horizontal (level)	40	60
3:1	45	69
2:1	57	87

2. *Seismic Surcharge Loadings:* For a wall height of H feet, the dynamic earth pressure increment imposed by an earthquake should be assumed to be a uniform pressure of the magnitude indicated in the table below. The associated static lateral earth pressure should be equal to the static value for cantilever walls and may be reduced to the value indicated in the table below for top-restrained walls. The total lateral earth pressure for either the cantilever or the top-restrained case is equal to the sum of the dynamic earth pressure increment and the static earth pressure.

Retained Soil Slope (horizontal: vertical)	**Seismic Increment** (psf)	**Reduced Static Pressure for top-restrained wall** (equivalent fluid, pcf)
Horizontal (level)	15H	50
3:1	18H	57
2:1	22H	72

3. *Surcharge-Induced Pressures:* A uniform lateral pressure equal to the uniform vertical pressure that could occur behind a wall, multiplied by the surcharge coefficient shown in the table below, should be used to account for a surcharge <u>directly</u> behind walls. This approach applies only to loadings separate from and in addition to the slope conditions accounted for in 2, above.

Retained Soil Slope (horizontal: vertical)	**Surcharge Coefficient (cantilever wall)**	**Surcharge Coefficient (top-restrained wall)**
Horizontal (level)	0.31	0.47
3:1	0.36	0.55
2:1	0.46	0.70

4. *Other Surcharge-Related Issues:* Surcharge pressures on retaining walls resulting from loads, such as foundations, that are located some distance behind the walls should be

evaluated on a case-by-case basis. In general, it can be assumed that there will be no surcharging influence from loads that are applied outside, or below, a 1.5:1 (horizontal: vertical) line. Within such an influence zone, however, surcharge effects should be evaluated individually.

A subdrain system should be installed to prevent hydrostatic pressures from developing against the retaining wall. The subdrain should consist of prefabricated drainage panels (Miradrain, or equal) with filter fabric on the side facing the earth, draining either into weep holes through the wall, or into a collector pipe running along the bottom of the wall. As alternatives to prefabricated drainage panels, clean drain rock or permeable material at least one foot thick may be used. If clean drain rock is used, it should be encased in filter fabric to prevent infiltration of the adjacent soil backfill. If permeable backfill material is used without filter fabric, it should conform to the gradation requirements for Class 2 Permeable Material as specified by the California Department of Transportation (Caltrans) Standard Specifications, Section 68.

Slabs

Interior Slabs: The design requirements for interior slabs-on-ground can be summarized as follows: a) prevent dampness and efflorescence in the slab; and b) support anticipated loads on the slab. To fulfill these objectives, the following section is recommended for slab-on-grade floors:

1. Reinforced concrete slab of minimum five-inch thickness. The amount of reinforcing should be determined by the designer, taking into account the anticipated use, expected loads on the slab, and desired performance.

2. Impervious membrane of good quality, per ASTM E1745, Class C. The membrane should be Stego Wrap or approved equal.

3. Granular cushion, with a minimum nominal thickness of four-inches and consisting of broken stone or crushed or uncrushed gravel, angular and free of deleterious matter. The gradation should conform to the following:

U.S. Series Sieve Size	**Percentage Passing Sieve (Dry Weight Composition)**
3/4-inch	100
No. 4	0-10
No. 200	0-2

The granular cushion should be compacted with a vibro-plate before subsequent construction. If preventing dampness and efflorescence is not necessary, the membrane can be eliminated.

Subdrainage and Waterproofing at Basement Floor Slabs: To provide additional protection against moisture and dampness in the basement, in the event that groundwater levels rise above those observed in this investigation, we recommend installing a drainage blanket and subdrain system beneath the basement floor slab. The drainage blanket should consist of a minimum

12-inch thick layer of clean ¾-inch drain rock, with a subdrain system of perforated collection piping leading to discharge points outside the building. Subdrain piping size and spacing should be selected by the building designer to suit the building layout. A basement floor level waterproofing system should be selected based on the planned occupancy of the space and its sensitivity to moisture.

Exterior Slabs: For exterior slabs-on-grade subjected to pedestrian traffic only (i.e. sidewalks or walkways), a minimum four-inch thick nominally reinforced concrete slab on prepared subgrade should be adequate, where moisture control is not required.

Site Preparation

The site areas affected by new improvements should be cleared of all obstructions, including pavement, base rock, demolition debris, trees, tree stumps and major roots, abandoned utilities, old footings and/or foundation members, and deleterious materials. Holes resulting from the removal of old footings and foundation members, underground structures, or improvements that extend below the existing grade should be cleared thoroughly and then backfilled with suitable material compacted to the requirements described in “Engineered Fill and Backfill Placement.”

Clearing should typically extend at least five feet beyond the footprint of new structures. Concrete, bricks, wood, and other debris should be hauled off the site. Soils exposed after clearing and stripping should be reviewed by the Geotechnical Engineer before subsequent construction is performed. Unless stripped materials are considered suitable for landscaping purposes or other re-use on site, they should be hauled off the site and disposed of properly.

If an existing below-grade structural element such as a utility structure is encountered within the footprint of proposed construction, it should be removed to at least three feet below the subgrade for new footings, concrete slabs and other flatwork, and the pit should be properly backfilled with site-derived or imported materials in accordance with “Fill and Backfill Materials” and “Engineered Fill and Backfill Placement.”

In the areas of new improvements, unpaved portions of the site should be stripped to the depth necessary to remove organic materials, debris and any other unsuitable materials. The stripping depth may be in the range of 6 to 9 inches below existing grade, or less. Concrete, wood, and other debris should be hauled off the site. In the existing paved areas, the asphalt and subgrade should be stripped to expose clean native soil or fill.

Excavation and Slopes

General: Conventional excavation and earthwork equipment should be satisfactory for mass grading, foundation and basement excavations, and utility trenching on this site.

Sloped Excavations: During the excavation operations, temporary cut slopes should be used, where feasible, to prevent movement of materials exposed on the excavation walls. A temporary slope gradient of 1:1 (horizontal: vertical) or flatter should be used. The Lambert Shale formation bedrock is friable and therefore potentially susceptible to erosion, slaking, and raveling if exposed to wetting and drying. Exposure of temporary slopes to the elements should be minimized as much as possible.

Permanent cut and fill slopes should have a gradient of 2:1 (horizontal: vertical) or flatter, in order to ensure stability, encourage plant growth, and minimize erosion. A steeper gradient (1.5:1) could be considered for cuts in the Lambert Shale formation, with the understanding that there might be increased periodic maintenance costs for using a gradient that is steeper than 2:1 (horizontal: vertical) for a permanent cut slope.

To provide erosion protection, permanent slopes should be initially stabilized with straw plugs and then planted with plants, grasses, and shrubs consistent with the approved landscaping plan.

The Contractor should be aware that slope height, slope inclination, and excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, or federal safety regulations, e.g. OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations.

Subgrade Preparation

After the site has been cleared and stripped of unsuitable materials and graded/excavated to the required subgrade elevation, the exposed surface should be reviewed by the Geotechnical Engineer to determine if zones of potentially expansive clay soils are present in the subgrade surface. If potentially expansive clays are exposed, they should be removed ("over-excavated") to a depth of at least 12 inches below the slab subgrade elevation and be replaced with non-expansive engineered fill; see "Engineered Fill and Backfill Placement," below.

The subgrade under slabs-on-grade, exterior flatwork, paving, or sitework should be scarified to a depth of six inches, moisture-conditioned to a moisture content of approximately two percent over optimum, and compacted to at least 95 percent relative compaction (based on ASTM Test Method D1557). Any loose site soils encountered that cannot be compacted to 95% should be removed ("over-excavated") to a depth of at least 24 inches below the subgrade surface, or as directed by the Geotechnical Engineer, and replaced as engineered fill.

Any exposed subgrade that will receive fill should be prepared by scarifying to a depth of six inches and moisture-conditioning. The moisture-conditioned material should then be compacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). Moisture conditions in the subgrade should be maintained until fill is placed.

Engineered Fill and Backfill Placement

In areas designated to receive fill, the subgrade-to-receive-fill should be prepared as described in the preceding section. Approved fill material should then be placed in lifts not exceeding eight inches in un-compacted thickness, moisture-conditioned to near the optimum moisture content of the material, and compacted to at least 90 percent relative compaction (ASTM D1557).

In areas to be overlain by a slab-on-grade, exterior flatwork, paving or sitework, each lift of engineered fill should be compacted, at suitable moisture content, to a minimum relative compaction of 95 percent in the uppermost six inches of all fill and backfill, and a minimum 90 percent at other depths.

In addition to being compacted to the required relative compaction, the engineered fill should be stable, i.e., not exhibit "pumping" behavior. Ponding or jetting should not be used to densify fill or backfill.

Fill and Backfill Materials

Material used for fill and backfill, whether derived from the site or imported from off-site, must be granular soil, free of organic matter, which does not exhibit excessive shrinkage or swelling behavior when subjected to changes in water content. Most native site soils and existing fill materials are expected to suitable for re-use as fill, with the exception of minor localized zones of potentially expansive clays.

If imported fill material is required, it should contain no environmental contaminants or construction debris, and should conform to the following:

1. Satisfy the following gradation requirements:

U.S. Sieve Size	Percentage Passing (Dry Unit Composition)
2 ½-inch	100
No. 8	25-45
No. 200	0-10

2. Be thoroughly compactable without excessive voids.

3. Meet the following plasticity requirements:
    a. Maximum Plasticity Index of 12 (ASTM D4318).
    b. Maximum Liquid Limit of 35 (ASTM D4318).

Paving

Asphalt Concrete Pavement: We anticipate that asphalt concrete pavement would be constructed in parking and roadway areas. The paved areas could potentially be subjected to traffic loads ranging from "infrequent traffic from relatively light loads" to "frequent relatively heavy loads". To account for this range of traffic loads, we are providing design pavement sections for Traffic Indices (TIs) of 5.0, 6.0, and 7.0.

For areas with infrequent traffic from relatively light loads, we recommend using a TI of 5.0. Such areas could include parking spaces and aisles. For areas with more frequent traffic that are subjected to relatively light loads, such as roadways with normal vehicle traffic, we recommend using a TI of 6.0. Furthermore, for any areas subjected to heavy vehicle loads, such as fire trucks, we recommend using a TI of 7.0.

Our pavement design recommendations are summarized below.

Table 4
Recommended Asphalt Concrete Pavement Design Sections

Vehicular Traffic Area	Assumed Traffic Index (TI)	Thickness of Asphalt Concrete (in.)	Thickness of Caltrans Class 2 Aggregate Base (in.)
Infrequent Traffic from Light Loads (see note)	5.0	2	8
		3	6
Frequent Traffic from Light Loads	6.0	3	9
Heavily Loaded Areas	7.0	3	12

Note: For infrequent traffic from light loads (TI = 5.0), two alternative design sections are presented in the table. The first alternative is based on a minimal thickness of asphalt, while the second is based on an increased asphalt thickness and correspondingly reduced base thickness. Although both sections are structurally comparable, the section with thicker asphalt is expected to offer better wearing surface performance, especially where vehicles are frequently moving and turning; it is recommended for areas subjected to such use or where wear and appearance are of particular concern.

These pavement sections are based on the California State Flexible Paving Design Method, using the assumed TI values. Selection of these design traffic parameters were based on assumed use and not on a detailed equivalent wheel load analysis or traffic study. Furthermore, our recommended pavement design sections were based on a minimum R-value of 30, which is based on a laboratory test of site soils (Boring RC-5). The Cleary (1996) investigation included one R-Value test result of 45 and based its pavement section recommendations on a reduced value of 35, which is slightly less conservative than the our sections recommended above.

It should be noted that the pavement sections described above were not designed to accommodate construction traffic. The Contractor should be aware of this and should sequence the construction in such a way that new pavement sections are not subjected to construction traffic.

Concrete Pavement: For concrete paving subjected to traffic loads equivalent to a TI of 6.0 to 7.0, the pavement section should typically consist of 6 inches of appropriately reinforced concrete slab overlying 9 inches of aggregate base. Concrete paving or slabs subjected to heavy vehicular traffic, such as large fire trucks, should be designed on a special-case basis using an accepted rigid paving design methodology that takes into account parameters such as the expected wheel loads, frequency, and design life.

For slabs-on-grade subjected to pedestrian traffic only, a minimum four-inch thick nominally reinforced concrete slab on prepared subgrade should be adequate.

Unit Pavers: Where unit pavers are used, the paving system should be designed to support the weight of fully loaded fire vehicles wherever the area is accessible to such vehicles. Pavers in other areas should be designed for loadings appropriate for the usage. In all cases, the soil subgrade should be prepared, and the base and pavers should be installed, in accordance with the paving supplier's design recommendations.

Street Pavement: Where street paving is breached and needs to be replaced, the existing pavement section thickness should be restored if the performance/condition of the existing pavement is acceptable.

Pavement Subgrade Preparation and Drainage

Paving Subgrade: The subgrade for all paving types should consist of existing non-organic site soils (after stripping) scarified to a depth of six inches, moisture-conditioned, and re-compacted to a minimum 95 percent relative compaction (based on ASTM Test Method D1557).

Pavement Drainage: Our observations of pavement performance indicate that there is a strong correlation between poor pavement drainage conditions and the amount of pavement failures (potholes, settlement bowls, alligator cracks, etc.) observed. For this reason, we recommend that new pavement sections should be adequately drained by providing swales, culverts, or subdrains, as deemed necessary.

Aggregate Base Materials

Where aggregate base material is specified, the furnished material should meet the requirements of Class 2 Aggregate Base as described in the California Department of Transportation (Caltrans) Standard Specifications. Aggregate base materials should consist of virgin rock aggregates only, unless the Contractor can provide certification that any proposed recycled materials are free of hazardous and/or deleterious contaminants. The Contractor should provide written certification from the quarry stating that aggregate base materials meet all the requirements of Caltrans Class 2 Aggregate Base.

Controlled Low Strength Material (CLSM)

In cases where backfilling is required (e.g. at utility trenches), Controlled Low Strength Material (CLSM) can be used, if approved by the Geotechnical Engineer. Controlled Low Strength Material (also known as flowable compacting fill) should be a flowable and self-compacting mixture of Portland cement, fly ash, fine aggregates, water, and entrained air, conforming to ACI 229R. The mix shall have the following properties:

1. Minimum Compressive Strength: 25 psi at 1 day; 300 psi at 90 days. Strength shall not exceed 1,500 psi at 90 days for applications where future removal may be required (utility backfill, for example).

2. Slump: Six inches minimum to ten inches maximum, when tested in accordance with ASTM C143.

Corrosion Potential and Below-Grade Construction

Soils within the zone of influence of the project consist predominantly of soils which have a moderate to high corrosion potential. To mitigate the potential for corrosion effects, we recommend the following for below-grade concrete construction:

1. Allow for minimum 3-inch concrete cover over reinforcing steel for construction in contact with native soils.

2. Use dense concrete with the following characteristics:

    a. 4000 psi unconfined compression strength

    b. Type 2 Portland cement mixed thoroughly and integrally with 15-20 percent fly ash.

Subsurface utilities should be designed using materials and installation methods appropriate for an environment of moderate to high corrosion potential. A qualified corrosion engineer should be hired for detailed recommendations regarding corrosion protection of utilities.

Drain Rock and Filter Fabric

Drain rock, if required, should consist of Class 2 Permeable Material, meeting gradation and other requirements contained in the California Standard Specifications. Alternatively, three-quarter-inch crushed rock encapsulated in filter fabric (Mirafi 140N or equivalent) can be used instead of Class 2 Permeable Material. The Contractor should provide written certification to the Geotechnical Engineer stating that drain rock materials meet all the requirements of Caltrans Class 2 Permeable Material.

Surface Drainage and Erosion Control

Finished grading for surface drainage should be designed to direct surface runoff away from new structures toward discharge facilities. Ponding of surface water should not be allowed adjacent to structures. Downspouts and gutters should be provided, and water from downspouts should be directed through non-perforated pipes to storm drains. Alternatively, drainage culverts may be used to direct water from downspouts to storm drains.

Various best management practices for surface runoff, subsurface seepage, and erosion control can be employed either singularly or jointly to mitigate the potential for erosion. These include using curbs to keep runoff on the paved roadway; directing the runoff to strategically placed catch basins; providing swales at the toes of slopes to capture surface runoff; directing flow in swales to the storm drain system; and using erosion control matting and/or vegetation.

Utility Trench Backfilling

Site-Derived Backfill: Utility trench backfill generally consists of bedding, initial backfill, and final backfill. The bedding and initial backfill materials are selected based on the type of pipe in the trench. The Civil Engineer or other designers of utility installations should specify the type of bedding and initial backfill materials that are appropriate for the utility line in the trench. Site-derived soils from the trenches, except those containing organic materials, can be used as final backfill material. The Contractor should selectively stockpile site-derived soils that meet this general requirement.

Compaction Requirements: Approved initial and final backfill materials should be placed in lifts not exceeding eight inches in un-compacted thickness, moisture-conditioned to a moisture content of about two percent above the optimum moisture content of the material, and compacted to at least 90 percent relative compaction (ASTM D 1557). In areas where a trench is to be overlain by a pavement, the upper 6 inches of the backfill should be compacted to a minimum relative compaction of 95 percent.

Use of Controlled Density Fill (CDF) or Controlled Low Strength Material (CLSM): Conventional soil backfilling and compaction of trenches could be problematic for deep trenches required in some locations on the site, or under conditions of excessive soil moisture content. If acceptable to the designer from the performance point of view, in these conditions consideration should be given to fully or partially backfilling trenches with CDF or CLSM.

Moisture Flow Control Barriers: Utility trench backfill, even when properly compacted, can still serve as the path of least resistance for flow of moisture from storm water runoff or artificial sources. Moisture flow control barriers made up of low permeability clay soil or concrete should be installed at strategic locations to prevent moisture flow into utility structures or buildings.

Winter Construction

If earthwork operations are performed during the winter or the rainy season, the potential for erosion may increase and provisions would need to be made to minimize erosion.

Also, provisions should be made to dewater the excavations and to minimize the flow of surface runoff into the excavations if earthwork is performed during the rainy season.

We must note that the moisture content shown on the boring logs for the native soils reflects the moisture conditions at the time of the field exploration. The moisture content of those materials should be expected to be much higher if earthwork is performed during the winter or rainy season.

If earthwork operations are performed during the winter or the rainy season, long delays may result from the Contractor's inability to properly moisture-condition the mostly clayey, silty and sandy surface soils to achieve the required relative compaction. In that case, lime or cement treatment could be employed to make the site soils workable and compactable. Alternatively, geotextile fabric might be used to stabilize exposed wet subgrade in order to facilitate subsequent construction. Mirafi 500X or approved equal could be used in that case, but subgrade stabilization would require at least 12 inches of over-excavation before the placement of the fabric. Once the subgrade soils have been properly stabilized or compacted, a six-inch layer of Caltrans Class 2 Aggregate Base can be placed over the subgrade as a cap to maintain suitable working conditions, if necessary.

A gravel surface course may be required on construction traffic roads.

Impact of Site Conditions on Construction

Although this investigation was performed primarily for design purposes, a brief discussion of the impact of the site conditions on construction is presented for information purposes only. The discussion must not be considered a presentation of every possible impact of site conditions on construction.

Unanticipated Structures: Buried structures or concrete elements might be encountered. Efforts should be made to prevent contamination of site-derived fill materials by concrete and other debris.

Dust, Noise, and Vibration Control: Dust, noise and vibration control may be necessary to minimize the impact of construction activities on nearby buildings.

Rock: The term "rock" as used in this report encompasses materials ranging from moderately to very heavily weathered and fractured material. However, in compensation for drilling or excavation work on this site, no differentiation should be made between rock of various hardness.

Excavation: The rate of drilling through the rock encountered is one of many indicators of the ease with which the rock that will be removed. The drilling rates suggest that the bedrock formation could be excavated with slight to moderate effort using conventional construction equipment.

SECTION 4
FIELD EXPLORATION AND LABORATORY TESTING PROGRAMS

FIELD EXPLORATION PROGRAM

Scope

We conducted a subsurface exploration program on December 19, 2014. The purpose of the exploration was to provide geologic and geotechnical data for the project. The exploration program consisted of the following elements:

1. Obtaining San Mateo County permit for drilling, as notification to the County of San Mateo Environmental Health Department, under Annual Geotechnical Drilling Permit No. AGDP-14-1314.

2. Notifying USA North for subsurface utility marking (Ticket No. 512835) on December 8, 2014.

3. Performing geophysical survey by NORCAL Geophysical Consultants to locate existing leach field and check proposed boring locations for utilities, on December 9, 2014.

4. Mobilization of equipment by HEW Drilling on December 19, 2014.

5. Drilling, logging and sampling on December 19, 2014.

6. Grouting of holes and demobilization of equipment on December 19, 2014.

7. Selection of samples for subsequent geotechnical testing.

8. Analysis of laboratory test data and preparation of logs of borings.

Preparatory Activities

<u>Preparation</u>: Our staff marked proposed boring locations in the field using white paint. Borings are identified by the prefix "RC-", followed by a number. The approximate surface elevations of the exploratory holes are shown on the logs of borings.

<u>Coordination</u>: We coordinated with the on-site staff of Cal Fire regarding our drilling work and maintaining fire department operations without interruption or interference.

Field logistics were coordinated by our staff in conjunction with field geologist, Rick Ford, working as a subconsultant to Rutherford + Chekene. Cal Fire personnel visited the site briefly during the drilling operations.

Subsurface Exploration

Drilling: Drilling was performed by HEW Drilling Company of East Palo Alto. HEW deployed a truck-mounted CME 75 drilling rig fitted with 6-inch solid stem augers. Five exploratory borings were drilled to the depths shown in the following table:

Table 5
Exploratory Boring Depths

Boring	Approximate Ground Surface Elevation (feet)	Depth Below Existing Ground Surface (feet)
RC-1	-	26.5
RC-2	-	25.25
RC-3	-	25.4
RC-4	-	26.5
RC-5	-	11.5

The locations of the borings are shown on Figure 2 - Site and Boring Location Plan, in Appendix A.

Logging: The field geologist visually classified the soil using the Unified Soil Classification System (USCS) and the rock samples using the applicable classification system.

Our boring logs contain the information obtained in this exploration program. The boring logs show our interpretation of the subsurface conditions at the boring location on the date indicated, and it is not warranted that the logs are representative of subsurface conditions at other locations and times. The stratification lines shown represent the approximate boundaries between material types, and the transitions may be gradual. Also, we have developed soil and subsurface profiles by interpolation between the available data points, between which variations may occur in the actual conditions. Logs of the borings are included in Appendix B.

The locations of the borings were determined by measuring from physical features shown on the topographic survey, and surface elevations at the borings were obtained by interpolating between contours on the survey. The locations and elevations of the borings should be considered accurate only to the degree implied by the methods used.

Sampling: We obtained disturbed samples using a Standard Penetration Test (SPT) split-spoon sampler with equipment and procedures in accordance with ASTM Test Method D1586; liners were not used in the SPT sampler. We also obtained larger diameter, less disturbed samples in brass liners using a Modified California sampler with an outside diameter of about 2.5 inches and an inside diameter of 2.438 inches. The samplers were driven using a 140-pound automatic hammer falling and average of 30 inches. For each of the samples taken using either method, the number of blows required for every six-inch increment of penetration (or fraction thereof) was recorded. For each test, the total for the last 12 inches is the blow count. The blow counts on our boring logs represent the actual number of blows recorded during sampling; no conversions were made to the blow counts on the logs. For each sample obtained using an SPT sampler, the blow count is the Standard Penetration Test value, N. Using the method of Fang (1991), the actual blow counts of the Modified California sampler may be converted to approximately equivalent N values, by multiplying by 0.6.

At the completion of drilling, we retained representative samples for laboratory testing and future reference. Brass liner samples were capped and labeled. The SPT samples were placed in labeled plastic bags that were sealed.

Geophysical Survey

A geophysical survey was performed on December 9, 2014 by NORCAL Geophysical Consultants of Cotati, California. The purpose of the survey was to locate the existing leach lines associated with the site sanitary sewer system. The methods used and the survey findings are presented in NORCAL's report dated January 7, 2015, which is included as Appendix E.

LABORATORY TESTING PROGRAM

Engineering Properties

We commissioned Cooper Testing Laboratory (CTL) of Mountain View to perform laboratory testing aimed at evaluating index characteristics of selected soil samples from the borings.

Our program of index property testing consisted of tests on 23 samples to determine their moisture contents, according to ASTM D2216. We also had four samples taken with liner type samplers tested to determine their moisture contents and dry densities, in accordance with ASTM D2937; these four samples were also tested to determine their unconfined compressive strength using procedures in accordance with ASTM D2166. Sieve analyses were performed on four samples to determine their gradation characteristics in accordance with ASTM D422. Finally, four samples of clayey soils were tested to determine their Atterberg limits, according to ASTM D4318.

One soil sample taken from boring RC-5 was tested to determine the R-Value in accordance with Caltrans Test Method 301.

The results of the index property tests are presented on the boring logs at the appropriate sample depths. The laboratory test reports are presented in Appendix C.

Corrosivity Analyses

We commissioned CERCO Analytical of Pleasanton to perform corrosivity analyses of two soil samples taken from the exploratory borings (RC-2 at 5 feet and RC-4 at 5 feet). Tests were performed to measure the resistivity; chloride, sulfate and sulfide ion concentrations; pH; and redox potentials of the samples.

CERCO concluded, based on the resistivity measurements, that both samples are classified as moderately corrosive.

The chloride and sulfate ion concentrations in both samples were none detected, with a detection limit of 15mg/kg.

The pH of the samples ranged from 5.11 to 7.24. As noted by CERCO, any soils with a pH of less than 6.0 are considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials are both 350 mV and are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

CERCO's report is contained in Appendix D.

SECTION 5
REFERENCES

REFERENCES

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APPENDIX A
Figures for this Report

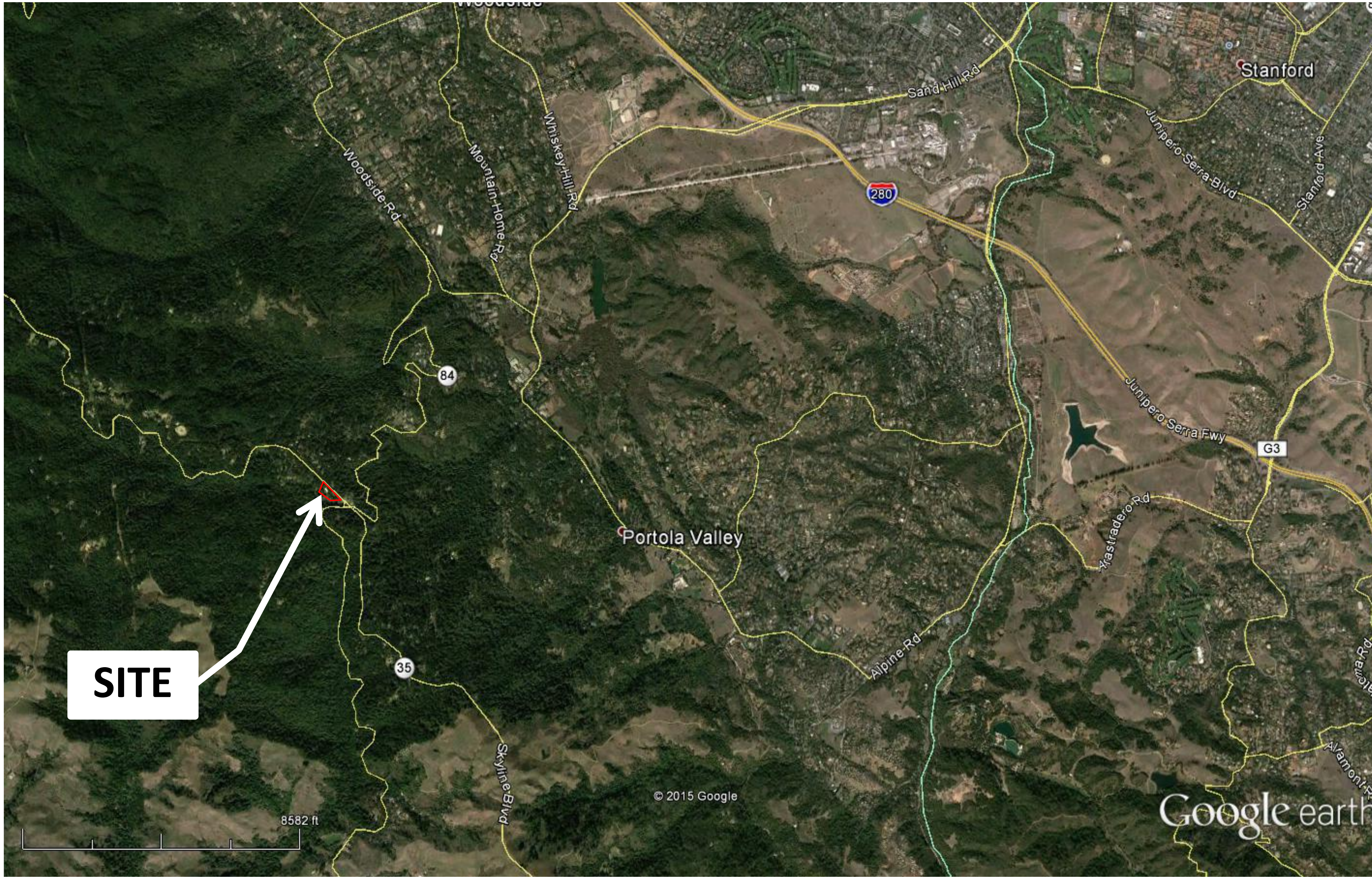

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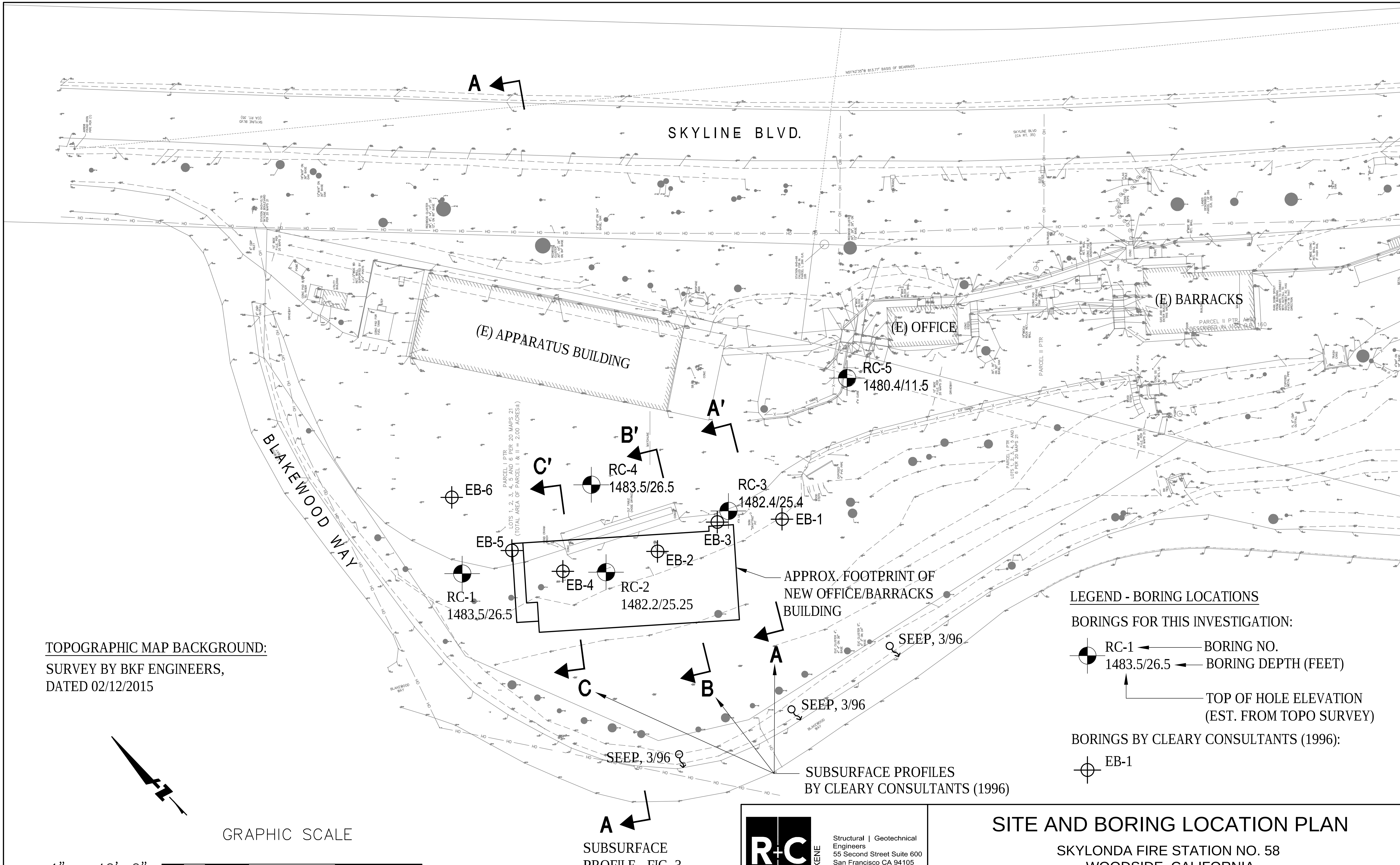
FIG. 1 - SITE VICINITY MAP

SKYLONDA FIRE STATION NO. 58

Woodside, San Mateo County, California

JOB NUMBER	DATE	SCALE	PAGE
2014-128G	4/10/15	As Shown	A1



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SITE AND BORING LOCATION PLAN

SKYLONDA FIRE STATION NO. 58
WOODSIDE, CALIFORNIA

SCALE:	1"=40'	DATE:	4/10/15	BY:	JB	R&C JOB No.:	2014-128G	FIGURE:	2

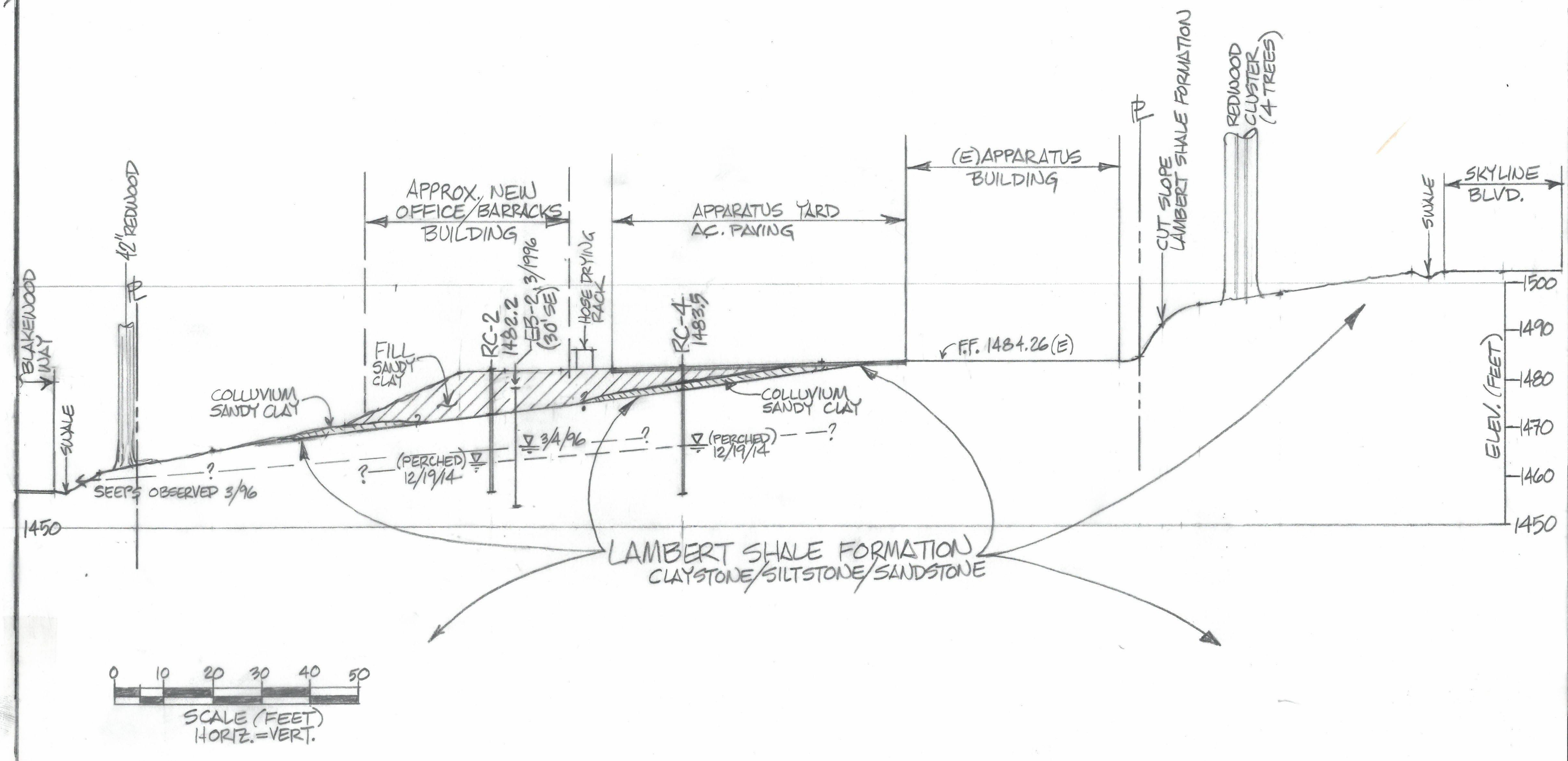

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FIG. 3 - SUBSURFACE PROFILE A-A

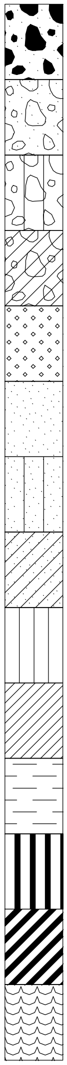
SKYLONDA FIRE STATION NO. 58

Woodside, San Mateo County, California

JOB NUMBER	DATE	SCALE	PAGE
2014-128G	4/10/15	1" = 20'	A3

APPENDIX B
Exploratory Boring Logs

SOIL SYMBOLS AND DESCRIPTIONS



GROUP ABBREVIATION (U.S.C.S.)	SYMBOL	GROUP NAME
GW		WELL GRADED GRAVELS
GP		POORLY GRADED GRAVELS
GM		SILTY GRAVELS
GC		CLAYEY GRAVELS
SW		WELL GRADED SANDS
SP		POORLY GRADED SANDS
SM		SILTY SANDS
SC		CLAYEY SANDS
ML		LOW PLASTICITY SILT
CL		LOW PLASTICITY CLAY
OL		LOW PLASTICITY ORGANIC SILT AND CLAY
MH		HIGH PLASTICITY SILT
CH		HIGH PLASTICITY CLAY
OH		HIGH PLASTICITY ORGANIC SILT AND CLAY

SAMPLE TYPES

SYMBOL	SAMPLE METHOD OR TOOL
	STANDARD PENETRATION TEST
	MODIFIED CALIFORNIA (2.0" O.D.)
	MODIFIED CALIFORNIA (2.5" O.D., 1.92" I.D.)
	CORE
	BULK SAMPLE
	NO RECOVERY

STANDARD PENETRATION TEST (SPT) SAMPLES ARE TAKEN BY DRIVING A STANDARD 1.4" I.D. SPLIT-SPOON SAMPLER INTO THE GROUND WITH A 140- POUND WEIGHT (HAMMER) FALLING 30 INCHES, PER ASTM D1586.

WATER LEVEL SYMBOLS

▽ WATER LEVEL DURING DRILLING, WITH DATE

▼ WATER LEVEL AFTER DRILLING, WITH DATE

SOIL DESCRIPTION TERMINOLOGY

SOILS ARE IDENTIFIED AND CLASSIFIED IN THIS REPORT ACCORDING TO THE UNIFIED SOIL CLASSIFICATION SYSTEM WITH THE FOLLOWING MODIFIERS:

CONSISTENCY OF SOILS

SPT, N BLOW COUNT	RELATIVE DENSITY
< 4	VERY LOOSE
4 - 10	LOOSE
10 - 30	MED. DENSE
30 - 50	DENSE
> 50	VERY DENSE

SPT, N BLOW COUNT	CLAY CONSISTENCY	UNCONFINED COMPRESSION STRENGTH (PSF)
< 2	VERY SOFT	< 500
2 - 5	SOFT	500 - 1000
5 - 10	MED. STIFF	1000 - 2000
10 - 20	STIFF	2000 - 4000
20 - 30	VERY STIFF	4000 - 8000
> 30	HARD	> 8000

SOIL MOISTURE

DESCRIPTIVE TERM	DESCRIPTION
DRY	DRY OF STANDARD PROCTOR OPTIMUM
DAMP	SAND ONLY
MOIST	NEAR STANDARD PROCTOR OPTIMUM
WET	WET OF STANDARD PROCTOR OPTIMUM
SATURATED	FREE WATER IN SAMPLE

PARTICLE SIZES

COMPONENTS	SIEVE OR SIEVE NO.
BOULDERS	OVER 12 INCHES
COBBLES	3 TO 12 INCHES
GRAVEL- COARSE	3/4 TO 3 INCHES
- FINE	NO. 4 TO 3/4 INCH
SAND - COARSE	NO. 10 TO NO. 4
- MEDIUM	NO. 40 TO NO. 10
- FINE	NO. 200 TO NO. 40
FINES (SILT AND CLAY)	BELOW NO. 200

NOTE:

1) THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS SHOWN, AND ARE NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THE OTHER LOCATIONS AND TIMES.

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KEY TO EXPLORATORY BORING LOGS

Skylonda Fire Station No. 58
San Mateo County Department of Public Works
Woodside, California

JOB No.: 2014-128G	Date: 4/10/2015	FIGURE: B1	PAGE: B1

EXPLORATORY BORING LOG

Ground Surface Elevation and Datum	Drilling Company		Notes	Boring Number
1483.5 feet,	HEW Drilling			RC-1
Groundwater Depth and Time	**Drill Rig and Drilling Method**			**Page**
	CME 75, Solid Stem Auger			1 of 1

Start Date	Finish Date	Driller Name	Drilling Fluid
12/19/2014	12/19/2014	Perfecto	None

Logged By	Borehole Diameter	Backfill Method	Hammer Type / Hammer Drop
Rick Ford	6 inches	Grout	140-lb Auto Hammer, 30"

Depth (feet)	Blows/6 inches or pressure	SOIL DESCRIPTION group name (symbol), color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	LABORATORY DATA: Moisture-Density: Moisture Content (%)	LABORATORY DATA: Moisture-Density: Dry Density (pcf)	LABORATORY DATA: Classification: Plasticity Index	LABORATORY DATA: Classification: Liquid Limit	LABORATORY DATA: Classification: % Fines (-#200)	OTHER DATA: Pocket Pen. (PP), Direct Shear (DS), Triaxial (Tx), Unconf. Compr.(UC)
0–5.5	3, 4, 4	SANDY CLAY (CH): Brown, gray, yellow-orange, etc., mottled, moist, medium stiff to stiff, medium plasticity. [Fill]	35					
5.5–8	3, 6, 7	SANDY CLAY (CL): Dark brown, slightly moist, stiff, fine sand. [Colluvium]	24					
8–15	13, 18, 25	CLAYSTONE: Light gray to pale grayish yellow with pervasive yellow-orange and trace black oxidation, very thin-bedded, low hardness, friable, deeply weathered to very stiff clay locally. [Lambert Shale]	35 34	80.7 87.6	37	70		UC = 2796 psf UC = 5539 psf
15–20	11, 12, 13	SILTSTONE: Pervasive yellow orange oxidation	36					
20–25	9, 7, 9	Pale yellow gray with yellow orange oxidation	41					
25–26.5	9, 15, 20	Dark gray brown to red brown oxidation mottling	39					
26.5		Boring terminated at 26.5' bgs No groundwater encountered						

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EXPLORATORY BORING LOG RC-1

Skylonda Fire Station No. 58

Woodside, California

JOB NUMBER	DATE	FIGURE	PAGE
2014-128G	4/10/2015	2	B 2

EXPLORATORY BORING LOG

Ground Surface Elevation and Datum 1482.2 feet,	**Drilling Company** HEW Drilling	**Notes**	**Boring Number** **RC-2**
Groundwater Depth and Time 19 feet, 9:15 am	**Drill Rig and Drilling Method** CME 75, Solid Stem Auger		**Page** 1 of 1
Start Date 12/19/2014 — **Finish Date** 12/19/2014	**Driller Name** Perfecto — **Drilling Fluid** None		
Logged By Rick Ford	**Borehole Diameter** 6 inches — **Backfill Method** Grout	**Hammer Type / Hammer Drop** 140-lb Auto Hammer, 30"	

Depth (feet)	Blows/6 inches or pressure	SOIL DESCRIPTION group name (symbol), color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	LABORATORY DATA — Moisture-Density: Moisture Content (%)	Dry Density (pcf)	Classification: Plasticity Index	Liquid Limit	% Fines (-#200)	OTHER DATA: Pocket Pen. (PP), Direct Shear (DS), Triaxial (Tx), Unconf. Compr.(UC)
1		SANDY CLAY with GRAVEL (CL): Light brown and yellow mottled, moist, soft to medium stiff, fine sandstone gravel, medium plasticity. [Fill]						
2–3	2 3 4		26				72	Push first 0.9'
5–6	3 3 4	Medium stiff; yellow orange oxidation on gravel fragments						Corrosion Test
9		Harder drilling SANDSTONE: Pale yellow-brown to yellow with yellow-orange and yellow-red oxidation, very thin-bedded, low hardness, friable, fine grained. [Lambert Shale]						
10–11	8 10 19		25					
15–16	11 13 17		15					
19		∇ Perched groundwater at 19' (approx. elev. 1463.2)						
20–21	14 32 50/5"		31					
25	50/3"	Light gray with yellow-red oxidation	19					
26		Boring terminated at 25.25' bgs						

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EXPLORATORY BORING LOG

Ground Surface Elevation and Datum	Drilling Company	Notes	Boring Number
1482.4 feet,	HEW Drilling		RC-3
Groundwater Depth and Time	**Drill Rig and Drilling Method**		**Page**
	CME 75, Solid Stem Auger		1 of 1

Start Date	Finish Date	Driller Name	Drilling Fluid
12/19/2014	12/19/2014	Perfecto	None

Logged By	Borehole Diameter	Backfill Method	Hammer Type / Hammer Drop
Rick Ford	6 inches	Grout	140-lb Auto Hammer, 30"

Depth (feet)	Blows/6 inches or pressure	SOIL DESCRIPTION group name (symbol), color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	LABORATORY DATA: Moisture-Density: Moisture Content (%)	Dry Density (pcf)	Classification: Plasticity Index	Liquid Limit	% Fines (-#200)	OTHER DATA: Pocket Pen. (PP), Direct Shear (DS), Triaxial (Tx), Unconf. Compr.(UC)
0		SANDY CLAY with GRAVEL (CH): Brown, yellow, yellow-orange etc. mottled, moist, soft. [Fill]						
2–3	2 2 2		29		25	54		
5–6	2 2 2		33				46	
6		MUDSTONE (CLAYSTONE): Light brown, pale yellow-gray, very thin-bedded, low hardness, friable, deeply weathered. [Lambert Shale]						
8.5–10	7 7 10		32 31	84.5 88.8	20	46		UC = 903 psf UC = 2026 psf
15–16.5	7 7 12	SILTSTONE: Pale yellow gray, yellow orange oxidation, very thin-bedded, low hardness, friable, moderately weathered. [Lambert Shale]	31					
20–21.5	22 30 50/5"	Interbedded SANDSTONE: Pale yellow-gray with red-brown oxidation	29					
23.5		SANDSTONE: Dark gray, thin-bedded, very fine grained, low hardness, weak, moderately weathered. [Lambert Shale]						
25	50/5"		11					
25.4		Boring terminated at 25.4' bgs No groundwater encountered						

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EXPLORATORY BORING LOG RC-3
Skylonda Fire Station No. 58
Woodside, California

JOB NUMBER	DATE	FIGURE	PAGE
2014-128G	4/10/2015	4	B 4

EXPLORATORY BORING LOG

Ground Surface Elevation and Datum 1483.5 feet,	**Drilling Company** HEW Drilling		**Notes**	**Boring Number** **RC-4**
Groundwater Depth and Time 16.5 feet, 11:00 am	**Drill Rig and Drilling Method** CME 75, Solid Stem Auger			
Start Date 12/19/2014	**Finish Date** 12/19/2014	**Driller Name** Perfecto	**Drilling Fluid** None	**Page** 1 of 1
Logged By Rick Ford	**Borehole Diameter** 6 inches	**Backfill Method** Grout	**Hammer Type / Hammer Drop** 140-lb Auto Hammer, 30"	

Depth (feet)	Blows/6 inches or pressure	SOIL DESCRIPTION group name (symbol), color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	LABORATORY DATA: Moisture-Density: Moisture Content (%)	Dry Density (pcf)	Classification: Plasticity Index	Liquid Limit	% Fines (-#200)	OTHER DATA: Pocket Pen. (PP), Direct Shear (DS), Triaxial (Tx), Unconf. Compr.(UC)
0–3		AC Paving/Base over SANDY CLAY (CL): Dark brown, moist. [Fill]						
2–3.5	4, 3, 6		28				64	
3–4.2		SANDY CLAY (CL): Dark brown, moist, stiff. [Colluvium]						
4.2		SILTSTONE: Gray to light gray, yellow-orange oxidation mottling, very thin-bedded, low hardness, friable, deeply to moderately weathered. [Lambert Shale]						
5–6.5	7, 7, 10							Corrosion Test
10–11.5	7, 8, 8	SILTSTONE/CLAYSTONE: Pervasive yellow-orange oxidation	38					
15–16.5	6, 7, 9		31					
16.5		Perched groundwater at 16.5' bgs (approx. elev. 1467.0)						
20–21.5	10, 10, 11	Light brown with pink hue, red-brown oxidation	37					
25–26.5	5, 11, 24	Dark brown to dark red-brown oxidation pervasive, slightly harder but low hardness, friable. [Lambert Shale]	28					
26.5		Boring terminated at 26.5' bgs						

EXPLORATORY BORING LOG

Ground Surface Elevation and Datum	Drilling Company		Notes	Boring Number
1477.5 feet,	HEW Drilling			RC-5
Groundwater Depth and Time	**Drill Rig and Drilling Method**			
	CME 75, Solid Stem Auger			
Start Date 12/19/2014 — **Finish Date** 12/19/2014	**Driller Name** Perfecto	**Drilling Fluid** None		**Page** 1 of 1
Logged By Rick Ford	**Borehole Diameter** 6 inches	**Backfill Method** Grout	**Hammer Type / Hammer Drop** 140-lb Auto Hammer, 30"	

Depth (feet)	Sample Type/Interval	Blows/6 inches or pressure	Graphic Log	SOIL DESCRIPTION group name (symbol), color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	Moisture Content (%)	Dry Density (pcf)	Plasticity Index	Liquid Limit	% Fines (-#200)	Pocket Pen. (PP), Direct Shear (DS), Triaxial (Tx), Unconf. Compr.(UC)
0–1				SANDY CLAY (CL): Red-brown, moist, stiff, trace very fine gravel. [Colluvium]	24					R-Value = 30
2–3	■	4, 4, 6			29		13	37	84	
5–6	■	6, 8, 14								
5.5				SILTSTONE: Light red-brown (pink), yellow-orange oxidation, very thin-bedded, low hardness, friable to weak, deeply to moderately weathered. [Lambert Shale]	30					
10–11.5	■	16, 16, 19		Light gray, light yellow-orange mottled	34					
11.5				Boring terminated at 11.5' bgs No groundwater encountered						

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EXPLORATORY BORING LOG RC-5

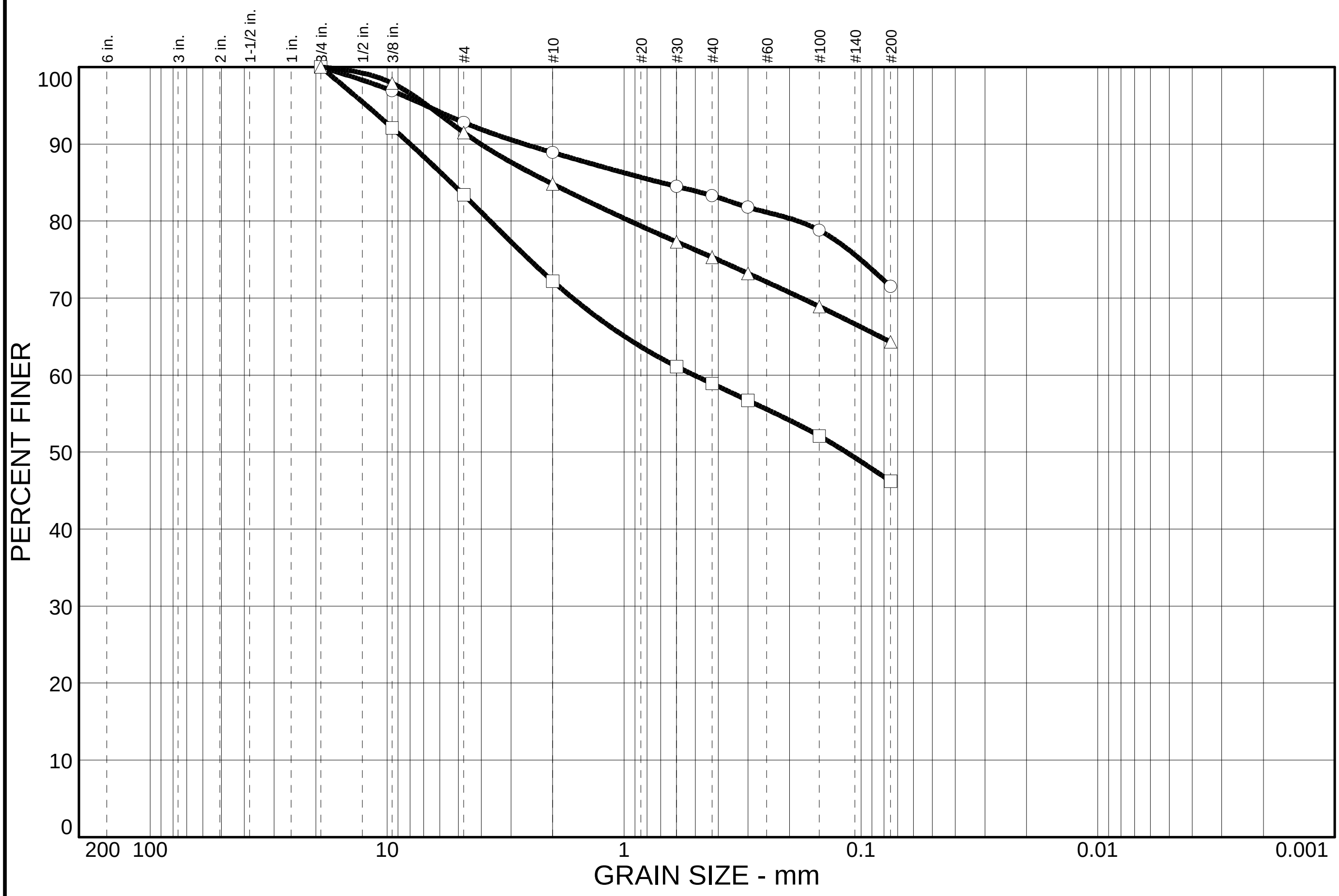
Skylonda Fire Station No. 58

Woodside, California

JOB NUMBER	DATE	FIGURE	PAGE
2014-128G	4/10/2015	6	B 6

APPENDIX C
Laboratory Test Reports
Cooper Testing Laboratories, Inc.

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		7.2	21.3	71.5					
□		16.6	37.2	46.2					
△		8.5	27.2	64.3					

SIEVE inches size	PERCENT FINER ○	□	△
3/4"	100.0	100.0	100.0
3/8"	96.9	92.1	97.9

	GRAIN SIZE		
D_{60}		0.507	
D_{30}			
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	92.8	83.4	91.5
#10	88.9	72.2	84.8
#30	84.5	61.1	77.3
#40	83.3	58.9	75.3
#50	81.8	56.7	73.2
#100	78.8	52.1	68.9
#200	71.5	46.2	64.3

SOIL DESCRIPTION

○ Yellowish Brown CLAY w/ Sand

□ Yellowish Brown Clayey SAND w/ Gravel

△ Dark Olive Brown Sandy CLAY

REMARKS:

○

□

△

○ Source: 2	Sample No.: 1	Elev./Depth: 1.5'
□ Source: 3	Sample No.: 2	Elev./Depth: 5'
△ Source: 4	Sample No.: 1	Elev./Depth: 2'

COOPER TESTING LABORATORY

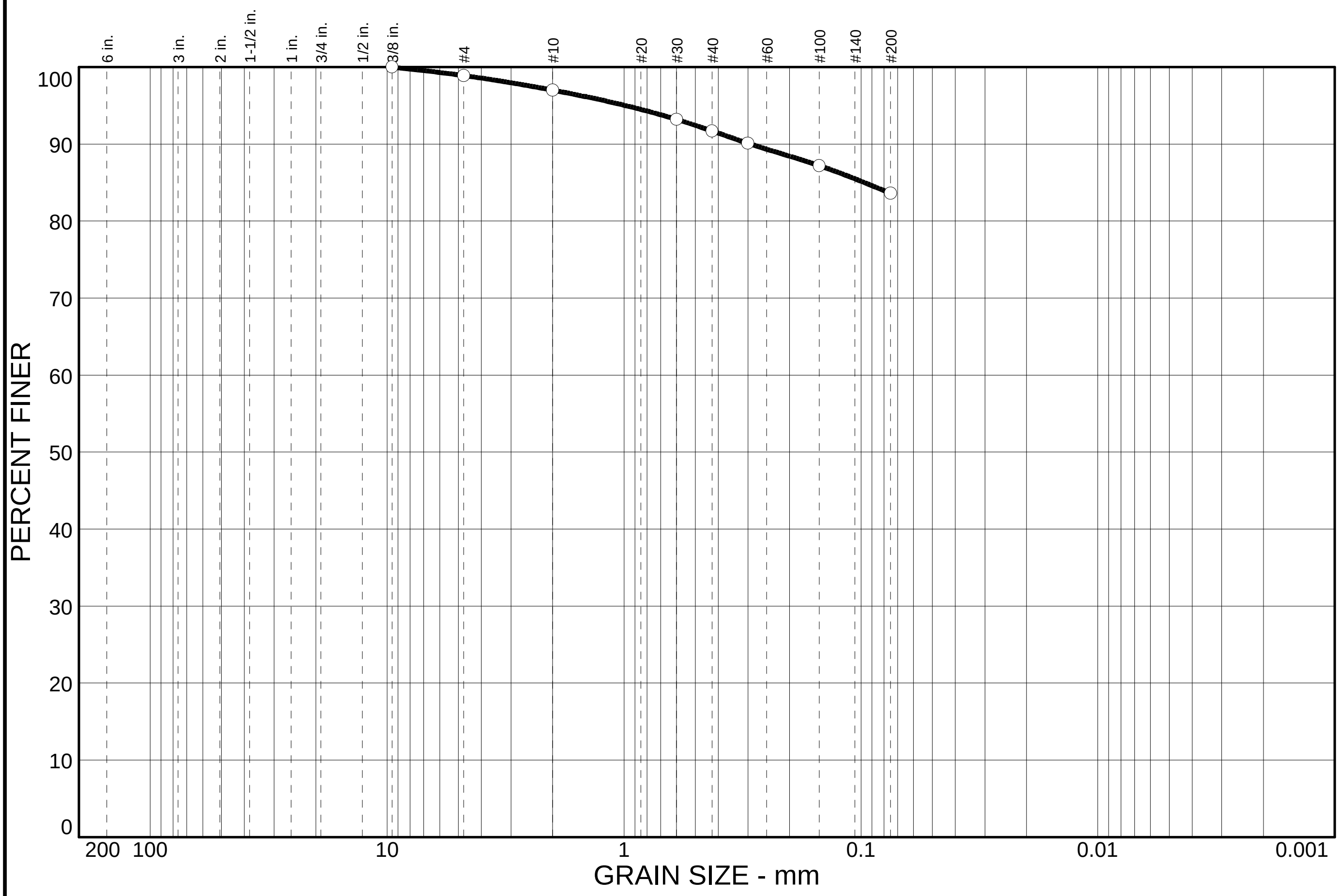
Client: Rutherford & Chekene

Project: Sky Londa Fire Station - 2014-128G

Project No.: 335-181

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		1.1	15.3	83.6		CL		24	37

SIEVE inches size	PERCENT FINER ○		
3/8"	100.0		

	GRAIN SIZE		
D_{60}			
D_{30}			
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○		
#4	98.9		
#10	97.0		
#30	93.2		
#40	91.7		
#50	90.1		
#100	87.2		
#200	83.6		

SOIL DESCRIPTION

○ Dark Yellowish Brown Lean CLAY w/ Sand & organics

REMARKS:

○

○ Source: 5 Sample No.: 1 Elev./Depth: 2'

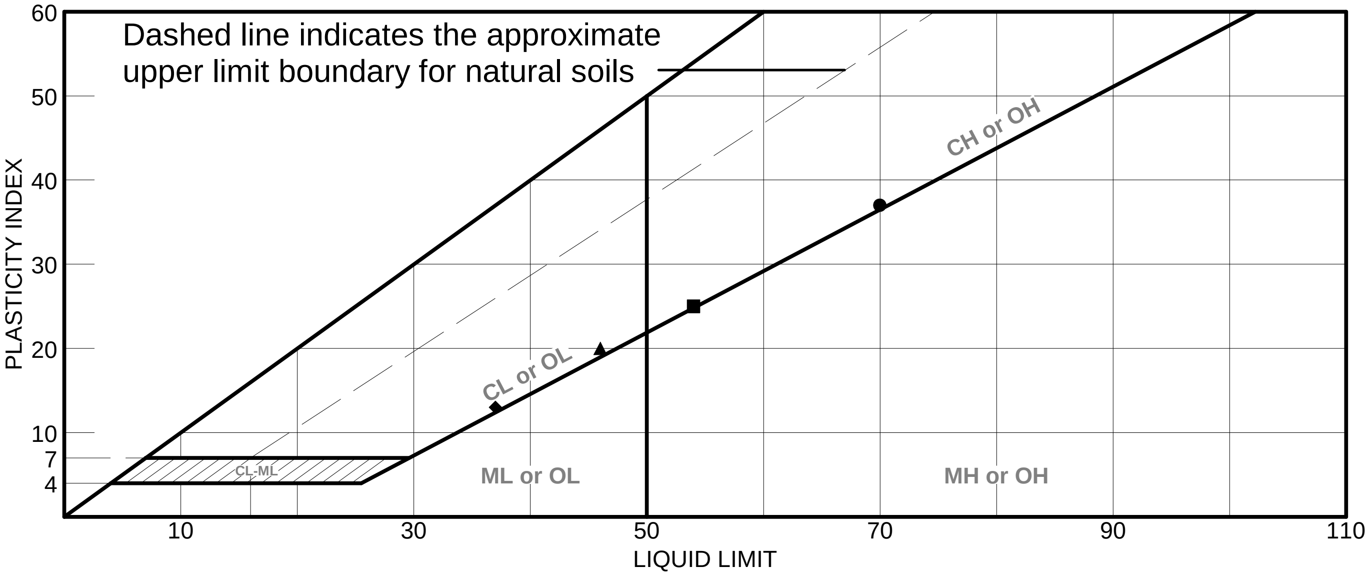
COOPER TESTING LABORATORY

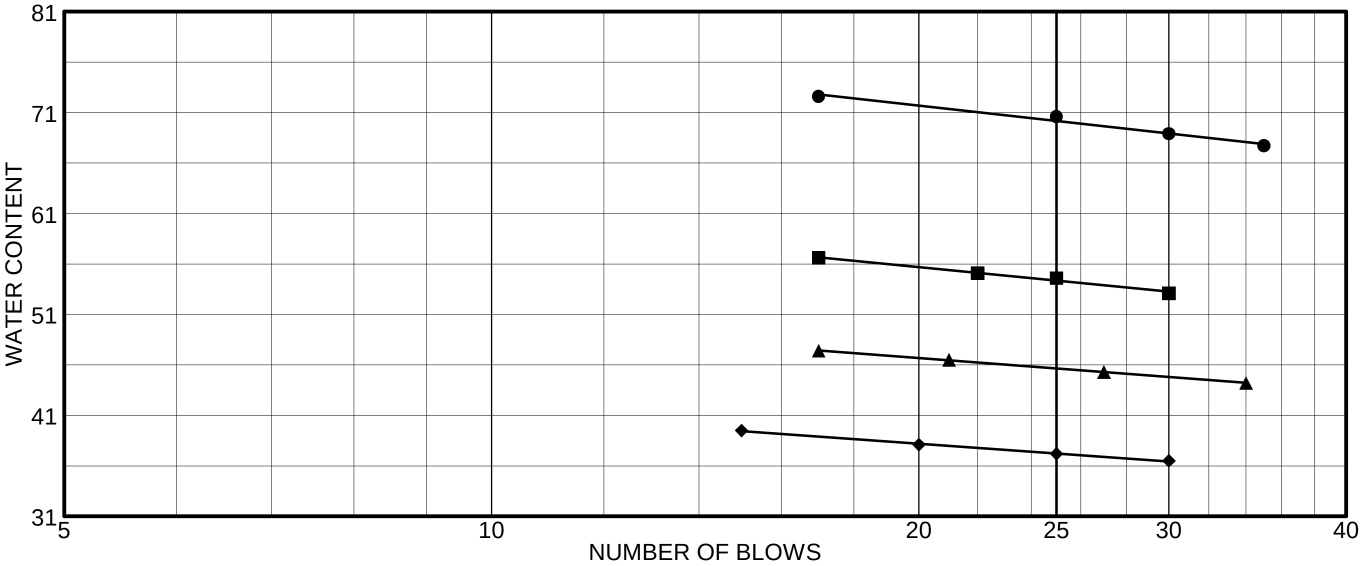
Client: Rutherford & Chekene

Project: Sky Londa Fire Station - 2014-128G

Project No.: 335-181 Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Yellowish Brown Sandy Fat CLAY	70	33	37			
■	Dark Olive Brown Fat CLAY w/ Gravel	54	29	25			
▲	Yellowish Brown Sandy Lean CLAY change to Olive Brown	46	26	20			
◆	Dark Yellowish Brown Lean CLAY w/ Sand & organics	37	24	13	91.7	83.6	CL

Project No. 335-181 **Client:** Rutherford & Chekene

Project: Sky Londa Fire Station - 2014-128G

● **Source:** 1 **Sample No.:** 3A **Elev./Depth:** 9'

■ **Source:** 3 **Sample No.:** 1 **Elev./Depth:** 2'

▲ **Source:** 3 **Sample No.:** 3A **Elev./Depth:** 9'

◆ **Source:** 5 **Sample No.:** 1 **Elev./Depth:** 2'

Remarks:

●

■

▲

◆

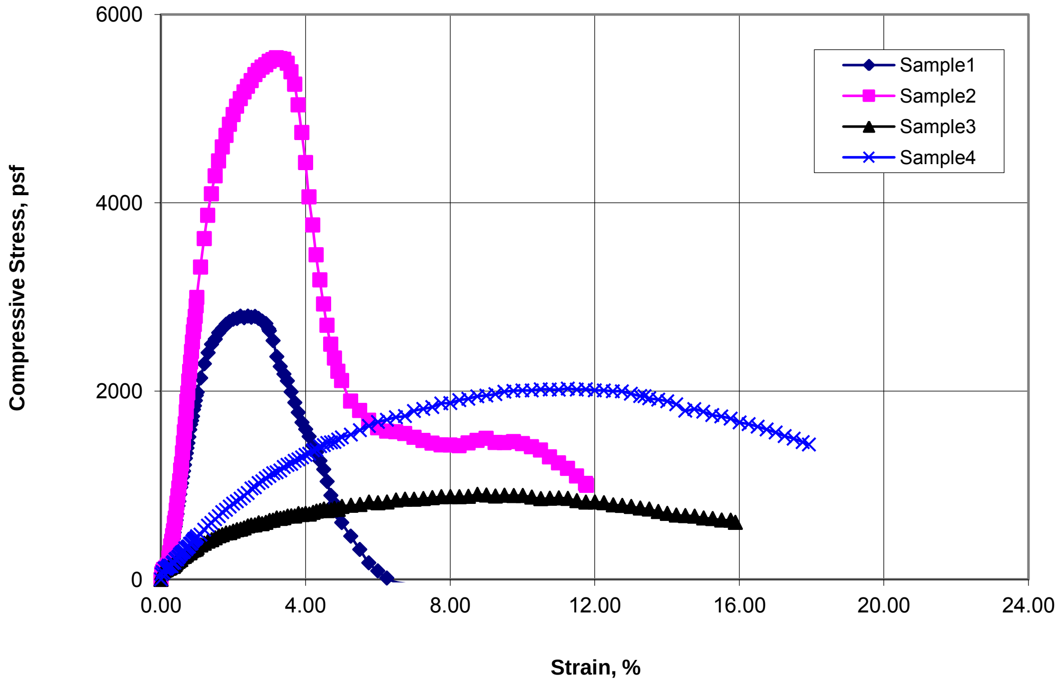
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Unconfined Compressive Strength

ASTM D2166



Sample No.:	1	2	3	4	
Unconfined Compressive Strength, psf	2796	5539	903	2026	
Unconfined Compressive Strength, psi	19.4	38.5	6.3	14.1	
Undrained Shear Strength, psf	1398	2770	452	1013	
Failure Strain, %	2.4	3.2	8.8	11.3	
Strain Rate, % per minute	1.0	1.0	1.0	1.0	
Strain Rate, inches/minute	0.05	0.05	0.05	0.05	
Moisture Content, %	34.7	34.2	31.5	31.1	
Dry Density, pcf	80.7	87.6	84.5	88.8	
Saturation, %	86.1	99.9	85.4	93.5	
Void Ratio	1.088	0.923	0.996	0.899	
Specimen Diameter, inches	2.400	2.398	2.399	2.406	
Specimen Height, inches	5.03	5.02	4.87	5.04	
Height to Diameter Ratio	2.1	2.1	2.0	2.1	
Assumed Specific Gravity	2.70	2.70	2.70	2.70	

Sample Location				
	Boring	Sample	Depth, ft.	Soil Description
1	1	3A	9	Dark Yellowish Brown Sandy Fat CLAY
2	1	3B	9.5	Dark Yellowish Brown CLAY w/ Sand
3	3	3A	9	Yellowish Brown Sandy Lean CLAY change to Olive Brown
4	3	3B	9.5	Dark Yellowish Brown CLAY

Job No.:	335-181	Type of Sample	Undisturbed
Client:	Rutherford & Chekene		
Project:	Sky Londa Fire Station - 2014-128G	Remarks:	
Date:	1/28/2015 By: MD/RU		

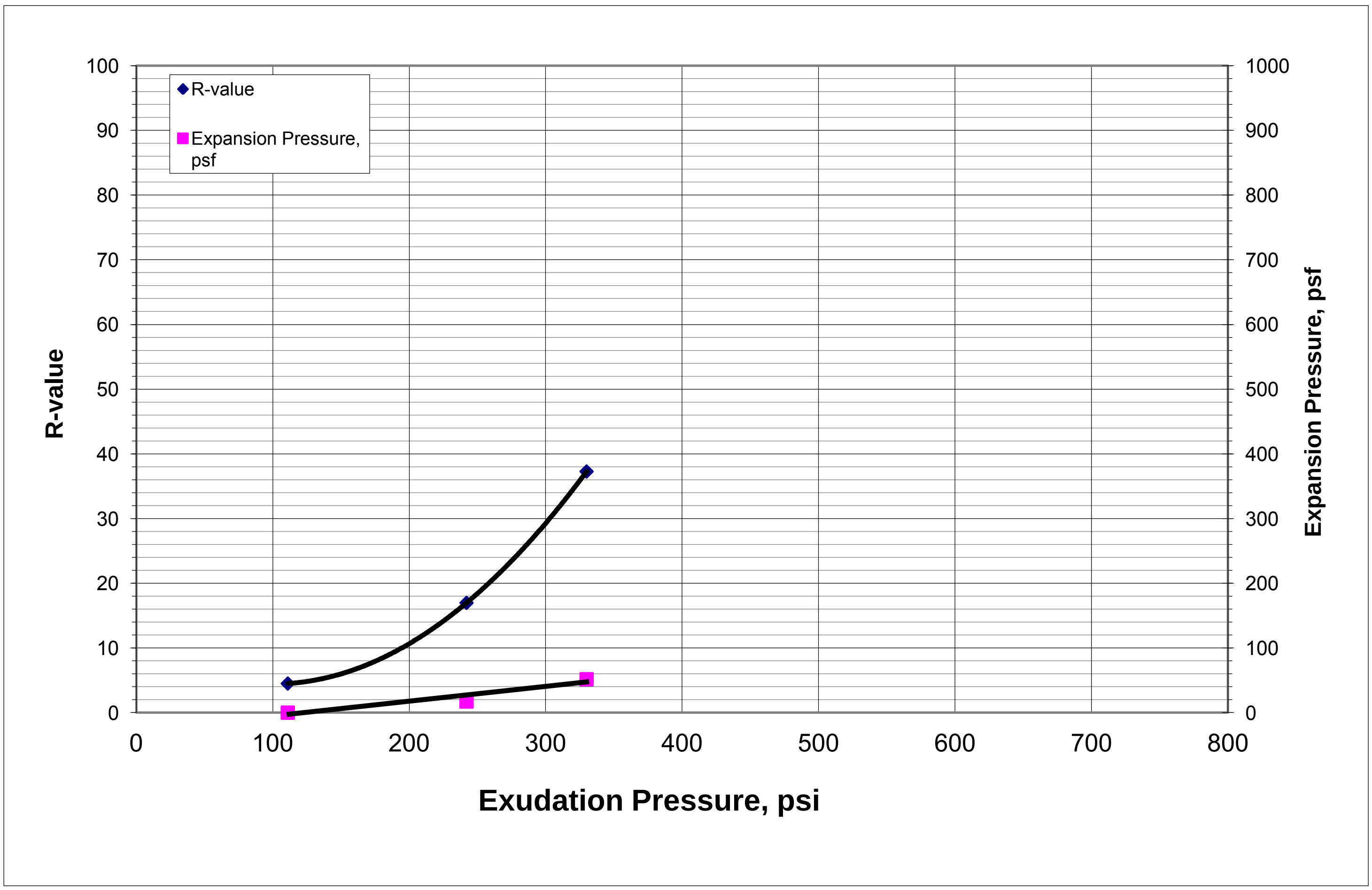
COOPER TESTING LABORATORY

R-value Test Report (Caltrans 301)

Job No.:	335-181	**Date:**	01/20/15	**Initial Moisture,**	24.0%
Client:	Rutherford & Chekene	**Tested**	MD	**R-value by Stabilometer**	**30**
Project:	Sky Londa Fire Station - 2014-125G	**Reduced**	RU		
Sample	5 Bag	**Checked**	DC	**Expansion Pressure**	**40** psf
Soil Type:	Dark Yellowish Brown Sandy CLAY				

Specimen Number	A	B	C	D
Exudation Pressure, psi	111	242	330	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	36	-29	-60	
Weight of Soil & Mold, grams	3110	3098	3111	
Weight of Mold, grams	2106	2116	2106	
Height After Compaction, in.	2.64	2.4	2.41	
Moisture Content, %	27.7	21.0	17.8	
Dry Density, pcf	90.2	102.4	107.2	
Expansion Pressure, psf	0.0	17.2	51.6	
Stabilometer @ 1000				
Stabilometer @ 2000	150	126	91	
Turns Displacement	3.7	3.1	2.9	
R-value	4	17	37	

Remarks:



APPENDIX D
Corrosivity Analysis
CERCO Analytical

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

29 January, 2015

Job No. 1501137
Cust. No.11288

Mr. John Burton
Rutherford & Chekene
55 Second Street, Suite 600
San Francisco, CA 94105

Subject: Project No.: 2014-128G
Project Name: Sky Londa Fire Station
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Burton:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on January 21, 2015. Based on the analytical results, a brief corrosivity 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 are none detected to 15 mg/kg.

The sulfate ion concentrations are none detected to 15 mg/kg.

The pH of the soils range from 5.11 to 7.24. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials are both 350-mV and are 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.

Cheryl McNeil for

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

JDH/jdl
Enclosure

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FEB 04 2015
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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: Rutherford & Chekene
Client's Project No.: 2014-128G
Client's Project Name: Sky Londa Fire Station
Date Sampled: 19-Dec-14
Date Received: 21-Jan-15
Matrix: Soil
Authorization: Transmittal dated 01/20/15

Date of Report: 29-Jan-2015

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)*
1501137-001	EB-2 #2 @ 5'-6'	350	7.24	-	6,400	-	N.D.	N.D.
1501137-002	EB-4 #2 @ 5'-6.5'	330	5.11	-	4,400	-	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Jan-2015	27-Jan-2015	-	26-Jan-2015	-	27-Jan-2015	27-Jan-2015

Cheryl McMillen
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 E
Geophysical Survey Report
NORCAL Geophysical Consultants, Inc.

January 7, 2015

Mr. John Burton
Rutherford + Chekene
55 Second Street, Suite 600
San Francisco, California 94105

Subject: Geophysical Survey
Skylonda Fire Station No. 58, Woodside

NORCAL Job Number 14-603.02

Dear Mr. Burton:

This letter presents the findings of a geophysical investigation performed by NORCAL Geophysical Consultants, Inc. on the subject property located in Woodside, CA. The field survey was conducted on December 9th, 2014 by NORCAL California Professional Geophysicist David T. Hagin PGp 1033 and Staff Geophysicist Hunter S. Philson. Logistical support was provided onsite by the fire station staff.

1.0 INTRODUCTION and PURPOSE

The fire station is scheduled for improvement, and prior to construction it is desired to know the locations of the leach lines associated with the site sanitary sewer system. The lines are within the asphalt covered area in front of the Apparatus Building, in the area indicated by the dashed green line on Plate 1. The survey area is generally open and flat with the metallic Apparatus Building bounding the area to the north and the top of slope forming the southern boundary. A metal rack, hose reel and fire hose bib are found near the top of the slope. The site was dry at the time of the survey.

The purpose of this survey was to obtain subsurface geophysical information within the designated survey limits to aid in identifying the locations of the leach lines. Additionally, we performed a utility location survey to complement our interpretation of the geophysical data.

2.0 FIELD INVESTIGATIONS

2.1 METHODOLOGY

It is anticipated that the leach lines are of non-metallic construction; however, when the leach line trenches were excavated and subsequently backfilled, the electrical properties of the soil may have been significantly altered. These variations may be detectable by certain geophysical methods. For this investigation we employed electromagnetic terrain conductivity (TC) and ground penetrating radar (GPR) methods. Additionally, we used the MD (metal detection) method to scan for near surface metal objects and the presence of utilities. Descriptions of the TC, GPR, and MD methods are provided in Appendix A.

2.2 DATA ACQUISITION

In order to provide horizontal position control for the acquisition of data we set out a survey grid over the area of investigation. The grid established a rectangular coordinate system based on the orientation of the adjacent Apparatus Building. We marked out the grid using a fiberglass measuring tape and marking paint. The marking paint was used to mark the grid nodes every 10- by 10-ft on the ground. These grids were then used to guide the respective surveys.

For the geophysical surveys, we first performed a site scan using the MD and GPR equipment. Initially, the MD and GPR scanned along both south-north and west-east trending traverses spaced 5-ft apart. When a buried object or trench was detected, the equipment was then employed along additional traverses at various angles in order to better define the target. The location of any detected object was subsequently marked on the ground surface with spray paint.

We then conducted the TC survey over the established grid. These data were acquired at approximately 5-ft intervals (stations) along traverse lines spaced 5-ft apart, resulting in data acquisition density approximating a 5 X 5 ft grid. Following data acquisition, we transferred the data to a personal computer and converted them into a format for contouring. The contouring program (***SURFER Version 12.0 by Golden Software***) calculates an evenly spaced array of values (grid) based on the observed field data. Finally, these gridded values were used to produce a TC contour map. This map provides a general characterization of the lateral conductivity variations and can be used to assess the existence of backfilled areas, buried debris and other subsurface objects.

Following the geophysical investigation, we drafted a site diagram of the survey area using the established grid and a measuring wheel. This diagram was then used to create the AutoCAD generated site plan on Plate 1.

3.0 RESULTS AND INTERPRETATION

The results of all of the geophysical methods used are summarized on Plate 1. Three utility lines and five leach lines were detected. Electric, water and undifferentiated (unknown) utility lines were delineated with the MD. The locations of the septic tank leach lines were identified by detecting the associated backfilled trenching using the GPR method, as indicated by the dashed green lines. The actual lines are apparently non-metallic and beyond the depth of exploration of the GPR.

The thin black lines on Plate 1 represent the TC contours expressed in millisiemens per meter. Areas on the TC contour map with tightly spaced contours indicate large variations in the measured values. These large variations are expected when the instrument is close to a known source such as a metallic building or buried utility; however, when large variations are not attributable to any identifiable source they are considered anomalous.

The TC contours show the approximate locations of the electric and undifferentiated utility lines that were detected with the MD; however, they do not indicate the locations of the leach lines. This may be due to insufficient difference in the soil electrical properties or possibly the trenches are too narrow to provide detectable variations of the TC values. Tightly spaced contour lines are also apparent adjacent to the apparatus building and reinforced concrete slab to the south, as expected.

4.0 LIMITATIONS

In general, there are limitations unique to the geophysical methods used for this investigation. For example, subsurface objects may be buried deeper than the detection capabilities of the geophysical method. There may be a lack of contrast in physical properties between native soils and buried objects. Above or below ground cultural features, such as utilities, fences, and debris, may cause interference that limits or masks the detection of nearby buried objects. Since the accuracy of our findings is subject to these limitations, it should be noted it is possible that not all buried objects or features may be detected or characterized. Descriptions of the MD, TC, and GPR methods and limitations are presented in Appendix A.

5.0 STANDARD CARE AND WARRANTY

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the shallow subsurface. 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 standard of care 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 having the opportunity to provide our services to Rutherford + Chekene for this investigation.

Respectfully,

NORCAL Geophysical Consultants, Inc.

David Hagin

David T. Hagin
California Professional Geophysicist, PGp 1033

DTH/KGB/tt

Enclosure: Plate 1
Appendix A GEOPHYSICAL METHODOLOGY

Appendix A

GEOPHYSICAL METHODOLOGY

Appendix A

ELECTROMAGNETIC TERRAIN CONDUCTIVITY (TC)

Methodology

The electromagnetic method is used to measure variations in subsurface electrical conductivity that may be due to buried foreign objects or changes in subsurface materials. The electromagnetic system utilizes two coils separated by a specified distance. One of these coils transmits a time-varying electromagnetic signal (primary magnetic field) which induces current flow in the earth. This in turn creates a secondary magnetic field which is detected by the receiver coil. The secondary signal is complex and has both quadrature and in-phase components. The amplitude of the quadrature component is proportional to the electrical conductivity of the subsurface materials. The in-phase component is proportional to conductivity, but is also affected by electrical properties associated with metal objects. The instrument displays the quadrature component in units of milliSiemens/meter (mS/m). Since this measurement represents the conductivity of the volume of material sampled, rather than individual layers, it is an apparent value and is referred to as terrain conductivity.

Electromagnetic surveys are typically conducted using a Geonics EM31-DL ground conductivity meter connected to an Omnidata data recorder. The EM31 has a fixed coil separation of 12 feet, which results in a total depth of investigation of approximately 10 to 15 feet depending upon local site conditions. The data recorder automatically stores EM values as well as station locations and annotations regarding cultural features.

Data Analysis

Computer Processing

The TC data are down loaded to a lap-top computer and converted it into a format for contouring. The contouring program (SURFER Version 8.0 by Golden Software) calculates an evenly spaced array of values (grid) based on the observed field data. Finally, these gridded values are contoured to produce a TC contour map.

Contour Map Interpretation

The TC contour map shows the variations in the electromagnetic terrain conductivity values within the survey area. The contour map is characterized by a series of contour lines that represent specific values. Areas that lack contour lines, or where the contours are spaced far apart, indicate a minimal change or variation in the respective values. This is indicative of relatively uniform conditions. Areas where contours are closely spaced indicate variations that are not uniform and probably caused by local sources.

In areas where there are significant quantities of above or below ground metal objects, the measured values are relatively large. These areas are characterized by numerous closely spaced contours. If the source of the anomaly is linear (e.g. underground utilities, railroad spurs, culvert, etc.), then the contours tend to parallel the object, and are closely spaced in close proximity to the object. If the below ground source is localized (e.g. buried drum, isolated metal debris, etc.), then the contours tend to form circular or elliptical closures that enclose the object. The larger the object and the closer it is to the geophysical instrument, the more contours there are in a given area. Variations that cannot be attributed to known above and/or below ground objects (metal well casings, reinforced concrete surface drain, above ground 55 gallon drums, utilities, etc.) are caused by unknown buried objects and are considered anomalous.

Buried landfill material is often characterized by circular to elliptical contour closures. These closures can vary from large circular closures that cover broad areas, to clusters of small closures that occur in zones. If the composition of the landfill is generally homogenous and nonmetallic, the contours tend to form large closures representing low values. If the fill material consists of both nonmetallic and metallic debris that varies significantly throughout the landfill, the contours tend to occur as numerous small closures representing both high and low values.

Limitations

There are inherent limitations associated with TC techniques that may not allow for the detection of all subsurface features of interest. These limitations are related to the composition of the subsurface feature, its size and depth of burial, and its proximity to other above or below ground features. In general, as the distance between a subsurface object and the respective geophysical instrument increases, the intensity of the associated field decreases, thereby making detection more difficult. In addition, above and below ground objects, such as buildings, debris, utilities, above ground electric lines, etc., typically produce interference that may mask effects from nearby buried features (targets).

Apart from the physical limitations of the instruments and the unwanted effects from secondary objects, the ability to detect subsurface features is also dependent upon the density of data acquisition points. If the distance between data acquisition points is significantly larger than the size of the subsurface feature, then this object may not be detectable.

GROUND PENETRATING RADAR (GPR)

Methodology

Ground penetrating radar is a method that provides a continuous, high resolution cross-section depicting variations in the electrical properties of the shallow subsurface.

The method is particularly sensitive to variations in electrical conductivity and electrical permittivity (the ability of a material to hold a charge when an electrical field is applied).

The GPR system operates by radiating electromagnetic pulses into the ground from a transducer (antenna) as it is moved along a traverse. Since most earth materials are transparent to electromagnetic energy, the signal spreads downward into the subsurface. However, when the signal encounters a contrast in electrical permittivity, a portion of the electromagnetic energy is reflected back to the surface. When the signal encounters a metal object, all of the incident energy is reflected. The reflected signals are received by the same transducer and are printed in cross-section form on a graphical recorder. Changes in subsurface reflection character on the GPR records can provide information regarding the location of voids, USTs, sumps, buried debris, underground utilities, and variations in the shallow stratigraphy.

The depth of investigation is dependent upon antenna frequency and ground conductivity, as determined by soil conditions. Clayey soils are typically high in water content and relatively conductive, potentially limiting the depth of investigation. Locally, optimum conditions for GPR are dry, sandy soils, although the method has been quite successful when used on snow and ice.

The GPR system used was a Geophysical Survey Systems, Inc. SIR-3000 Subsurface Interface Radar equipped with a 500 megahertz (MHz) transducer. This transducer is near the center of the available frequency range and is used to provide high resolution at shallow depths.

Data Analysis

GPR records are examined to identify reflection patterns characteristic of voids, USTs, utilities, and other buried debris. Typically, USTs, conduits and pipes are manifested by broad localized hyperbolic (upside-down "U" shape) reflection patterns, whereas voids may be quite irregular in shape. The intensity of a reflection pattern is usually dependent upon the condition of the respective object or void, its burial depth, and the type of fill over the feature. Utilities and other buried debris are typically manifested by narrow localized hyperbolic reflections that vary in intensity.

Limitations

The ability to detect subsurface targets is dependent on site specific conditions. These conditions include depth of burial, the size or diameter of the target, the condition of the specific target in question, the type of backfill material associated with the target, and the surface conditions over the target (reinforced concrete, etc.). Under ideal conditions, the GPR can generally detect objects buried to approximately six feet. However, as the clay content in the subsurface increases, the GPR depth of detection decreases. Therefore, it is possible that on-site soil conditions and target features may limit the depth of detection to the upper one to two feet below ground surface.

ELECTROMAGNETIC LINE LOCATION / METAL DETECTION (EMLL / MD)

Methodology

Electromagnetic line location techniques are used to locate the magnetic field resulting from an electric current flowing on a line. These magnetic fields can arise from currents already on the line (passive) or currents applied to a line with a transmitter (active). The most common passive signals are generated by live electric lines and re-radiated radio signals. Active signals can be introduced by connecting the transmitter to the line at accessible locations or by induction.

The detection of underground utilities is affected by the composition and construction of the line in question. Utilities detectable with standard line location techniques include any continuously connected metal pipes, cables/wires or utilities with tracer wires. Unless the utilities carry a passive current, they must be exposed at the surface or in accessible utility vaults. These generally include water, electric, natural gas, telephone, and other conduits related to facility operations. Utilities that are not detectable using standard electromagnetic line location techniques include those made of non-electrically conductive materials such as PVC, fiberglass, vitrified clay, and pipes with insulated connections.

Buried objects can also be detected, without direct contact, by using the induction mode. This is used to detect buried near surface metal objects such as rebar, manhole covers, USTs, and various metallic debris. The induction mode is used by holding the transmitter-receiver unit above the ground and continuously scanning the surface. The unit utilizes two orthogonal coils that are separated by a specified distance. One of the coils transmits an electromagnetic signal (primary magnetic field) which in turn produces a secondary magnetic field about the subsurface metal object. Since the receiver coil is orthogonal to the transmitter coil, it is unaffected by the primary field. Therefore, the secondary magnetic fields produced by buried metal object will generate an audible response from the unit. The peak of this response indicates when the unit is directly over the metal object.

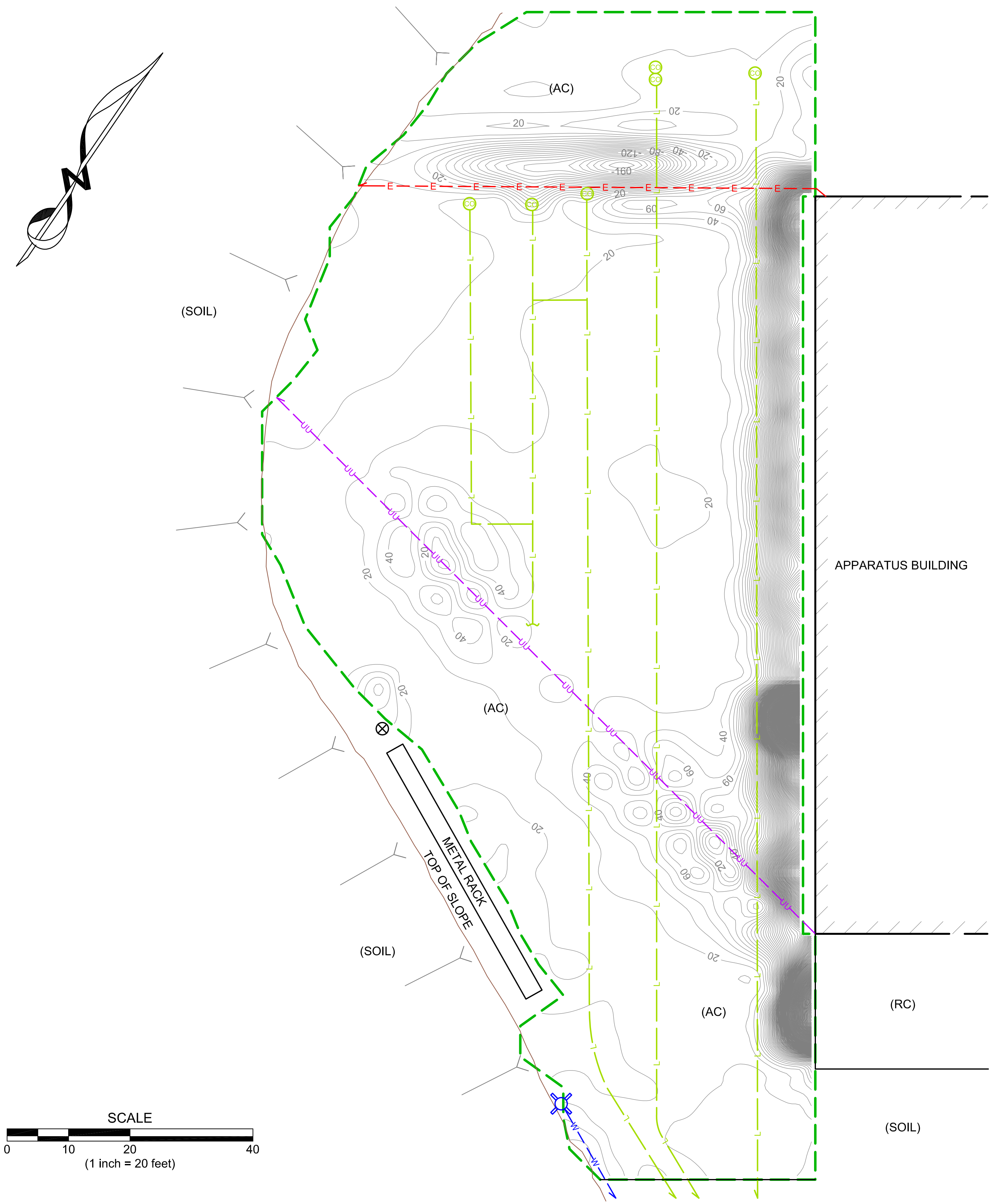
The instrumentation we used for the EMLL survey consists of a Radio Detection RD-400 and a Fisher TW-6 inductive pipe and cable locator.

Data Analysis

The EMLL instrumentation indicates the presence of buried metal by emitting an audible tone; there are no recorded data to analyze. Therefore, the locations of buried objects detected with the EMLL method are marked on the ground surface during the survey.

Limitations

The detection of underground utilities is dependent upon the composition and construction of the line of interest, as well as depth. Utilities detectable with standard line location techniques include any continuously connected metal pipes, cables/wires or utilities with tracer wires. Unless carrying a passive current these utilities must be exposed at the surface or accessible in utility vaults. These generally include water, electric, natural gas, telephone, and other conduits related to facility operations. Utilities that may not be detectable using standard electromagnetic line location techniques include certain abandoned utilities, utilities not exposed at the ground surface, or those made of non-electrically conductive materials such as PVC, fiberglass, vitrified clay, and metal pipes with insulating joints. Pipes generally deeper than about five to seven feet may not be detected.



VICINITY MAP

LEGEND

	LIMITS OF GEOPHYSICAL SURVEY
	TERRAIN CONDUCTIVITY CONTOUR (CONTOUR INTERVAL = 10 mS/m)
E	ELECTRIC LINE
L	SEPTIC TANK LEACH LINE
UU	UNDIFFERENTIATED UTILITY LINE
W	WATER LINE
	APPARENT UTILITY LINE TERMINATION (LINE BECOMES UNDETECTABLE AND IS SUSPECTED TO END)
	UTILITY LINE CONTINUATION (LINE IS SUSPECTED TO CONTINUE BEYOND DETECTED LOCATION)
	FIRE HOSE BIB
	HOSE REEL
	SEWER CLEANOUT
(AC)	ASPHALT
(RC)	REINFORCED CONCRETE

NORCAL

GEOPHYSICAL SURVEY MAP
SKY LONDA FIRE STATION NO. 58
17290 SKYLINE BOULEVARD

LOCATION: WOODSIDE, CALIFORNIA

CLIENT: RUTHERFORD + CHEKENE

JOB #: 14-603.02

NORCAL GEOPHYSICAL CONSULTANTS INC.

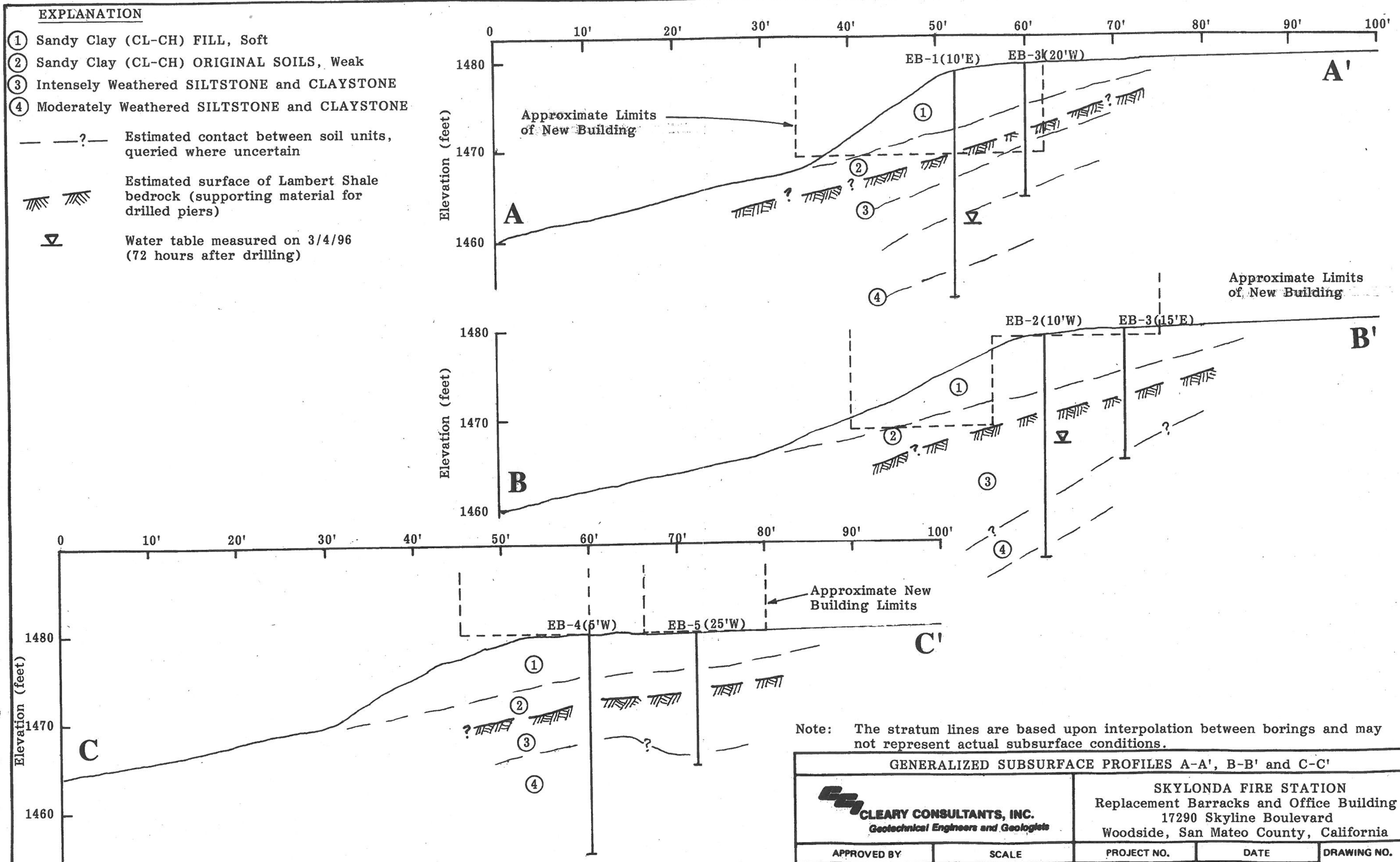
DATE: JAN. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DTH

PLATE 1

APPENDIX F
Excerpts from
Cleary Consultants Report, 1996



Note: The stratum lines are based upon interpolation between borings and may not represent actual subsurface conditions.

GENERALIZED SUBSURFACE PROFILES A-A', B-B' and C-C'

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

SKYLONDA FIRE STATION
Replacement Barracks and Office Building
17290 Skyline Boulevard
Woodside, San Mateo County, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 10'±	869.1	March 1996	4

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

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

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

GRAIN SIZES

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

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

†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.

SOIL STRENGTH

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

KEY TO BORING LOGS

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	5

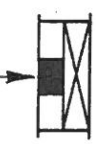
FIELD SAMPLING PROCEDURES

The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D-2487).

Representative soil samples were obtained from the borings at selected depths appropriate to the soil investigation. All samples were returned to our laboratory for classification and testing.

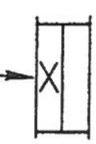
The penetration resistance blow counts were obtained by dropping a pound hammer through a 30-inch free fall. The 2-inch O.D. split spoon sampler was driven 18 inches or to practical refusal and the number of blows were recorded for each 6-inch penetration interval. The blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the penetration sampler the final 12 inches. In addition, 3.0 inch O.D. x 2.42 inch I.D. drive samples were obtained using a Modified California Sampler and the pound hammer. Blow counts for the Modified California Sampler are shown converted to equivalent split spoon blow counts by multiplying by 0.6. The sample type is shown on the boring logs in accordance with the designation below.

6" x 2.42" liner



Modified California Sampler

Bag Sample



Standard Split Spoon Sampler

Where obtained, the shear strength of the soil samples using either Torvane (TV) or Pocket Penetrometer (PP) devices is shown on the boring logs in the far right hand column.

SUMMARY OF FIELD SAMPLING PROCEDURES

CLEARY CONSULTANTS, INC.
Geological and Geotechnical Engineers

SKYLONDA FIRE STATION
Replacement Barracks and Office Building
17290 Skyline Boulevard
Woodside, San Mateo County, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC		869.1	March 1996	6

LABORATORY TESTING PROCEDURES

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 52 samples of the materials recovered from the borings in accordance with the ASTM D2216 Test Procedure. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry density determinations were performed on 15 samples to measure the unit weight of the subsurface soils in accordance with the ASTM D2937 Test Procedure. The results of these tests are shown on the boring logs at the appropriate sample depths.

Atterberg Limit determinations were performed on three samples of the subsurface soils in accordance with the ASTM D4318 Test Procedure to determine the range of water contents over which the materials exhibited plasticity. The Atterberg Limits are used to classify the soils in accordance with the Unified Soil Classification System and to evaluate the soil's expansion potential. The results of these tests are presented on Drawing 17 and on the boring logs at the appropriate sample depths.

Unconfined compression tests were performed in accordance with the ASTM D2166 Test Procedure on three undisturbed samples of the subsurface soils and rocks to evaluate the undrained shear strength of the material. The unconfined test was performed on a sample having a diameter of 2.43 inches and a height-to-diameter ratio of at least two. Failure was taken at the peak normal stress or at five percent strain, whichever occurred first. The results of these tests are presented on the boring logs at the appropriate sample depth.

The percent soil fraction passing the #200 sieve was determined on five samples of the subsurface soils in accordance with the ASTM D1140 Test Procedure to aid in the classification of the soils. The results of these tests are shown on the boring logs at the appropriate sample depths.

Free swell tests were performed on 10 samples of the soil materials to evaluate the swelling potential of the materials. The tests were performed by pouring ten grams of the dry material into a 100 mL graduated cylinder containing about 40 mL of distilled water. The mixture was stirred repeatedly and allowed to equilibrate for 24 hours, then distilled water was added up to the 100 mL mark. The graduated cylinder was stoppered and left undisturbed to equilibrate. The free-swell volume was then noted. The percent free swell was calculated by dividing the free-swell volume by ten and multiplying by 100 percent. The results of these tests are presented on the boring logs.

A resistance (R-Value) test was performed on a representative sample of the surface soils from Boring 6 to provide data for pavement design. the test was performed in accordance with ASTM D-2844 Test Procedure. The results of this test are presented on Drawing 18.

Drawing No.7

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1478.0±	LOGGED BY RS
DEPTH TO GROUNDWATER 17.0'	DEPTH TO BEDROCK 16.5'±	DATE DRILLED 3/1/96

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
weeds & roots in upper 1"									
SANDY CLAY, wet, clayey silt, sandy silt, silty sand and organics in varying proportions, occasional gravels to 1", fine to coarse sand, weak, silty organic @3.5-5.0'	Orange, Brown, Yellow and Black	Firm	CL-SC	1			21	90	
		Soft		2		6	21		
				3		4	35		
@1.5' : Finer than #200 = 34% Free Swell = -10%	Dark Gray-Brown			4			36	61	
wet silty clay with gravels, some organics @5.5' FILL				5		3	44		.36TV
				6		4	34		
SANDY CLAY, wet, fine to coarse sand, trace fine gravels, some silt, plastic	Light Brown with Orange to Yellow Particles	Stiff	CH	7					
@9.0' : Free Swell = 40%				8					
@9.5' : Liquid Limit = 57% Plasticity Index = 34% Free Swell = 15%				9			33	82	*1.41 ksf 5% strain
				10		8	34		1.5TV
				11					
SILTY CLAY, moist to wet, very plastic gray clay, mixed with slightly plastic orange-yellow silty clay, trace fine to medium sand, weak bedding	Mottled Orange, Yellow and Gray	Stiff	CH	12					
@14.0' : Free Swell = 50%				13					
(Completely weath. claystone)				14			31		
				15		15			
				16					
CLAYSTONE, moist, friable, thinly bedded, interbedded thin orange siltstone	Red-Brown	Hard	(CL-ML)	17			∇	measured 3 days after drilling (3/4/96)	
thinly bedded @9.5', 10° bedding, predominantly siltstone				18					
				19			21		
							23	100	
				20		47	19		2.8TV

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF BORING NO. 1

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	8

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1478.0±	LOGGED BY RS
DEPTH TO GROUNDWATER 17.0'	DEPTH TO BEDROCK 16.5±	DATE DRILLED 3/1/96

DESCRIPTION AND CLASSIFICATION				DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE						
CLAYSTONE, continued	Orange-Brown	Hard	(CL-CH)	21, 22					
SANDY SILTSTONE, moist, fine to medium sand	Orange-Brown and Yellow	Hard	(ML)	23					
poorly bedded, 10° maximum dip on irregular bedding planes @24.0'	Black-Gray			24	X	41	20		
Bottom of Boring = 25.0'				25					
Hole caved to 19.0' (3/4/96)				26					
* = Unconfined compression test				27					
				28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF BORING NO. 1

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	9

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1478.4±	LOGGED BY RS
DEPTH TO GROUNDWATER 12.0'	DEPTH TO BEDROCK 14.5'±	DATE DRILLED 3/1/96

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
weeds & roots in upper 1"								
SANDY CLAY, wet, with mixed sandy silt, clayey silt and silty sand, trace organics, occasional fine to medium gravels, siltstone fragments, fine to coarse sand	Mixed Brown, Orange and Yellow	Soft	CL-ML	1		26		
				2	4	35	73	
				3		34		
@2.5' : Liquid Limit = 46% Plasticity Index = 18% Finer than #200 = 68% Free Swell = 30%				3	3			
				4		22		
				5	3	33	73	
FILL				6		25		
				6	12			
SANDY SILT, moist, trace clay, very fine to occasionally coarse sand, horizontal siltstone bedding (Highly weathered siltstone)	Orange-Yellow	Stiff	(ML-SM)	7				
				9		20	94	*.88 ksf 2.5% stra 1.6TV
				10	10	33		
				12		▽ measured 3 day after drilling (3/4/96)		
grading stiffer, more rock-like @14.5'		Very Stiff		14.5				
				16		26		
				16	21			
increasing clay content with occasional clay layers @18.0'				18				
SILTY CLAY, moist to wet, trace fine sand, horizontal bedding	Red-Brown	Hard	CL	19		23		
				20	37	43		

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF BORING NO. 2

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	10

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1478.4±	LOGGED BY RS
DEPTH TO GROUNDWATER 12.0'	DEPTH TO BEDROCK 14.5'±	DATE DRILLED 3/1/96

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SILTY CLAY, continued	Red-Brown	Hard	CL	21 22 23					
CLAYEY SILTSTONE, slightly moist, thinly laminated, friable, very fine micaceous sand	Gray-Black	Hard	(CL-SC)	24	X	73/11"	17		
Bottom of Boring = 25.0' Hole caved to 17.0' * = Unconfined compression test				25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
Geological and Geotechnical Engineers

LOG OF BORING NO. 2

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	11

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1479.7±	LOGGED BY RS
DEPTH TO GROUNDWATER Not Enc.	DEPTH TO BEDROCK 9.0'±	DATE DRILLED 3/1/96

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SANDY CLAY, wet, with mixed sandy silt, clayey silt and silty sand, wood fragments, occasional fine to medium siltstone gravels, fine to coarse sand, weak @1.0' : Free Swell = 10% FILL	Brown-Orange-Yellow	Soft	CL	1–2		2	38 36	72	
				3		2	37		
SANDY SILT, moist, clay, occasional siltstone fragments, very fine sand, friable	Mottled Orange-Yellow and Gray-Brown	Soft to Firm	ML	5		3	36	75	
				6		4			
CLAYSTONE, moist, intensely weathered to silty clay, weakly cemented @9.5' : Finer than #200 = 99% Free Swell = 40%	Orange-Brown	Stiff	(CL-CH)	9–10		9	36 37		2.2TV
grading more silty, very fine sand @14.0'	Red-Brown		(CL-ML)	14–15		12	23		
Bottom of Boring = 15.0'									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF BORING NO. 3

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	12

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1479.2±	LOGGED BY RS
DEPTH TO GROUNDWATER Not Det.	DEPTH TO BEDROCK 12.0'±	DATE DRILLED 3/1/96

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
weeds & roots in upper 1" SANDY CLAY, moist to wet, with sandy silt, clayey silt and silty sand, minor organics and manmade debris, occasional fine to medium siltstone gravel fragments, fine to coarse sand, weak @1.0' : Liquid Limit = 55% Plasticity Index = 29% Finer than #200 = 72% Free Swell = 50% FILL	Brown-Orange-Yellow	Stiff Firm	CH	1 2 3 4		10 5 5	29 31 32 32 33	90 74	
SILTY CLAY, wet, fine to medium sand (original topsoil)	Dark Brown	Firm	CL	5 6 7		6	24		
SANDY CLAY, moist, occasional siltstone fragments, possibly intensely weathered claystone @9.0' : Free Swell = 40%	Mottled Brown-Orange-Yellow	Stiff to Very Stiff	CL	8 9 10 11		17	36	83	
SANDY SILTSTONE, moist, very fine uniform subrounded sand, friable, sugary texture, weak horizontal bedding, intensely weathered and soil-like	Yellow-Gray-Brown	Very Stiff Hard	(ML-SM)	12 13 14 15 16 17 18 19 20		15 34	20 15		

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

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Geotechnical Engineers and Geologists

LOG OF BORING NO. 4		
SKYLONDA FIRE STATION 17290 Skyline Boulevard Woodside, San Mateo County, California		
PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	13

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1479.2±	LOGGED BY RS
DEPTH TO GROUNDWATER Not Det.	DEPTH TO BEDROCK 12.0'±	DATE DRILLED 3/1/96

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SANDY SILTSTONE, continued increasing clay content and harder drilling @22.0' grading more silty @24.0'	Mottled Brown, Orange and Yellow	Hard	(ML)	21 22 23					
			(ML)	24	X	41	19		
Bottom of Boring = 25.0'				25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

LOG OF BORING NO. 4

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	14

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1480.2±	LOGGED BY RS
DEPTH TO GROUNDWATER Not Enc.	DEPTH TO BEDROCK 14.0'±	DATE DRILLED 3/1/96

DESCRIPTION AND CLASSIFICATION									
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
4" AC/6" Baserock									
SANDY CLAY, wet, with sandy silt, clayey silt and silty sand, trace organic and manmade debris, occasional gravels and silstone fragments, minor fine to coarse sand, weak	Brown, Orange and Yellow	Stiff	CL-ML	1			30		
		Firm		2		11	28	78	
				3			29		
moist @2.5', more silty						5			
FILL				4			30	66	
glass debris @4.9'									
SANDY CLAYEY SILT, moist, minor fine to medium sand, clayey, lenses, friable siltstone fragments	Dark Brown	Firm	CL-ML	5		5	28		
				6			33		
						6			
SANDY CLAY, moist, siltstone fragments, minor fine sand	Mottled Orange-Yellow and Brown	Firm	CL	7					
(Intensely weath. claystone)				8					
		Stiff		9			48	72	*2.29 ksf 1.7% str 1.7TV
				10		9	46		
				11					
				12					
@14.0' : Finer than #200 = 99% Free Swell = 40%				13					
rock-like structure @14.0'		Stiff to Very Stiff	(CL-CH)	14			37		
				15		16			
Bottom of Boring = 15.0'				16					
				17					
				18					
				19					
				20					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

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LOG OF BORING NO. 5

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	15

EQUIPMENT 8" Hollow Stem Auger	ELEVATION 1481.1±	LOGGED BY RS
DEPTH TO GROUNDWATER Not Enc.	DEPTH TO BEDROCK Not Enc.	DATE DRILLED 3/1/96

DESCRIPTION AND CLASSIFICATION									
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
3" AC/9" Baserock				1					
SANDY SILT, with sandy silt, clayey silt and silty sand, trace organics, occasional gravels and siltstone fragments, fine to coarse sand, wet	Mixed Brown, Orange and Yellow	Firm to Stiff	SM-ML	2	X		24		
				3					
				4					
Bottom of Boring = 4.5' *Bulk sample from 1' to 4' (R-Value Test)				5					
				6					
				7					
				8					
				9					
				10					
				11					
				12					
				13					
				14					
				15					
				16					
				17					
				18					
				19					
				20					

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.

CLEARY CONSULTANTS, INC.
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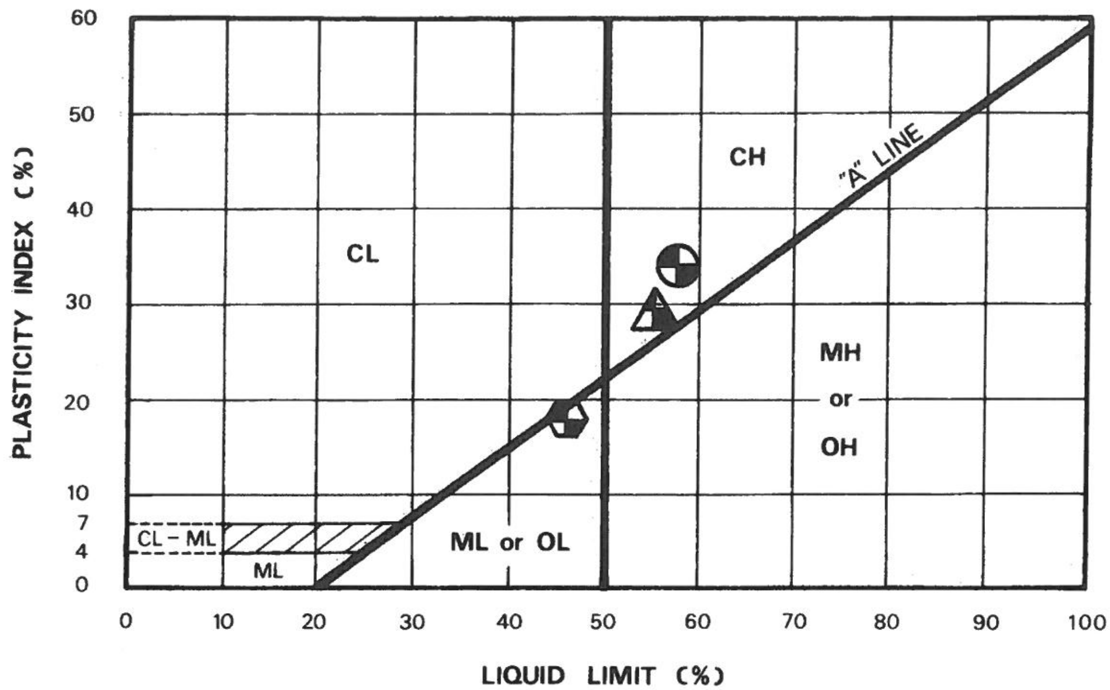
LOG OF BORING NO. 6

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	16



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
⊕	1	9.0	33	57	34	- - -	0.3	CH
⬡	2	2.5	34	46	18	68	0.3	ML-CL
△	4	1.0	29	55	29	72	0.1	CH

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PLASTICITY CHART

SKYLONDA FIRE STATION
17290 Skyline Boulevard
Woodside, San Mateo County, California

PROJECT NO.	DATE	DRAWING NO.
869.1	March 1996	17

RESULTS OF "R" VALUE TEST
(ASTM D-2844-69)

Sample No.	Description of Material	Water Content (%)	Dry Density (pcf)	Exudation Pressure (psi)	"R" Value	Expansion Pressure (psf)
Bulk	Brown	24.5	94.8	118	2.9	0
Sample	SANDY SILT	21.6	100.7	185	10.9	0
	with some	19.1	105.3	314	48.2	166
	gravel	18.2	106.8	474	60.6	284

R-Value at 300psi exudation pressure = 45

R-VALUE DETERMINATION

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

SKYLONDA FIRE STATION
Replacement Barracks and Office Building
17290 Skyline Boulevard
Woodside, San Mateo County, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	- - -	869.1	March 1996	18

RUTHERFORD + CHEKENE
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