QUESTA
ENGINEERING CORP.

# Geotechnical Investigation
## For Additional Dwelling Unit
## 914 Everett Ave, Oakland, CA

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*Prepared For:*

**Mr. Andrew Sundblad**
*914 Everett Avenue*
*Oakland, CA*

*Prepared By:*

**Questa Engineering Corporation**
*1220 Brickyard Cove Road,*
*Suite 206*
*Point Richmond, CA 94801*

*May 13, 2019*

Civil,
Environmental
& Water
Resources

QUESTA ENGINEERING CORP.
Civil, Environmental & Water Resources

May 13, 2019

Mr. Andrew Sundblad
914 Everett Ave.
Oakland, CA

Subject: Geotechnical Investigation for 2-story Residential Additional Dwelling Unit, 914 Everett Avenue, Oakland, California

Dear Mr. Sundblad:

This report presents results of a geotechnical investigation for an additional dwelling unit in the rear portion of the property at 914 Everett Avenue, Oakland, California located as shown on **Figure 1**. Questa performed a subsurface investigation that included hand augering and dynamic cone penetrometer test holes, laboratory soils testing of physical properties of soils, engineering analysis and development of geotechnical recommendations for development of a residential/studio addition.

## SUBSURFACE INVESTIGATION

Questa performed a subsurface investigation that included the completion of hand augering and hand sampling of surface soils and the completion of four Dynamic Cone Penetrometer (DCP) test holes using the Triggs Wildcat system. The Triggs Wildcat utilizes a 35-pound hammer to drive a 1.4 inch diameter cone tip connected by steel rods. No drilling is performed during the testing. The Triggs Wildcat DCP data is in blows per 4 inches, which is equivalent to the Standard Penetration Test (SPT) which utilizes a 140-pound hammer dropped from 30 inches and provides blow counts at six inch increments, the last 12 inches of which are known as the N-value. The N-value is indicative of the strength of the material being penetrated. All DCP holes are backfilled with lean cement concrete following completion.

Four DCP holes were completed at the locations as shown on **Figure 1**. **Figures 2, 4, 5,** and **7** present the Wildcat Dynamic Cone Logs for the holes T-1, T-2, T-3, and T-4. Three hand-auger holes, HA-1, HA-2, and HA-3 (**Figures 3, 6** and **8**), with depths ranging from 6.5 to 9.5 feet below ground surface (BGS), were completed at the locations shown on **Figure 1.** Soil samples were logged from the hand-auger holes and samples were collected for laboratory testing and physical properties description. Undisturbed soil samples were collected using the AMS hand sampler in the hand-auger holes at depths of 5.0 to 5.5 feet BGS in HA-1 and 9.0 to 9.5 feet BGS in HA-3. The logs of hand augers describe the soils in accordance with the Unified Soil Classification System (USCS), which is summarized in **Figure 9**.

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***Soils Encountered***

Based on review of the soil samples and DCP logs, the site soil profile consists generally of approximately 1 to 2 feet of sandy clay with organic surface soils overlying sandy clay sub-soils, which increase in stiffness and decrease in moisture content with increasing depth. Surface soils consisted of dark brown sandy clay with plant roots. The surface materials had moderate plasticity, were soft and moist to wet.

DCP T-1 (**Figure 2**) was located at the southeast corner of the proposed residential unit at the existing house level, as shown in **Figure 1**. T-1 penetrated sandy clay soils of medium stiffness from 1 to 3.5 feet BGS, with a soft layer from 4 to 4.5 feet BGS, and medium stiff sandy clay from 4.5 to 6.0 feet BGS. Stiff sandy clay was encountered at 6.0 feet BGS, which transitioned to very stiff sandy clay material at 6.5 feet BGS. At 7.5 feet BGS, the hard material was encountered until the end of the hole at 10 feet BGS.

Hand-auger hole HA-1 was located adjacent to T-1 (shown in **Figure 1**) and revealed soft to medium stiff dark brown sandy clay in the upper 2.5 feet BGS (**Figure 3**). At approximately 3 feet BGS, medium stiff reddish brown sandy clay was present. From 4.0 feet to 6.0 feet BGS, dark yellowish brown sandy clay was present, which transitioned to brownish yellow sandy clay with brownish gray mottling, with the material increasing in stiffness. From 6.0 feet to 7.5 feet BGS, stiff to very stiff moist brownish yellow sandy clay was encountered, which then transitioned to brownish yellow clayey sand with gray clay mottling from 7.5 to 8.5 feet BGS, the bottom of the hole.

DCP T-2 (**Figure 4**) was located at the northeast corner of the proposed residential unit at a terrace roughly 10.5 feet below T-1. DCP T-4 (**Figure 5**) was at the same general elevation as T-2, but at the northwest corner of the proposed unit. Both T-2 and T-4 encountered similar subsurface conditions. T-2 encountered soft to medium stiff material within the first foot. From 1.0 to 2.5 feet, medium stiff to stiff sandy clay was found, although this was likely attributed to rocky fill material bordering the foundation wall. From 2.5 to 3.5 feet BGS, medium stiff sandy clay was found, which transitioned from stiff to very stiff sandy clay from 3.5 to 5 feet BGS. From 5 feet to refusal at 7 feet BGS, hard weathered bedrock was encountered. T-4 encountered soft to medium stiff soil from ground surface to 2.0 feet BGS. From 2 feet BGS to 5 feet BGS, medium stiff sandy clay soils were encountered, which transitioned to hard weathered rock from 4.0 to 7.0 feet BGS.

Hand-auger hole HA-2 was located adjacent to T-2 (shown in **Figure 1**) and revealed soft to medium stiff dark brown sandy clay in the upper 1.5 feet BGS (**Figure 6**). From 1.5 to 4.0 feet BGS, yellow brown moist sandy clay, medium stiff to stiff, was encountered. From 4.0 to 5.0 feet BGS, brownish yellow sandy clay was encountered. From 5.0 feet to 6.5 feet BGS, brownish yellow clayey sand with shale fragments was encountered.

DCP T-3 (**Figure 7**) was located in the center of the proposed unit at a terrace roughly 4.5 feet below T-1. A 5-inch diameter hole was drilled in the concrete slab at the test location before initiating driving of the dynamic cone penetrometer, which started 0.5 feet BGS. Soft clay was encountered from 0.5 to 3.0 feet, with medium stiff to stiff sandy clay from 3.0 to 4.5 feet, and very stiff to hard sandy clay from 4.5 to 6.5 feet. A very high blow count was recorded at 6.5 feet BGS, with the material

decreasing to very stiff to hard, from 6.5 to 8.5 feet. From 8.5 to 14.0 feet the material was stiff to very stiff sandy clay, and from 14.0 to 15.0 feet (the total depth of the hole) it was hard weathered rock.

Hand-auger hole HA-3 (**Figure 8**) was located adjacent to T-3. Similar to T-3, the 0.5 feet thick concrete slab was drilled through before augering began. Material within the first 4.5 feet BGS was soft, dark brown sandy clay intermixed with brick and gravel, thought to be fill material. From 4.5 feet to 6.5 feet BGS, moist, brownish red sandy clay was encountered. There was a color change at 6.5 feet BGS to brownish yellow sandy clay, followed by brownish gray sandy clay with gravel from 7.0 feet to 8.5 feet BGS. From 8.5 feet to the bottom of the hole at 9.5 feet, brownish yellow sandy clay was encountered.

### *Groundwater Conditions*

No groundwater was found at the site. Moist soils were encountered in all holes, but no free groundwater was discovered.

## LABORATORY TESTING

Laboratory testing was performed on selected soil samples from the hand-auger holes and associated driven samples. Laboratory testing was performed in Questa's laboratory in general accordance with American Society for Testing and Materials (ASTM) standards for moisture content, dry density, particle size analysis, and liquid and plastic limits (including plasticity index). Unconfined compressive strength testing was performed using the pocket penetrometer and shear strength testing was performed using a torvane shear device.

### *Moisture-Density*

Moisture content and dry density testing were performed on undisturbed soil samples collected in HA-1 at 5 to 5.5 feet and HA-3 at 9 to 9.5 feet. Moisture content was also determined in three grab samples. Testing was performed in accordance with ASTM D 2937. In this test, the dry density of the soil is determined by a mathematical relationship between moisture content and wet density of the soil sample. Results of moisture content and dry density testing are summarized on the logs of hand auger holes HA-1, HA-2, and HA-3 (**Figures 3, 6, and 8**).

### *Particle Size Analysis*

Particle size analysis testing was performed in accordance with ASTM D 422. Three samples from HA-1, HA-2, and HA-3 were tested for particle size analysis. Results are presented on **Figure 10** and **Figure 11.** Based on the test results, the soils consist predominantly of sandy clay, with a greater sand content at depth.

### *Liquid Limit, Plastic Limit and Plasticity Index*

Testing of liquid limit, plastic limit and plasticity index were performed in accordance with ASTM D 4318. Results are presented on **Figure 12.** Site soils consisted of mostly lean to moderately lean sandy clays.

## GEOLOGY AND SEISMICITY

The site lies in the tectonically active Coast Ranges Geomorphic Province of Northern California in the Oakland, Alameda County area. Active tectonics in the region is controlled by the San Andreas fault system, the boundary between the Pacific and North American tectonic plates. Much of the landscape and resources of the San Francisco Bay Region have their origins in the region's complex geological history. The region lies within the California Coast Ranges geomorphic and physiographic province, a region dominated by active tectonics along the margin between the Pacific and North American tectonic plates. Over time, the tectonic forces have created the varied mountainous, valley, and fault-bound blocks which occur in the San Francisco Bay Region.

### *Regional Faulting and Seismicity*

The State of California has developed a system to assess the activity of faults. Under this system, faults are classified active if they have ruptured at the ground surface in the last 11,000 years, which is within the Holocene Epoch. Faults that have ruptured between 1,600,000 (1.6 million) years ago and 11,000 years ago, within the Pleistocene Epoch, are considered conditionally or potentially active. Other faults that do not show evidence of fault displacement within the last 1.6 million years are considered inactive.

The Fault Activity Map of California shows the faults that are considered active, potentially active or inactive[1]. There are no faults considered active by the State of California in the project site neighborhood. The nearest faults in the vicinity of the project site considered active in accordance with the Alquist-Priolo Earthquake Fault Zoning Act are the Hayward fault located approximately 1.5 miles to the northeast, the Calaveras fault located 13 miles to the southeast, the San Andreas fault 17 miles to the southwest, and the Concord fault located 19 miles to the northeast of the project site (**Figure 13**). These active faults may rupture during or following an earthquake, causing surface displacement or offset along the fault trace. Active seismicity in the San Francisco Bay area is controlled by the San Andreas Fault system. A list of active faults within approximately 50 miles of the project site is presented as **Table 1**.

The active seismicity of the region also results in numerous earthquakes. Many of these earthquakes are too small to be felt by humans. The largest historical earthquake to occur on the Hayward fault was the October 21, 1868 Richter Magnitude 6.8 earthquake, which resulted in widespread damage and destruction in the East Bay area. According to the Working Group on California Earthquake Probabilities (2015)[2], there is a 14.3% chance of a Magnitude 6.7 or greater earthquake occurring on the Hayward fault in the next 25 years (2014-2044), a 7.4% probability of a $M \geq 6.7$ on the Calaveras fault, and a 6.4% probability of a $M \geq 6.7$ on the San Andreas fault between 2014-2044.

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[1] California Geological Survey, 2010, Fault Activity Map of California and Adjacent Areas, Geologic Data Map No. 6, Scale 1:750,000

[2] Working Group on California Earthquake Probabilities, 2015, Long-term time-dependent probabilities for the third uniform California earthquake rupture forecast (UCERF3), Bulletin of the Seismological Society of America.

**Table 1: Major Faults Within 50 Miles of the Project Site**

| Fault Name | Distance from Project Site (mi.) | Direction | Last Surface Rupture | Status* | Maximum Moment Magnitude[3] |
|---|---|---|---|---|---|
| Hayward | 1.5 | NE | Historic | Active | 7.1 |
| Calaveras | 13 | SE | Holocene | Active | 6.9 |
| San Andreas | 17 | SW | Historic | Active | 7.9 |
| Concord | 19 | E | Historic | Active | 6.5 |
| Green Valley | 24 | NE | Holocene | Active | 6.6 |
| Seal Cove | 25 | SW | Holocene | Active | 7.4 |
| Greenville | 25 | SE | Historic | Active | 7.3 |
| West Napa | 28 | NE | Holocene | Active | 6.5 |
| Rodgers Creek | 28 | N | Holocene | Active | 6.9 |
| San Gregorio | 40 | SW | Late Quaternary | Potentially Active | -- |
| Maacama | 54 | NW | Holocene | Active | 7.3 |

*Faults showing displacement during Holocene time are considered active, faults showing evidence of displacement during Late Quaternary time are considered conditionally or potentially active, older faults are considered inactive.

### *Seismic Hazard Zones*

The project site is located in an area mapped as part of the Seismic Hazard Zone Program of the State of California. The map of Earthquake Zones of Required Investigation, Oakland West Quadrangle, shows the location of seismic hazard zones and earthquake fault zones[4]. Based on review of the map of the project area, the site is not located in an area expected to have seismically induced landslide hazards or liquefaction hazards. The nearest Alquist-Priolo Earthquake Fault Zone is for the Hayward fault and is located 1.5 miles to the east-northeast of the project site (**Figure 14**).

### *Geology*

The project site area is mapped on the Areal and Engineering Geology of the Oakland East Quadrangle, California, USGS GQ-769 (**Figure 15**), United States Geological Survey, 1967)[5] and on the Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties (United States Geological Survey, 2000)[6]. According to these maps, the project site is underlain predominantly by Franciscan Complex sandstone and shale. While no outcroppings were found in the area, shale, siltstone and sandstone fragments were found in hand auger samples from HA-2.

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[3] 2007 Working Group on California Earthquake Probabilities (WGCEP). Uniform California Earthquake Rupture Forecast, Version 2. USGS Open File Report 2007-1437, CGS Special Report 20, 2008 and 2008 USGS National Seismic Hazards Maps – Source Parameters

[4] California Geological Survey, 1982 and 2003, Earthquake Zones of Required Investigation, Oakland East and West Quadrangle, including the Alquist-Priolo Earthquake Fault Zones Map and the Seismic Hazard Zones Map.

[5] United States Geological Survey (D.H. Radbruch), 1967, Areal and Engineering Geology of the Oakland East Quadrangle, California, USGS GQ-769

[6] United States Geological Survey (RW Graymer), 2000, Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California, USGS MF 2342, Version 1.0.

### *Landslides*

The site property is located in an area of the Oakland hills. No evidence of unstable slopes was observed. All geologic maps reviewed, including the Quaternary Geology of Alameda County Map (USGS, 1997), showed no indication of landslide history or susceptibility on the site. No visual evidence of slope instability was observed at the site during the investigation. However, cuts made on slopes for development of the site destabilize slopes and require retention to prevent slope instabilities from occurring.

### *Expansive Soils*

The soils underlying the site are low to moderately expansive clay soils consisting of sandy lean clay. These low to moderately expansive soils are not suitable for foundation support in the top 4 feet below ground surface in their current condition. The surface sandy clay is susceptible to both short-term and long-term settlement as well as heaving due to the moderate plasticity of the material, low density, and the variable seasonal moisture contents. The underlying sandy clay soils have a moderate plasticity but are less susceptible to settlement due to higher density and soil strength and being below the zone of seasonal moisture fluctuations.

## DESIGN RECOMMENDATIONS

The proposed foundation for the new 2-story residential unit is a conventional spread footing foundation. The soils in the upper four to five feet are moderately expansive clay soils with some organic material in the upper two feet, with soil bearing characteristics that are not suitable for shallow foundations. Based on results of our field investigation and laboratory testing we recommend that the new 2-story unit be supported on a conventional spread footing foundation founded on non-expansive engineered fill soils such as Class 2 aggregate base, or alternatively controlled low-strength material (CLSM- sand-gravel cement slurry) extending to a depth of 48 inches below existing ground surface at the base of the structure. This can be accomplished by deepening footing excavations from a conventional 24 inches below grade to a minimum of an additional 24 inches below the footing depth as specified in typical site Footing Details, to a total of 48 inches below existing ground surface and by replacing the unsuitable soils with suitable structural soils below the foundation. The soil and organic material should be replaced with non-expansive engineered structural fill such as Class 2 AB compacted to a minimum of 95 percent of the maximum dry density at moisture contents within two percent of the optimum (in accordance with ASTM D 1557), or with CLSM (minimum compressive strength of 100 psi at 28 days).

### *Site Preparation and Grading*

Areas to be excavated for foundation and grade beam construction should first be stripped of vegetation and organic topsoil, which should be off-hauled or used in landscape areas. A minimum of 4 feet of soil and organic materials should be removed in footing excavation areas. Any excavated soils to remain on the site should be properly keyed in and compacted to a minimum of 90 percent of the maximum dry density at moisture contents of two to four percent above the optimum.

We should be present to observe and test subgrade soils and structural fill soils. All structural fill should be compacted to a minimum of 95 percent of the maximum dry density at moisture contents within two percent of the optimum as determined in accordance with ASTM D 1557. Areas to receive engineered structural fill should be stripped of organic topsoil as described above. Subgrade soils below the structural fill section should be scarified to a depth of ten inches, moisture conditioned from two to four percent above the optimum and compacted to a minimum of 90 percent of the maximum dry density. Moisture conditioning may involve drying soils if soils are wet or adding moisture if soils are dry.

Engineered structural fill should consist of a select material, with no rocks or lumps exceeding three inches in diameter, no organic material, a Liquid Limit of less than 40, a Plasticity Index of 12 or less, and a low expansion potential. Caltrans Class 2 aggregate base is an acceptable material for this use. If CLSM is used, it should have a minimum compressive strength of 100 psi at 28 days.

### *Foundation Design*

Footings**.** The new residence foundations can be standard spread footings and grade beams provided that the overlying weak surface soils and organic materials are excavated to a minimum of 48 inches below existing grade exposing the underlying competent stiff to very stiff sandy clay soils. The zone below the footing and grade beam depth and the stiff to very stiff sandy clay soils may be replaced with non-expansive engineered structural fill such as Class 2 AB or with CLSM as described above. For conventional spread footings founded on engineered structural fill or lean cement concrete, allowable bearing pressure of 2,000 pounds per square foot (psf) can be used for dead plus live loads, and can be increased by 33 percent for total loads, including wind or seismic forces. Resistance to lateral loads should be based on a passive pressure of 200 psf/ft of depth on the face of the footing in competent sandy clay. In addition, a friction coefficient of 0.35 on the base of the footing can be used.

Continuous footings should be a minimum of 12 inches wide; column footings should be a minimum of 18 inches square. Footings should have a minimum depth of 24 inches below the lowest adjacent grade.

### *Concrete Slabs-on-Grade*

Areas to receive concrete slabs-on-grade should span the weak and expansive sandy clay soils through the use of grade beams and perimeter spread footings founded on non-expansive fill or CLSM as described above. We also recommend that a minimum of 12 inches of Class 2 AB or non-expansive structural fill be placed beneath the slabs and be compacted to a minimum of 95 percent of the maximum dry density. The expansive sandy clay soils have a moderate expansion potential and may exert moderately high pressures below slabs during heaving. Steel reinforcing should be placed within the concrete to reduce the effects of cracking, and should be designed to resist the high expansion potential of the clay soils. Concrete slabs should be a minimum of 5 inches in thickness.

Concrete slabs-on-grade should be underlain by at least four inches of clean, free draining gravel or crushed rock, to act as a capillary moisture break. Slab rock should be graded as follows: 100

percent passing the 1.0 inch sieve, 90-100 percent passing the ¾-inch sieve, and 0 to five percent passing the No. 4 sieve. This material is commonly referred to as ¾-inch crushed rock. A vapor barrier such as Stego Wrap 15 mil vapor barrier or approved equivalent should be placed on the underside of the slab to prevent the movement of moisture through the slab. Sand may also be placed below the slab.

***Retaining Walls***

Retaining Walls at the site must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads from sloping backfills. Retaining walls that are free to rotate should be designed for active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation, then they should be designed for at rest earth pressures. Retaining wall design should resist the assumed lateral earth pressures as shown in the following **Table 3**.

**Table 3. Retaining Wall Design Parameters**

| | Active Pressure pounds per cubic foot (pcf) | At-Rest Earth Pressure (pcf) | Seismic Loading (psf) (for walls greater than 6' in height) |
|---|---|---|---|
| Level Backfill | 51 | 72 | 27H |

All backfilled retaining walls should be fully drained as described below under the section titled ***Drainage Measures*** and should have foundations as described above under ***Foundation Design.***

The seismic conditions should be determined by adding the pressures from earthquake loading to active pressure on the retaining walls. All walls greater than 6 feet in height should include seismic pressure. We recommend an incremental seismic pressure of 27H in pounds per square foot (psf), where H is the height of the retaining wall in feet. The pressure distribution may be considered to be an inverted triangle with the maximum pressure at the top and zero on the bottom. The resultant of this force may be assumed to be located at 1/3 the height of the wall below the top of the wall. Unit weight of the retained sandy clay soils is approximately 125 pcf. The effective internal angle of friction of the retained sandy clay soils can be assumed to be 25 degrees for design purposes.

***Drainage Measures***

The final grade should provide positive runoff away from, and prevent ponding or saturation of, the foundation and pavement areas in accordance with requirements of the California Building Code. Water should be removed to the nearest accessible drainage such as the storm drain at the street.

Areas adjacent to the residence foundation should also be graded to prevent water from ponding against or saturating any foundation area. The new residence should have roof gutters and all roof gutter downspouts should be tied into a solid or a tight line and be carried to a storm drain system, natural drainage or energy dissipating structure away from foundation areas.
A perimeter subsurface drain should be installed to minimize the flow of water to expansive clay soils underlying the residence and foundations. The perimeter sub-drain should consist of a perforated four-inch diameter PVC Schedule 40 or SDR 35 pipe covered in a minimum of 12

inches of drain rock and wrapped in filter fabric. This subsurface drain should extend to at least the base of the grade beam or footing, have a minimum slope of two percent and be gravity drained by connection to a non-perforated PVC tight line with water transmitted to a location away from the foundation.

Retaining walls should be back-drained with a perforated four-inch diameter PVC Schedule 40 or SDR 35 pipe covered in a minimum of 36 inches of ¾-inch diameter drain rock and wrapped in filter fabric such as Mirafi 160N. Subsurface drains should extend to at least the base of the grade beam or footing, have a minimum slope of two percent and be gravity drained by connection to a non-perforated PVC tight line with water transmitted to a non-erodible structure away from the foundation.

### *Corrosion*

Corrosion testing of site soils was not performed. Based on our experience in the area, site soils could potentially be corrosive to steel. As a precaution, corrosion resistant type II/V concrete should be used at the site, which is standard in this region.

### *Seismic Design Criteria*

The project seismic design criteria were calculated in accordance with provisions of 2010 ASCE 7-10 in accordance with the 2016 California Building Code, using the OSHPD Seismic Design Maps calculator on 5/13/2019. This is based on United States Geological Survey data. The project site was assigned to Site Class D, stiff soil conditions based on results of our Geotechnical Investigation. This information is summarized in **Table 4**, along with seismic design criteria for design of project elements required to be designed in accordance with the 2016 California Building Code seismic design criteria and 2010 ASCE 7-10 (with 2013 errata).

**Table 4. Seismic Design Criteria in accordance with ASCE 7-10 and 2016 CBC**

| | |
|---|---|
| Site Class | D |
| Soil Profile Name | Stiff Soil |
| Seismic Design Category | E |
| Mapped Spectral Response for Short Periods- 0.2 Sec ($S_s$) | 2.275 g |
| Mapped Spectral Response for Long Periods- 1 Sec ($S_1$) | 0.944 g |
| Adjusted Maximum Considered EQ Spectral Response for Short Periods ($S_{MS}$) | 2.275 |
| Adjusted Maximum Considered EQ Spectral Response for Long Periods ($S_{M1}$) | 1.415 |
| Design (5-percent damped) Spectral Acceleration Parameters at short periods ($S_{DS}$) | 1.516 |
| Design (5-percent damped) Spectral Acceleration Parameters at long periods ($S_{D1}$) | 0.944 |
| $F_a$ Site amplification factor at 0.2 second | 1.0 |
| $F_v$ Site amplification factor at 1.0 second | 1.5 |
| $T_L$ Long-period Transition period in seconds | 8 seconds |
| $MCE_G$ Peak Ground Acceleration (PGA) | 0.874 |
| $F_{PGA}$ Site Amplification factor at PGA | 1.0 |
| $PGA_M$ Site-modified Peak Ground Acceleration | 0.839 |
| $C_{RS}$ Mapped value of the risk coefficient at short periods | 1.010 |
| $C_{R1}$ Mapped value of the risk coefficient at a period of 1 second | 0.987 |

## CONCLUSIONS

The project is feasible from a geotechnical standpoint. Provided that the site is properly prepared and the foundations are designed and constructed as recommended, we estimate that post construction settlement will be small, less than 1.0-inch and differential settlements should be less than 0.5-inches within a 25-foot span.

We should review the prepared plans for conformance with our recommendations prior to issuance for construction. During construction, we should observe the excavation of footings as required by the California Building Code, to verify that the required penetration into competent soil is achieved in the footings. We should observe grade beam or footing preparation prior to concrete pouring, and we should observe that drainage measures are properly installed and constructed. Any filling on the site should be observed and tested to confirm that minimum compaction results are achieved.

## LIMITATIONS

This investigation was performed in accordance with present geotechnical and engineering geologic standards applicable to this project. In our opinion, the scope of services adequately supports the conclusions and recommendations presented. The findings are valid now, but should not be relied upon after three years without our review.

The recommendations of this report are based upon the assumption that the subsurface conditions do not deviate from those interpreted from the surface and subsurface data of this investigation. If any variation or undesirable conditions are encountered during construction, or if the proposed construction differs from that planned at the present time, we should be notified so that supplemental recommendations can be given. The recommendations of this report are intended for the site described only, and must not be extended to adjacent areas.

This report is issued with the understanding that it is the responsibility of the owner to ensure that contractors and subcontractors carry out the recommendations presented.

## ADDITIONAL SERVICES

We should review the project plans and specifications for conformance with the intent of our recommendations. During construction we should observe and test all site preparation and grading to check the results of work by your contractor. This will allow us to observe that subsurface conditions are as anticipated and to make supplemental recommendations when needed. These services during construction should include:

- Site preparation and fill placement should be observed and tested. This includes subgrade preparation and compaction below fills and engineered fill soils such as structural fill.

- Subgrade for all fill and concrete slabs-on-grade should be tested and approved before placing fill or rock.
- The excavation of footings should be observed to confirm that firm supporting material is encountered and to develop/verify depth criteria in accordance with building code requirements.
- Subsurface drains should be observed prior to covering in drain rock.

Results of our observations and testing during construction will be summarized in a written report following completion of the foundation and earthwork. We will require a minimum of two working days (48 hours) notice prior to the start of earthwork to schedule field personnel for the necessary observations and testing.

We trust this is the information you require at this time. Should you have any questions please contact the undersigned at (510) 236-6114, extension 222.

Sincerely,

Norman N. Hantzsche, P.E.
Civil Engineer #24750
Principal/Managing Engineer

Willard N. Hopkins, CEG. #1761, exp. 7-31-21
Sr. Engineering Geologist/Project Manager

Ref.: 1900044_report

# FIGURES

PROJECT NO. 1900044

DRAWN BY TJC 4-24-2019

# GEOTECHNICAL LOCATION

*914 EVERETT AVENUE GEOTECHNICAL INVESTIGATION*
*OAKLAND, CA*

QUESTA ENGINEERING CORP.

**FIGURE 1**

# WILDCAT DYNAMIC CONE LOG



## Figure 2

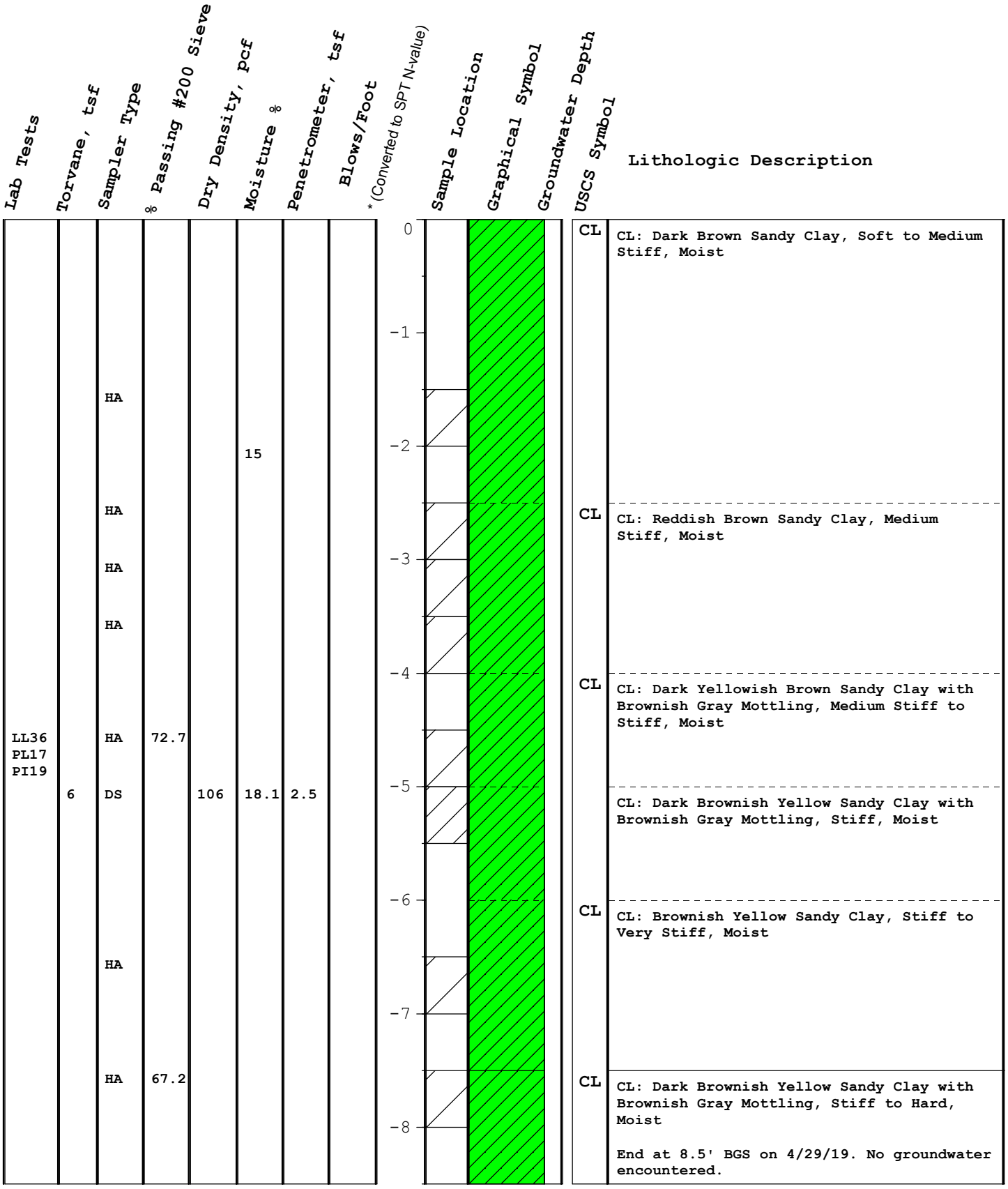
Questa Engineering
1220 Brickyard Cove Road
Richmond, CA 94801

PROJECT NUMBER: 1900044
DATE STARTED: 04-29-2019
DATE COMPLETED: 04-29-2019

HOLE #: T-1
CREW: TC, MV
PROJECT: Sunblad
ADDRESS: 914 Everett Ave
LOCATION: Oakland, CA

SURFACE ELEVATION:
WATER ON COMPLETION: None
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

| DEPTH | BLOWS PER 10 cm | RESISTANCE Kg/cm² | GRAPH OF CONE RESISTANCE 0 50 100 150 | N' | TESTED CONSISTENCY NON-COHESIVE | COHESIVE |
|---|---|---|---|---|---|---|
| - | 7 | 31.1 | ••••••••• | 8 | LOOSE | MEDIUM STIFF |
| - | 6 | 26.6 | ••••••• | 7 | LOOSE | MEDIUM STIFF |
| - 1 ft | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - 2 ft | 6 | 26.6 | ••••••• | 7 | LOOSE | MEDIUM STIFF |
| - | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - 3 ft | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - 1 m | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - | 5 | 19.3 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - 4 ft | 4 | 15.4 | •••• | 4 | VERY LOOSE | SOFT |
| - | 4 | 15.4 | •••• | 4 | VERY LOOSE | SOFT |
| - | 5 | 19.3 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - 5 ft | 5 | 19.3 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 6 | 23.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - | 7 | 27.0 | ••••••• | 7 | LOOSE | MEDIUM STIFF |
| - 6 ft | 9 | 34.7 | ••••••••• | 9 | LOOSE | STIFF |
| - | 12 | 46.3 | •••••••••••• | 13 | MEDIUM DENSE | STIFF |
| - 2 m | 18 | 69.5 | •••••••••••••••••• | 19 | MEDIUM DENSE | VERY STIFF |
| - 7 ft | 24 | 82.1 | •••••••••••••••••••••• | 23 | MEDIUM DENSE | VERY STIFF |
| - | 25 | 85.5 | ••••••••••••••••••••••• | 24 | MEDIUM DENSE | VERY STIFF |
| - | 35 | 119.7 | •••••••••••••••••••••••••••••••• | 25+ | DENSE | HARD |
| - 8 ft | 47 | 160.7 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | DENSE | HARD |
| - | 86 | 294.1 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | 69 | 236.0 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 9 ft | 25 | 85.5 | ••••••••••••••••••••••• | 24 | MEDIUM DENSE | VERY STIFF |
| - | 38 | 130.0 | •••••••••••••••••••••••••••••••••• | 25+ | DENSE | HARD |
| - | 56 | 191.5 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 3 m 10 ft | 48 | 164.2 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | DENSE | HARD |
| - | 110 | 336.6 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | | | | | | |
| - | | | | | | |
| - 11 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 12 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 4 m 13 ft | | | | | | |




*Sampling was performed by Questa Engineering using a hand auger.

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Civil Environmental & Water Resources
P.O. Box 70356 1220 Brickyard Cove Road Point Richmond, CA 94807

# LOG OF HAND AUGER HA-1

**Sundblad**

**Oakland, CA**

**Figure 3**

# WILDCAT DYNAMIC CONE LOG



## Figure 4

Questa Engineering
1220 Brickyard Cove Road
Richmond, CA 94801

PROJECT NUMBER: 1900044
DATE STARTED: 04-29-2019
DATE COMPLETED: 04-29-2019

HOLE #: T-2
CREW: TC, MV
PROJECT: Sunblad
ADDRESS: 914 Everett Ave
LOCATION: Oakland, CA

SURFACE ELEVATION:
WATER ON COMPLETION: None
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

| DEPTH | BLOWS PER 10 cm | RESISTANCE Kg/cm² | GRAPH OF CONE RESISTANCE 0 50 100 150 | N' | TESTED CONSISTENCY NON-COHESIVE | TESTED CONSISTENCY COHESIVE |
|---|---|---|---|---|---|---|
| - | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - 1 ft | 3 | 13.3 | ••• | 3 | VERY LOOSE | SOFT |
| - | 11 | 48.8 | •••••••••••••• | 13 | MEDIUM DENSE | STIFF |
| - | 8 | 35.5 | •••••••••• | 10 | LOOSE | STIFF |
| - 2 ft | 7 | 31.1 | ••••••••• | 8 | LOOSE | MEDIUM STIFF |
| - | 8 | 35.5 | •••••••••• | 10 | LOOSE | STIFF |
| - | 7 | 31.1 | ••••••••• | 8 | LOOSE | MEDIUM STIFF |
| - 3 ft | 7 | 31.1 | ••••••••• | 8 | LOOSE | MEDIUM STIFF |
| - 1 m | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - | 7 | 27.0 | ••••••• | 7 | LOOSE | MEDIUM STIFF |
| - 4 ft | 9 | 34.7 | •••••••••• | 9 | LOOSE | STIFF |
| - | 10 | 38.6 | ••••••••••• | 11 | MEDIUM DENSE | STIFF |
| - | 16 | 61.8 | ••••••••••••••••• | 17 | MEDIUM DENSE | VERY STIFF |
| - 5 ft | 25 | 96.5 | ••••••••••••••••••••••••••• | 25+ | MEDIUM DENSE | VERY STIFF |
| - | 64 | 247.0 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | 54 | 208.4 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 6 ft | 63 | 243.2 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | 64 | 247.0 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 2 m | 66 | 254.8 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 7 ft | 108 | 369.4 | •••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | | | | | | |
| - | | | | | | |
| - 8 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 9 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 3 m 10 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 11 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 12 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 4 m 13 ft | | | | | | |

# WILDCAT DYNAMIC CONE LOG



## Figure 5

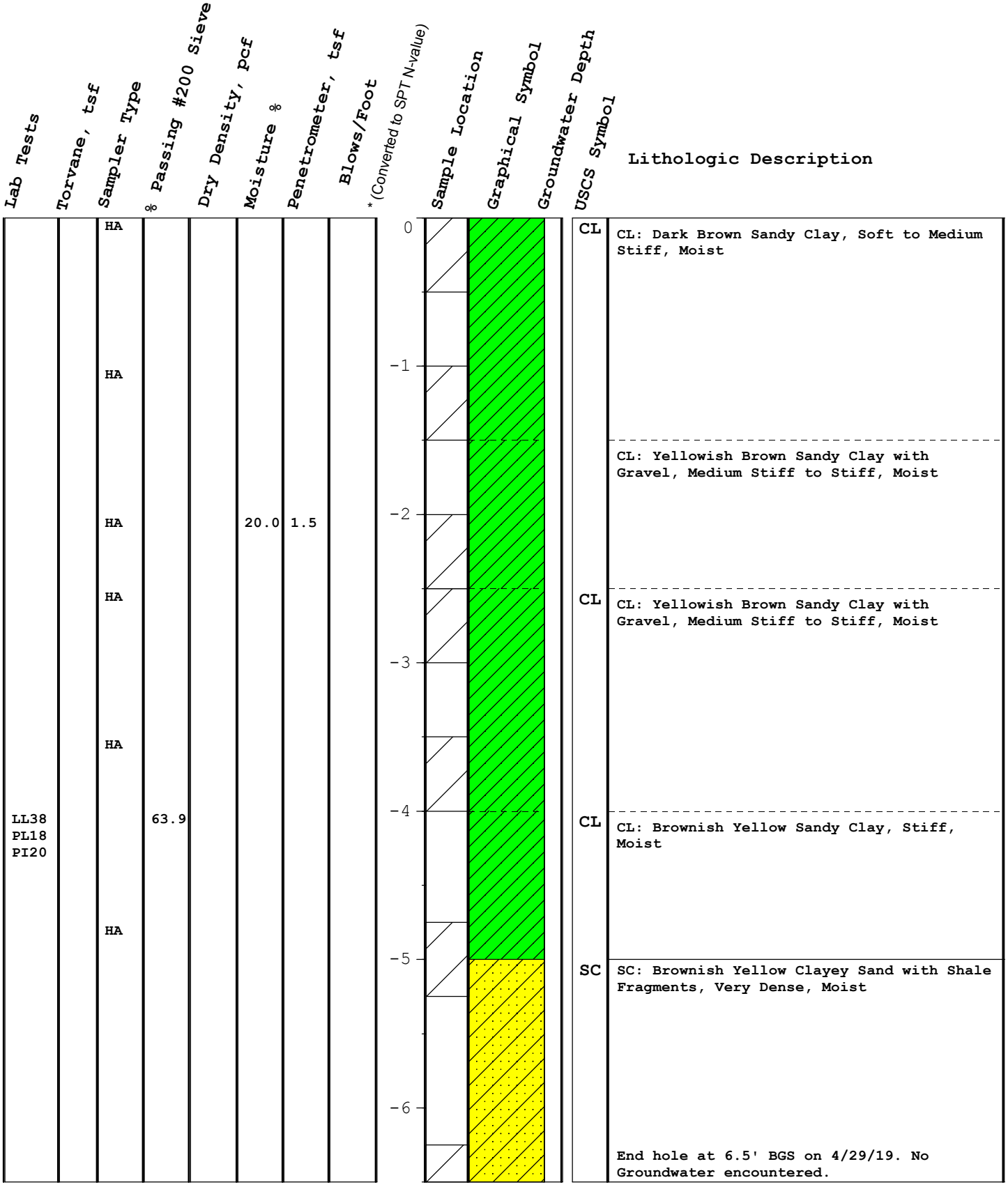
Questa Engineering
1220 Brickyard Cove Road
Richmond, CA 94801

PROJECT NUMBER: 1900044
DATE STARTED: 04-29-2019
DATE COMPLETED: 04-29-2019

HOLE #: T-4
CREW: TC, MV
PROJECT: Sunblad
ADDRESS: 914 Everett Ave
LOCATION: Oakland, CA

SURFACE ELEVATION:
WATER ON COMPLETION: None
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

| DEPTH | BLOWS PER 10 cm | RESISTANCE Kg/cm² | GRAPH OF CONE RESISTANCE 0 50 100 150 | N' | TESTED CONSISTENCY NON-COHESIVE | COHESIVE |
|---|---|---|---|---|---|---|
| - | 3 | 13.3 | ••• | 3 | VERY LOOSE | SOFT |
| - | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - 1 ft | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 3 | 13.3 | ••• | 3 | VERY LOOSE | SOFT |
| - 2 ft | 3 | 13.3 | ••• | 3 | VERY LOOSE | SOFT |
| - | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 5 | 22.2 | •••••• | 6 | LOOSE | MEDIUM STIFF |
| - 3 ft | 4 | 17.8 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - 1 m | 7 | 31.1 | ••••••••• | 8 | LOOSE | MEDIUM STIFF |
| - | 8 | 30.9 | •••••••• | 8 | LOOSE | MEDIUM STIFF |
| - 4 ft | 7 | 27.0 | ••••••• | 7 | LOOSE | MEDIUM STIFF |
| - | 8 | 30.9 | •••••••• | 8 | LOOSE | MEDIUM STIFF |
| - | 8 | 30.9 | •••••••• | 8 | LOOSE | MEDIUM STIFF |
| - 5 ft | 22 | 84.9 | •••••••••••••••••••••• | 24 | MEDIUM DENSE | VERY STIFF |
| - | 30 | 115.8 | •••••••••••••••••••••••••••••••• | 25+ | DENSE | HARD |
| - | 47 | 181.4 | •••••••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 6 ft | 69 | 266.3 | •••••••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | 73 | 281.8 | •••••••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 2 m | 93 | 359.0 | •••••••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 7 ft | 142 | 485.6 | •••••••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | | | | | | |
| - | | | | | | |
| - 8 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 9 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 3 m 10 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 11 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 12 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 4 m 13 ft | | | | | | |




*Sampling was performed by Questa Engineering using a hand auger.

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Civil Environmental & Water Resources
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# LOG OF HAND AUGER HA-2

**Sundblad**

**Oakland, CA**

**Figure 6**

# WILDCAT DYNAMIC CONE LOG

## Figure 7

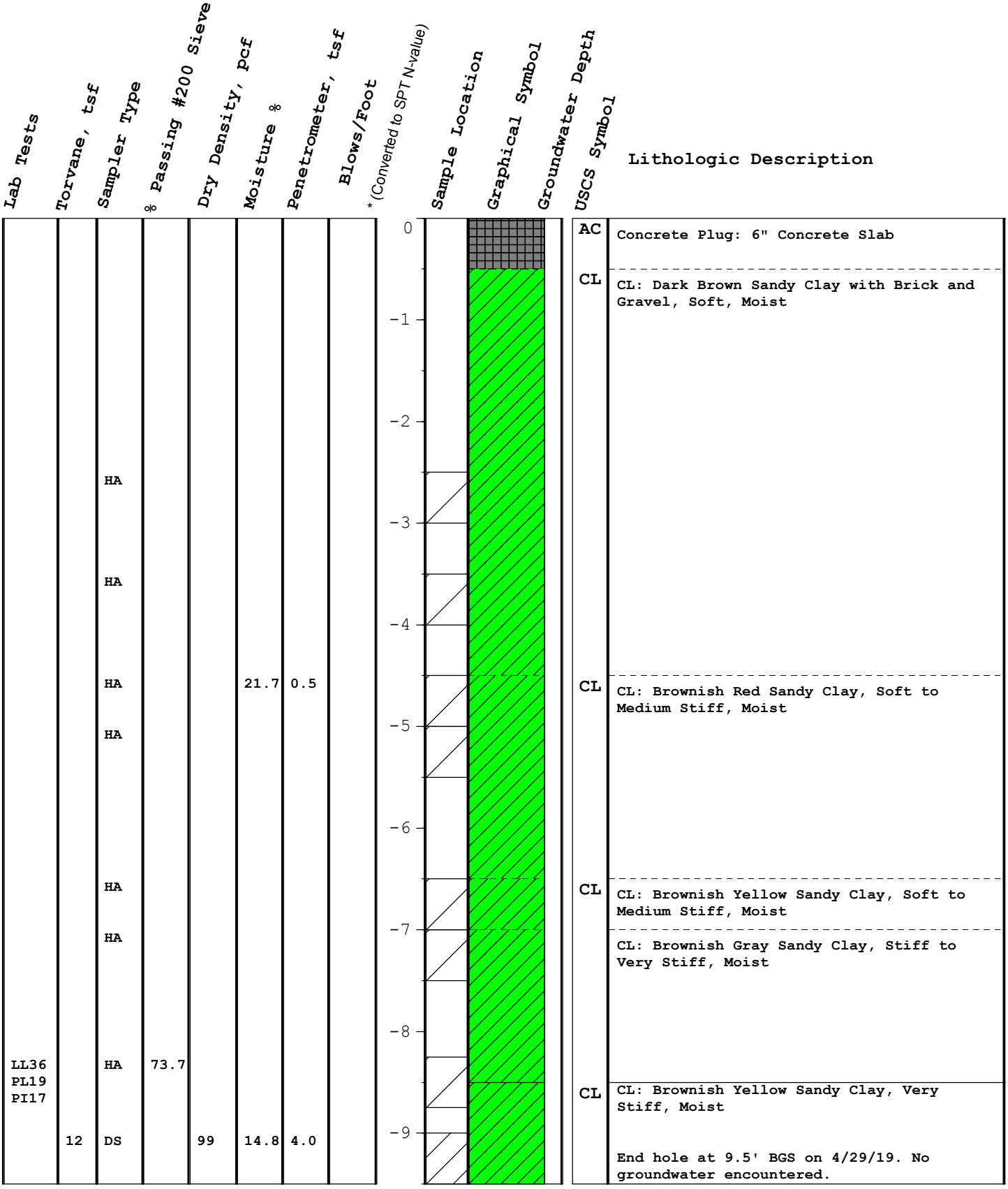
Questa Engineering
1220 Brickyard Cove Road
Richmond, CA 94801

PROJECT NUMBER: 1900044
DATE STARTED: 04-29-2019
DATE COMPLETED: 04-29-2019

HOLE #: T-3
CREW: TC, MV
PROJECT: Sunblad
ADDRESS: 914 Everett Ave
LOCATION: Oakland, CA

SURFACE ELEVATION:
WATER ON COMPLETION: None
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

| DEPTH | BLOWS PER 10 cm | RESISTANCE Kg/cm² | GRAPH OF CONE RESISTANCE 0 50 100 150 | N' | TESTED CONSISTENCY NON-COHESIVE | TESTED CONSISTENCY COHESIVE |
|---|---|---|---|---|---|---|
| - | | | | | | |
| - | | | | | | |
| - 1 ft | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - | 3 | 13.3 | ••• | 3 | VERY LOOSE | SOFT |
| - 2 ft | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - 3 ft | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - 1 m | 2 | 8.9 | •• | 2 | VERY LOOSE | SOFT |
| - | 3 | 11.6 | ••• | 3 | VERY LOOSE | SOFT |
| - 4 ft | 2 | 7.7 | •• | 2 | VERY LOOSE | SOFT |
| - | 2 | 7.7 | •• | 2 | VERY LOOSE | SOFT |
| - | 2 | 7.7 | •• | 2 | VERY LOOSE | SOFT |
| - 5 ft | 2 | 7.7 | •• | 2 | VERY LOOSE | SOFT |
| - | 3 | 11.6 | ••• | 3 | VERY LOOSE | SOFT |
| - | 3 | 11.6 | ••• | 3 | VERY LOOSE | SOFT |
| - 6 ft | 5 | 19.3 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - | 4 | 15.4 | •••• | 4 | VERY LOOSE | SOFT |
| - 2 m | 5 | 19.3 | ••••• | 5 | LOOSE | MEDIUM STIFF |
| - 7 ft | 5 | 17.1 | •••• | 4 | VERY LOOSE | SOFT |
| - | 11 | 37.6 | •••••••••• | 10 | LOOSE | STIFF |
| - | 17 | 58.1 | •••••••••••••••• | 16 | MEDIUM DENSE | VERY STIFF |
| - 8 ft | 18 | 61.6 | ••••••••••••••••• | 17 | MEDIUM DENSE | VERY STIFF |
| - | 20 | 68.4 | ••••••••••••••••••• | 19 | MEDIUM DENSE | VERY STIFF |
| - | 19 | 65.0 | •••••••••••••••••• | 18 | MEDIUM DENSE | VERY STIFF |
| - 9 ft | 17 | 58.1 | •••••••••••••••• | 16 | MEDIUM DENSE | VERY STIFF |
| - | 20 | 68.4 | ••••••••••••••••••• | 19 | MEDIUM DENSE | VERY STIFF |
| - | 22 | 75.2 | ••••••••••••••••••••• | 21 | MEDIUM DENSE | VERY STIFF |
| - 3 m 10 ft | 21 | 71.8 | •••••••••••••••••••• | 20 | MEDIUM DENSE | VERY STIFF |
| - | 20 | 61.2 | ••••••••••••••••• | 17 | MEDIUM DENSE | VERY STIFF |
| - | 20 | 61.2 | ••••••••••••••••• | 17 | MEDIUM DENSE | VERY STIFF |
| - | 26 | 79.6 | ••••••••••••••••••••••• | 22 | MEDIUM DENSE | VERY STIFF |
| - 11 ft | 31 | 94.9 | ••••••••••••••••••••••••••• | 25+ | MEDIUM DENSE | VERY STIFF |
| - | 60 | 183.6 | ••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - | 166 | 508.0 | ••••••••••••••••••••••••••••••••••••••••••• | 25+ | VERY DENSE | HARD |
| - 12 ft | | | | | | |
| - | | | | | | |
| - | | | | | | |
| - 4 m 13 ft | | | | | | |




*Sampling was performed by Questa Engineering using a hand auger.

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Civil Environmental & Water Resources
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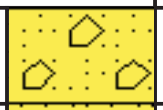
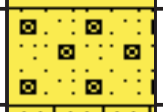
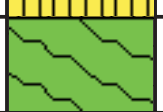
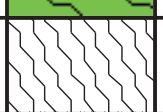
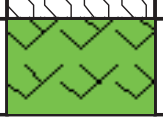
# LOG OF HAND AUGER HA-3

**Sundblad**

**Oakland, CA**

**Figure 8**

| MAJOR DIVISION | | | | | TYPICAL NAMES |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN HALF IS LARGER THAN #200 SIEVE | GRAVELS<br>MORE THAN HALF COARSE FRACTION IS LARGER THAN #4 SIEVE SIZE | CLEAN GRAVELS WITH LITTLE OR NO FINES | GW |  | Well graded Gravels, Gravel-Sand mixtures |
| | | | GP |  | Poorly graded Gravels, Gravel-Sand mixtures |
| | | GRAVELS WITH OVER 12% FINES | GM | | Silty Gravels, poorly graded, Gravel-Sand-Silt mixtures |
| | | | GC | | Clayey Gravels, poorly graded Gravel-Sand-Clay mixtures |
| | SANDS<br>MORE THAN HALF COARSE FRACTION IS LARGER THAN #4 SIEVE SIZE | CLEAN SANDS WITH LITTLE OR NO FINES | SW | | Well graded Sands, Gravelly-Sands |
| | | | SP | | Poorly graded Sands, Gravelly-Sands |
| | | SANDS WITH OVER 12% FINES | SM | | Silty Sands, poorly graded, Sand-Silt mixtures |
| | | | SC | | Clayey Sands, poorly graded, Sand-Clay mixtures |
| FINE GRAINED SOILS<br>MORE THAN HALF IS SMALLER THAN #200 SIEVE | SILTS AND CLAYS<br>LIQUID LIMIT 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 Clays and Organic Silty Clays of low plasticity |
| | SILTS AND CLAYS<br>LIQUID LIMIT 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 |

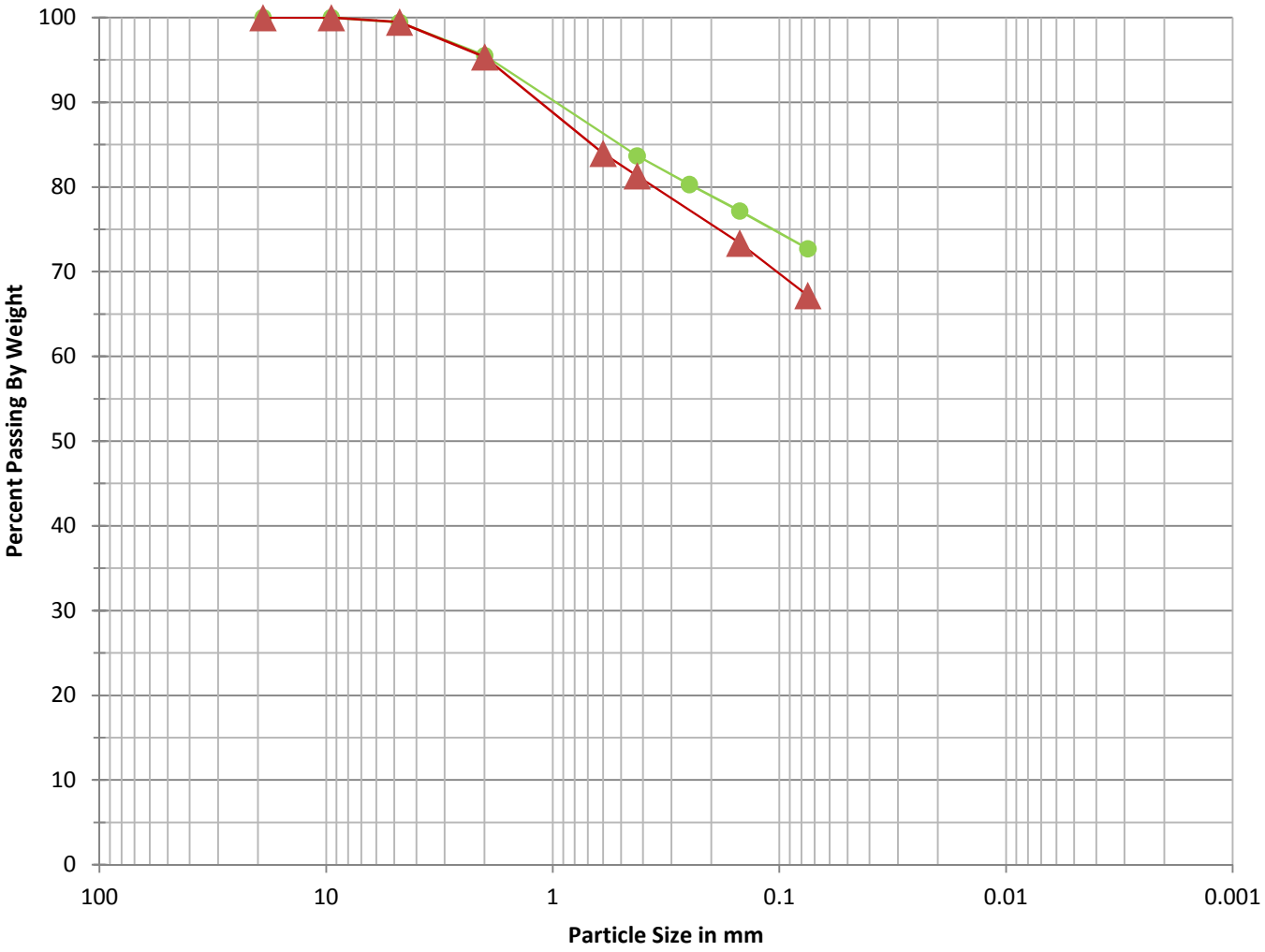
| | | | |
|---|---|---|---|
| | | DCP | DYNAMIC CONE PENETROMETER |
| | | DS | DRIVEN SAMPLE |
| | | HA | HAND AUGER |

Questa Engineering Corporation
P.O. Box 70356
1220 Brickyard Cove Road
Point Richmond, CA 94807
Phone: (510) 236-6114 FAX: (510) 236-2423

**UNIFIED SOIL CLASSIFICATION SYSTEM AND KEY TO ABBREVIATIONS**

FIGURE 9

# Particle Size Analysis




| Gravel | | Sand | | | Silt | Clay |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | | |

| Symbol | Source |
|---|---|
| ● | HA-1, 4.5' - 5' |
| ▲ | HA-1, 7.5' - 8' |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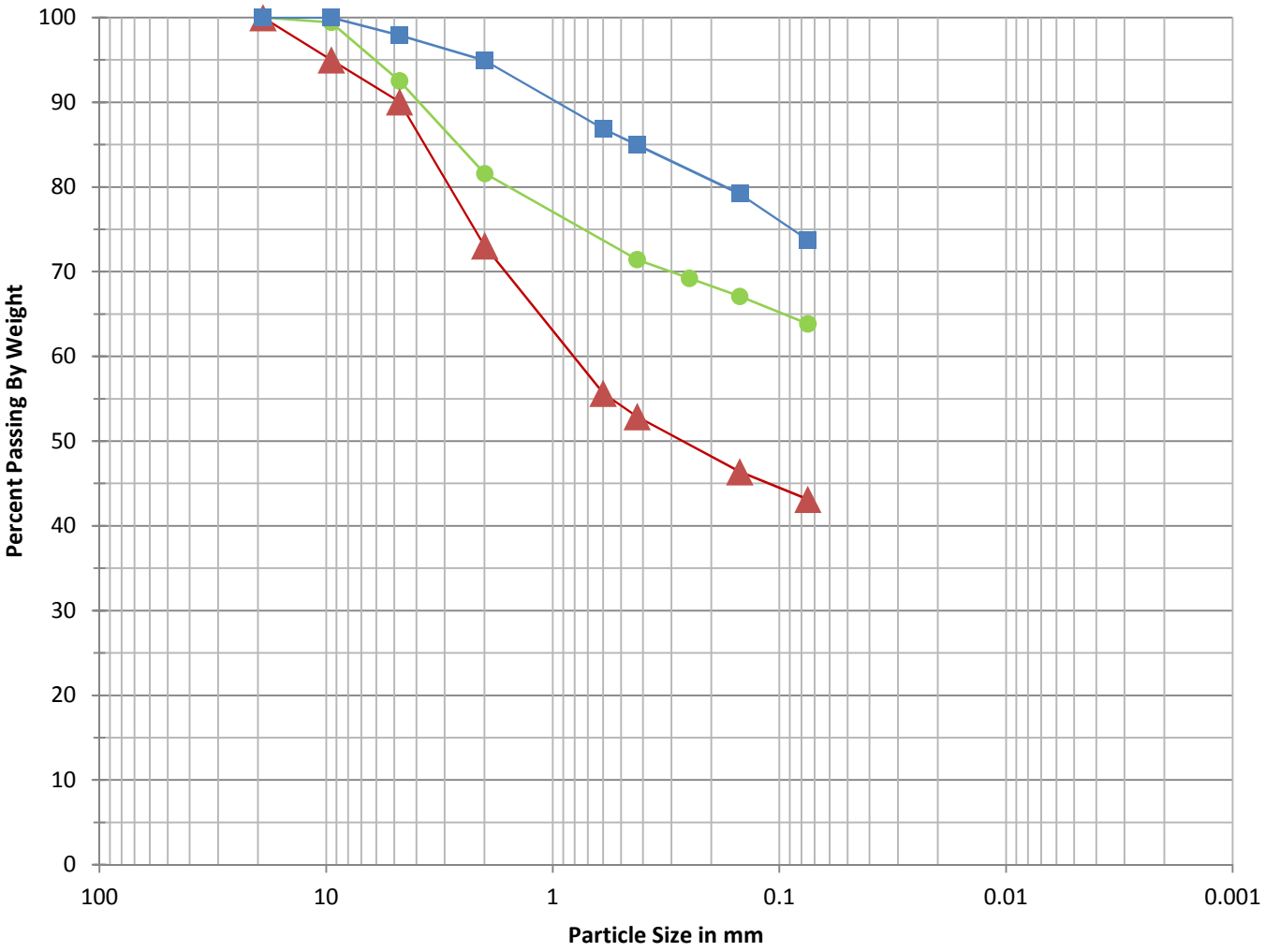
| Particle Size Analysis | Figure |
|---|---|
| Geotechnical Investigation<br>Sundblad<br>914 Everett Avenue, Oakland, California | 10 |

# Particle Size Analysis




| Gravel | | Sand | | | Silt | Clay |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | | |

| Symbol | Source |
|---|---|
| ● | HA-2, 4' - 4.5' |
| ▲ | HA-2, 6.5' |
| ■ | HA-3, 8.25'-8.75' |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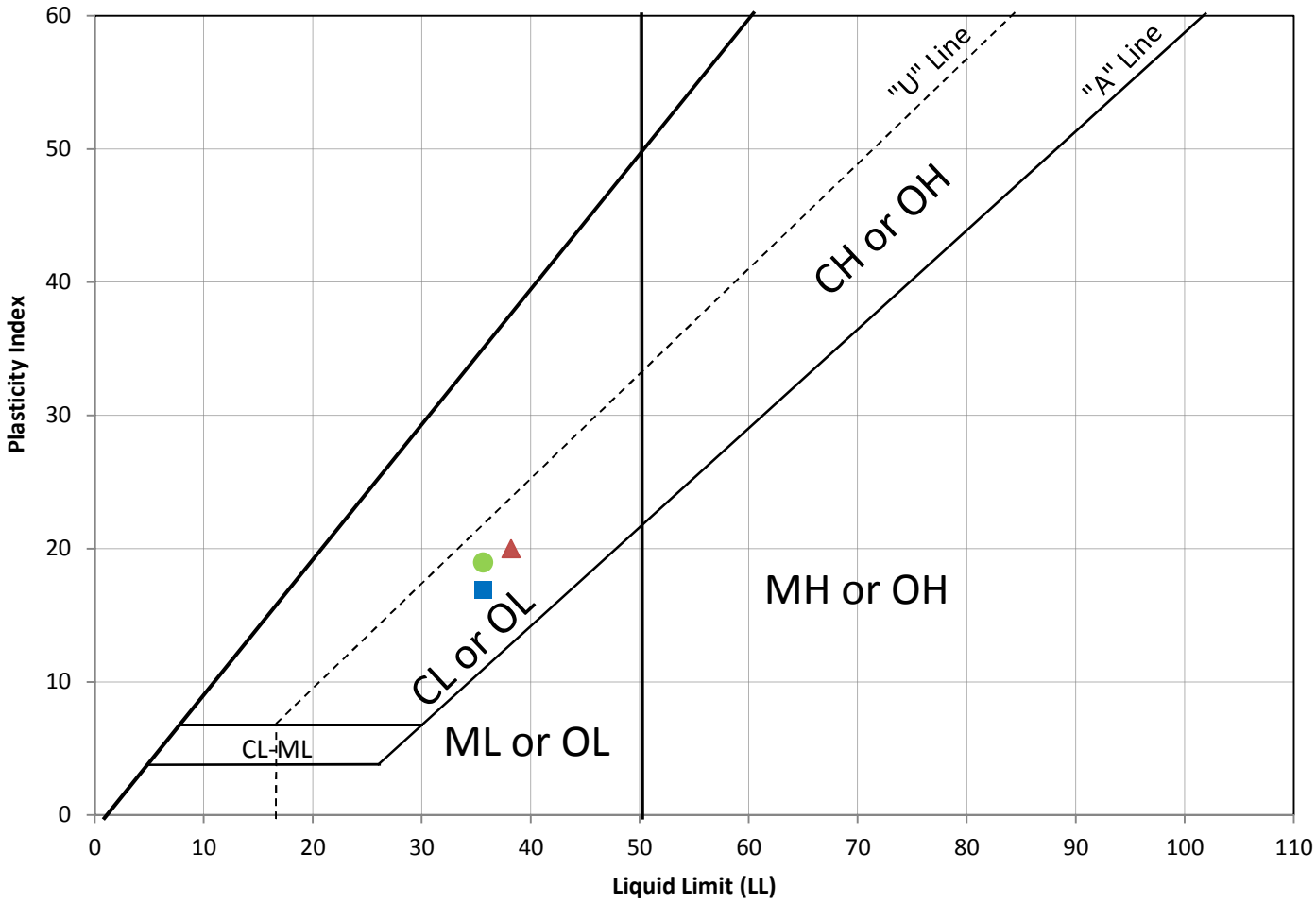
| Particle Size Analysis | Figure |
|---|---|
| Geotechnical Investigation<br>Sundblad<br>914 Everett Avenue, Oakland, California | 11 |

# Atterberg Limits




| Symbol | Classification & Source | Liquid Limit | Plastic Limit | Plasticity Index | % Passing #200 Sieve |
|---|---|---|---|---|---|
| ● | Dark Yellow Brown Sandy Lean Clay (CL), HA-1, 4.5'-5' | 36 | 17 | 19 | 72.7 |
| ▲ | Brownish Yellow Sandy Lean Clay (CL), HA-2, 4'-4.5' | 38 | 18 | 20 | 63.9 |
| ■ | Brownish Yellow Sandy Lean Clay (CL) HA-3, 8.25' - 8.75' | 36 | 19 | 17 | 73.7 |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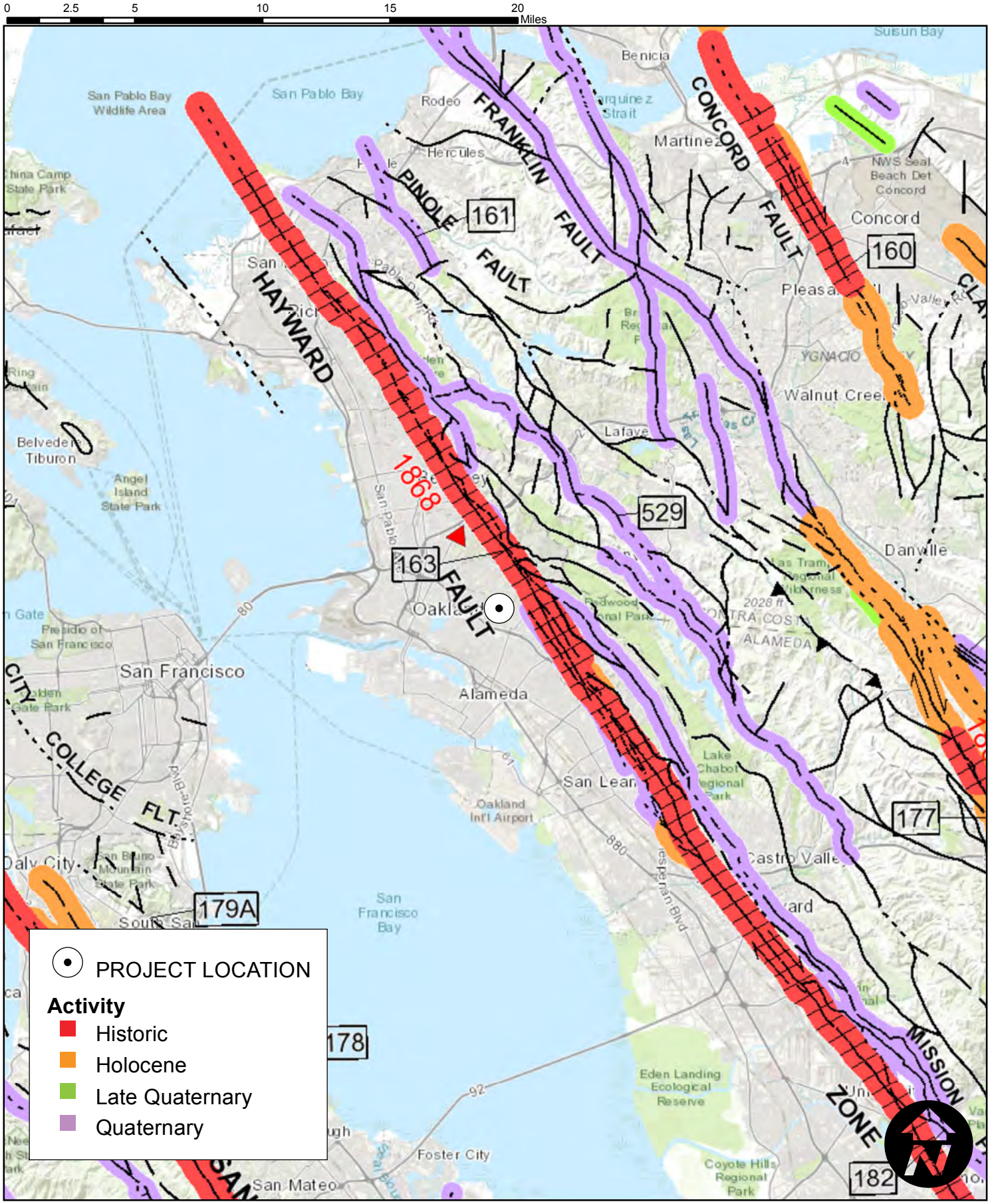
| Atterberg Limits (ASTM D4318) | Figure |
|---|---|
| Geotechnical Investigation<br>Sundblad<br>914 Everett Avenue, Oakland, CA | 12 |




PROJECT NO. 1900044

DRAWN BY TJC 4-18-2019

# FAULT MAP

*914 EVERETT AVENUE GEOTECHNICAL INVESTIGATION*
*OAKLAND, CA*

*SOURCE: CALIFORNIA GEOLOGICAL SURVEY (2010), MAPS OF THE LIQUIST PRIOLO FAULT ZONES*

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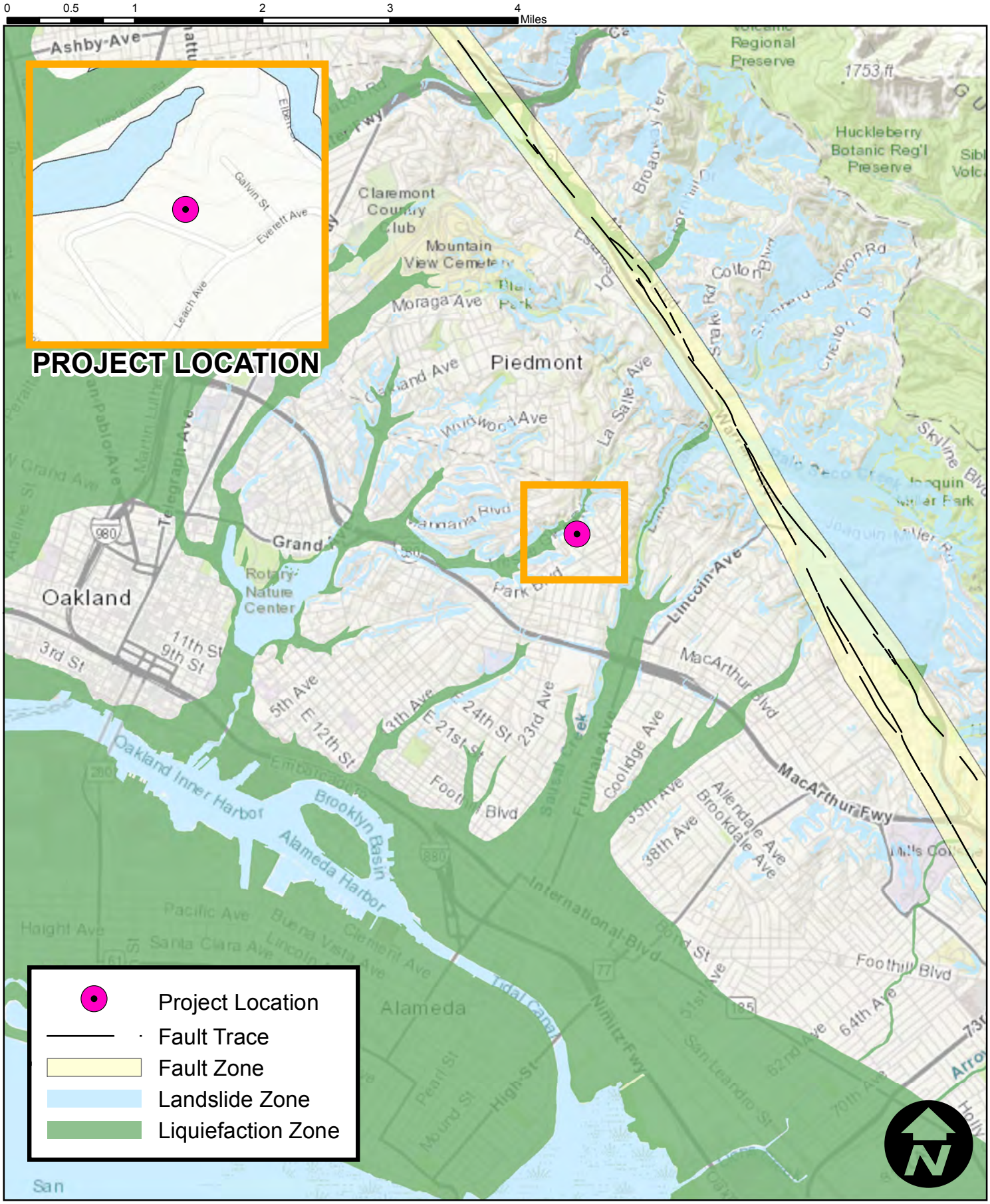
**FIGURE 13**




PROJECT NO. 1900044

DRAWN BY TJC 4-24-2019

# ALQUIST-PRIOLO FAULT ZONES

*914 EVERETT AVENUE GEOTECHNICAL INVESTIGATION*
*OAKLAND, CA*

*SOURCE: CGS, 2003, EARTHQUAKE ZONES OF REQUIRED INVESTIGATION OAKLAND EAST AND WEST*

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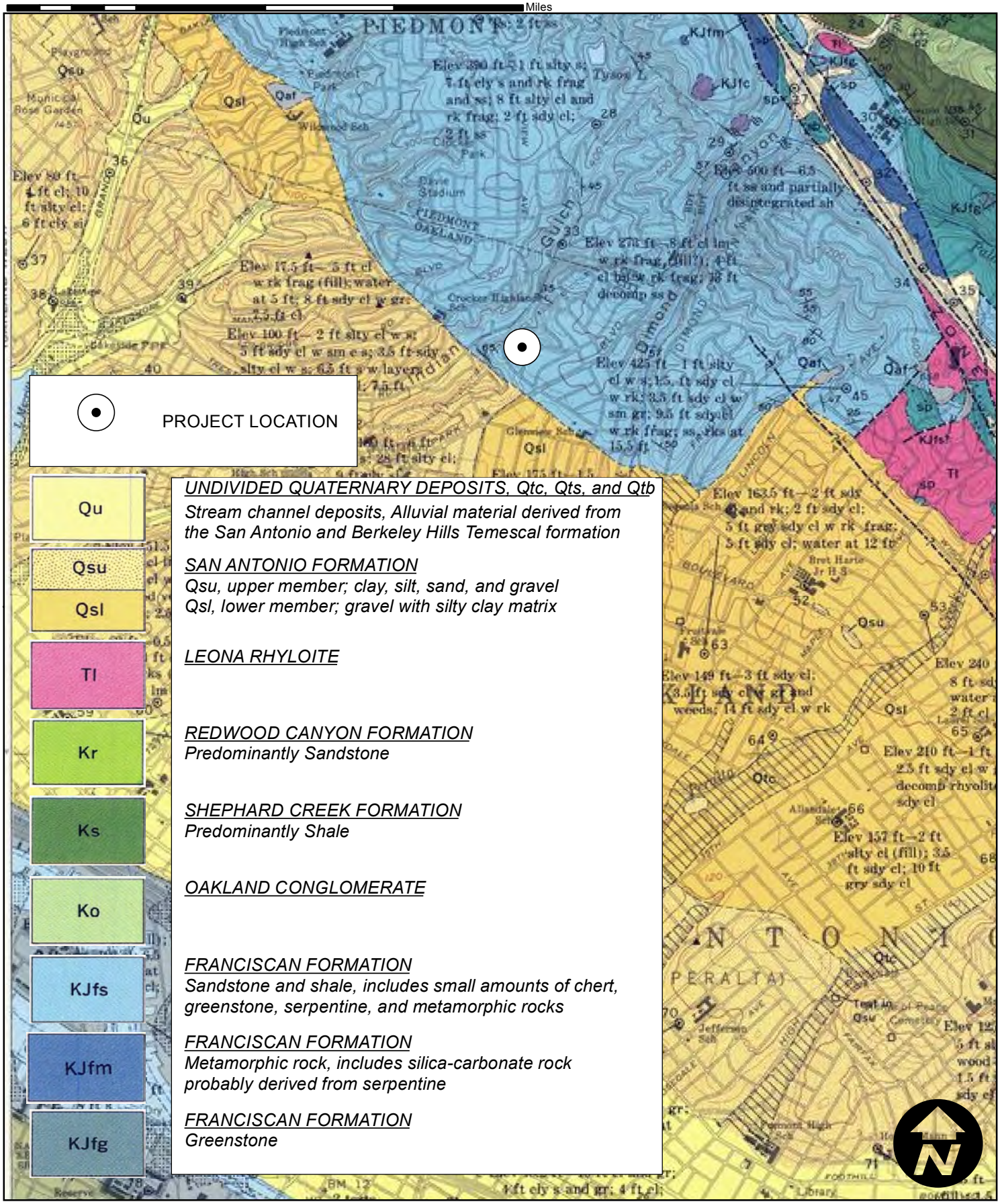
**FIGURE 14**




# GEOLOGY MAP

*914 EVERETT AVENUE GEOTECHNICAL INVESTIGATION*
*OAKLAND, CA*

*SOURCE: USGS, 1967, AREAL AND ENGINEERING GEOLOGY OF THE OAKLAND EAST QUADRANGLE, CALIFORNIA, GQ-769, RADBRUCH, DOROTHY H.*

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**FIGURE 15**