GEOTECHNICAL INVESTIGATION

3093 Broadway
Oakland, California

Prepared For:
3093 Broadway Holdings, L.L.C.
10877 Wilshire Boulevard, Suite 1200
Los Angeles, California 90024

Prepared By:
Langan Treadwell Rollo
555 Montgomery Street, Suite 1300
San Francisco, California 94111

REGISTERED PROFESSIONAL ENGINEER
NATHAN A. SHERWOOD
C 82679
EXP. 9/30/16
CIVIL
STATE OF CALIFORNIA

Nathan A. Sherwood

Nathan A. Sherwood, PE
Senior Staff Engineer

REGISTERED PROFESSIONAL ENGINEER
RICHARD D. RODGERS
NO. 732
EXP. 12/31/15
GEOTECHNICAL
STATE OF CALIFORNIA

Richard D. Rodgers

Richard D. Rodgers, GE
Managing Principal & Executive Vice President

2 July 2015
Langan Project No. 731637002

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL INVESTIGATION
3093 BROADWAY
Oakland, California

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed development at the 3093 Broadway project in Oakland. The project site is bound by Hawthorne Avenue to the north, Broadway to the east, Webster Street to the west, and a surface parking lot to the south, as shown on the Site Location Map, Figure 1. We previously performed a preliminary geotechnical investigation at the site and presented our preliminary conclusions and foundation design recommendations in a report dated 14 July 2014[1].

1.1 Existing Conditions

The 3093 Broadway site is a trapezoidal shaped parcel with an area of approximately 150,000 square feet. The site slopes down to the east; the drop in elevation from the northwest corner to the southeast corner is approximately 16 feet (from Elevation 68 feet to 52 feet[2]). In the east-west direction of the site, the site has an approximate plan dimension of 437 feet; in the north-south direction, the site has an approximate plan dimension of 347 feet.

The site is currently occupied by an abandoned, two-story concrete structure that was formerly a car dealership and a connected one-story garage, both of which are located along Hawthorne Avenue. There are two small, one-story structures in the middle of the site. Currently, the site is occupied by stored automobiles. There is an existing concrete retaining wall along Webster Street that is approximately 7 to 10 feet high.

---

[1] "Report of Preliminary Geotechnical Investigation and Consultation, Oakland, California" dated 14 July 2014

[2] Elevations are referenced to City of Oakland datum

1.2 Proposed Development

The proposed development, as shown on the Site Plan, Figure 2, includes demolition of the existing structures within the site except for the historic stucco façade section of the car dealership, located at the corner of Hawthorne Avenue and Broadway, as shown on the Site Plan. We understand the historic façade will be retained and incorporated into the construction of a seven-story mixed use commercial/residential building. The proposed plans[3] indicate the first floor will be used for parking and retail space. The finished floor elevation of the first floor ranges from Elevation 51.5 feet to Elevation 58 feet. The parking stalls will be at Elevation 53 feet. The remaining six levels will include residential units. Up to 16 feet of excavation will be required to achieve the desired finished floor elevation along the western side of the building where a podium is planned.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated 26 June 2014. The scope of our services consisted of reviewing available geologic and geotechnical data for the site and its vicinity, conducting a subsurface investigation, performing engineering analyses, and developing geotechnical design criteria for the proposed development.

Data acquired during our subsurface investigation, geotechnical laboratory testing results, and engineering analyses were used to develop geotechnical conclusions and parameters regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards including potential for ground rupture, liquefaction, lateral displacement, and seismic densification, as appropriate
- appropriate seismic hazard mitigation measures, as appropriate
- appropriate foundation type(s) for the proposed structure
- design criteria for the recommended foundation type(s)

---

[3] Elevations obtained from Sheet A121 from the "100% Design Development" drawings by Van Tilburg, Banvard & Soderbergh Architects, dated 22 May 2015.

- estimates of foundation settlements, including total and differential settlements
- lateral earth pressures for the design of below-grade basement walls and shoring systems
- inclination of temporary slopes
- site grading, including criteria for fill quality and compaction
- soil subgrade preparation
- floor slab support
- 2013 California Building Code (CBC) seismic design criteria
- construction considerations.

3.0 FIELD INVESTIGATION AND GEOTECHNICAL LABORATORY TESTING

As part of our preliminary field investigation, we performed four cone penetration tests (CPTs). To supplement available subsurface information and gain further site specific data, we drilled four borings for this field investigation. Site groundwater conditions were evaluated using water level readings from existing monitoring wells. The approximate locations of the CPTs, borings, and existing monitoring wells are shown on Figure 2. Prior to performing our field investigation, we obtained a soil boring permit from the Water Resources Wells Section of the Alameda County Department of Public Works Agency (ACDPWA), notified Underground Service Alert (USA), and retained a private underground utility locating service to check that locations of exploratory points were clear of existing utilities.

3.1 Borings

Between 25 and 26 August 2014, Pitcher Drilling Company of East Palo Alto, California, drilled four borings at the site, designated B-1 through B-4, to depths of 51.5 feet below the ground surface (bgs), using truck-mounted, rotary wash drilling equipment. The borings were drilled under the direction of our field engineer who logged the soil encountered and obtained representative samples for visual classification and laboratory testing. Logs of the borings are presented on Figures A-1a through A-4b in Appendix A. The soil encountered in the borings was classified in accordance with the Classification Chart, presented on Figure A-5 in Appendix A.

Soil samples were obtained using two driven split-barrel samplers. The sampler types are described below:

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

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

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy and are shown on the boring logs. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

Upon completion, the boreholes were backfilled with grout consisting of cement, bentonite, and water in accordance with the requirements of the ACDPWA. The grouting was completed under the intermittent observation of an ACDPWA Inspector. The soil cuttings and drilling fluid from the borings were placed in 55-gallon drums which were stored temporarily at the site, tested, and were transported off-site for proper disposal.

3.2 Cone Penetration Tests

On 20 June 2014, Gregg Drilling and Testing, Inc. of Martinez, California, advanced four CPTs at the site, designated CPT-1 through CPT-4, to depths of 50 feet bgs.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe, with a projected area of 10 square centimeters, into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges or load cells within the cone continuously measured the cone tip resistance and frictional resistance during the entire depth of each probing. Accumulated data is processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance, side friction, friction ratio, interpreted SPT N-values, and interpreted soil classification, are presented in Appendix B on Figures B-1 through B-5. Soil types were determined using the classification chart shown on Figure B-6.

A pore pressure dissipation test was performed in CPT-3; the results are presented on Figure B-6.

Soil cuttings were not generated during the CPTs. Upon testing completion the CPT holes were backfilled with cement grout in accordance with the requirements of ACDPWA.

3.3 Geotechnical Laboratory Testing

Soil samples obtained from the borings were examined in the office to confirm the field classifications and representative samples were selected for geotechnical laboratory testing. Laboratory tests were selected to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, gradation, fines content, plasticity, and shear strength, as appropriate. Results of the laboratory tests are shown on the boring logs and are included in Appendix C.

3.4 Previous Site Investigations and Existing Monitoring Wells

Environmental studies of the site were performed in the past to evaluate the presence of hazardous substances in the soil and groundwater. Although the associated site investigations were not for engineering purposes, the logs of the monitoring wells and CPTs available from these investigations contain some geotechnical data and are useful in defining the subsurface profile. Subsurface Consultants installed monitoring wells in 1990 and 1991 and logs of eight wells, MW-1, 2, 3, 4, 5, 6, 7, and 9 were provided for our review. Pangea Environmental Services, Inc. studied the site in 2005 and developed a detailed subsurface profile through the northern portion of the site. The information was used by us to supplement our own geotechnical investigation.

3.5 Soil Corrosivity Testing

Corrosive soil can adversely affect underground utilities and foundation elements. To measure the corrosion potential of the soil, laboratory testing was performed on two samples retrieved at 5 feet bgs. The corrosivity of the soil samples was evaluated by Cerco Analytical of Concord, California, using ASTM Test Methods. The laboratory corrosion test results and corrosivity evaluation are presented in Appendix D.

4.0 SUBSURFACE CONDITIONS

The results of our field investigation indicate, in general, the site is underlain by fill and interlayered dense to very dense clayey sand and very stiff to hard silty to sandy clay. A summary of the site subsurface conditions is presented below.

Fill

The site is blanketed by about 2.5 to 5 feet of sandy clay fill with gravel and trace amounts of wood and red brick fragments.

Clay, Silt, and Sand

The fill is underlain by dense to very dense clayey sand or very stiff to hard clay, silty to sandy clay, and silt. The clay and sand layers vary from 5 to 15 feet in thickness. Gravel was encountered within the clay and sand layers intermittently with depth. The upper 5 to 10 feet of the clay deposit is very stiff.

Groundwater

The groundwater level was obscured during our August 2014 investigation, due to the rotary wash drilling method. However, based on readings taken by Langan Treadwell Rollo on 20 May

2014 in existing monitoring wells, the groundwater level appears to be near Elevation 31 feet along the eastern side of the site (Broadway). The pore water pressure dissipation test in CPT-3, near the western edge of the site, indicates the groundwater level is at approximately Elevation 33 feet. Therefore, we estimate the groundwater level is generally near Elevation 31 to 33 feet across the site. The groundwater level at the site may fluctuate with rainfall.

5.0 REGIONAL GEOLOGY AND SEISMICITY

5.1 Regional Geology

The site is within the Coast Ranges geomorphic province, typically characterized by northwest-southeast trending mountain ridges and valleys; these developed and are controlled by folding and faulting that resulted from the collision of the Farallon and North American plates, and subsequent shear along the San Andreas Fault system.

According to the map of Quaternary Geology of Alameda County and Surrounding Areas (Helly and Graymer, 1997), the subject site is underlain by Holocene-age (approximately 11,000 years old to present) alluvial fan deposits, as shown on the Regional Geologic Map of West Oakland, Figure 3. These deposits generally consist of layers of variable composition containing varying amounts of overconsolidated clay, silt, sand, and gravels.

5.2 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, San Gregorio, Calaveras, and Hayward Faults. These and other faults of the region are shown on Figure 4. For each of the active faults within 50 kilometers of the site, the distance from the site and estimated mean characteristic Moment magnitude, M_w [2007 Working Group on California Earthquake Probabilities (WGCEP) (2007) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1

Regional Faults and Seismicity

Fault Name	Approximate Distance from Fault (kilometers)	Direction from Site	Mean Characteristic Moment Magnitude, M_w
Total Hayward	4	East	7.00
Total Hayward-Rodgers Creek	4	East	7.33
Mount Diablo Thrust	21	East	6.70
Total Calaveras	22	East	7.03
N. San Andreas - Peninsula	25	West	7.23
N. San Andreas (1906 event)	25	West	8.05
Green Valley Connected	26	East	6.80
N. San Andreas - North Coast	28	West	7.51
San Gregorio Connected	31	West	7.50
Rodgers Creek	34	Northwest	7.07
Greenville Connected	38	East	7.00
West Napa	39	North	6.70
Monte Vista-Shannon	42	South	6.50
Great Valley 5, Pittsburg Kirby Hills	43	East	6.70

The greater San Francisco Bay Area is recognized by geologists and seismologists as one of the most active seismic regions in the United States. The three major faults that pass through the Bay Area in a northwest direction have produced approximately 12 earthquakes per century strong enough to cause structural damage. The earthquakes are generated within the San Andreas Fault system, a major shear zone in the earth's crust that extends approximately 700 miles throughout the length of California. Local fault zones associated with the San Andreas Fault system include the San Andreas, Total Hayward-Rodgers Creek, and Calaveras fault zones, which are 25 kilometers to the west, 4 kilometers to the east, and 22 kilometers to the east, respectively.

Figure 4 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (refer to Figure 5) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 430 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 93 kilometers from the site. The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 44 kilometers north of the site, with a M_w of 6.0.

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

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

TABLE 2

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

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

6.0 SEISMIC HAZARDS

The site is in a seismically active area and will be subject to strong shaking during a major earthquake on a nearby fault. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5], seismic densification[6], and fault rupture.

Each of these conditions has been evaluated based on our literature review, field investigation, and studies, and is discussed in this section.

---

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

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

[6] Seismically-induced densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

6.1 Liquefaction and Lateral Spreading

If a soil liquefies during an earthquake, it experiences a significant temporary loss of strength. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. The site is not within a designated liquefaction hazard zone as mapped by the California Division of Mines and Geology (CDMG) prepared in accordance with the Seismic Hazards Mapping Act and adapted by the City of Oakland in 2003, as shown on the Regional Seismic Hazards Map, Figure 6. Based on our studies and considering the dense to very dense clayey sands and medium stiff to hard silty to sandy clays that underlie the site, we conclude the potential for liquefaction and lateral spreading at the site is low.

6.2 Seismic Densification

Seismically-induced densification can occur during strong ground shaking in loose, clean granular deposits above the groundwater level, resulting in ground surface settlement. The sandy clay fill and dense to very dense clayey sand above the groundwater table are not susceptible to seismically-induced densification during a major earthquake on a nearby fault.

6.3 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Therefore, we conclude the risk of surface faulting and consequent secondary ground failure is low.

7.0 DISCUSSION AND CONCLUSIONS

On the basis of the results of our investigation and review of the available subsurface information, we conclude the project is feasible from a geotechnical standpoint.

The primary geotechnical issues affecting the selection of safe, economical foundation systems are:

- the presence of environmental remediation borings
- the presence and depth of non-engineered fill
- need for shoring during construction
- presence of buried elements from the demolished buildings
- presence of potentially corrosive soil.

Our conclusions and recommendations regarding these issues are presented in the following sections.

7.1 Environmental Remediation Borings

As detailed in Langan's "Feasibility Study and Corrective Action Plan" report[7], three former underground storage tanks (UST) were removed beneath the Hawthorne Avenue sidewalk in 1989. Subsequent environmental sampling and testing has indicated that the USTs leached petroleum hydrocarbons, volatile organic compounds, and heavy metals into the site groundwater.

To remediate the presence of hazardous contaminants in the groundwater, Langan's environmental team plans to install 51 remediation borings in the northern section of the site (as shown on Figure 7). We understand the borings will be installed in rows perpendicular to the direction of groundwater flow to create a bioremediation treatment zone. The borings will be drilled with 12-inch-diameter, hollow-stem augers to 15 feet below the groundwater table (approximately Elevation 18 to 15 feet). The lower 15 feet of each boring will be filled with a

---

[7] Langan Treadwell Rollo, "Feasibility Study and Corrective Action Plan" report, dated 21 May 2015.

mixture of 50 percent sand and 50 percent gypsum pellets. The unsaturated portions of the borings will be grouted up to the ground surface.

In May 2015, Langan installed seven pilot study remediation borings at the site. The purpose of the pilot study was to gather information needed to refine the final remediation boring design. We have assessed the geotechnical impacts of the seven installed pilot borings in conjunction with the final 51 planned borings.

We understand the gypsum pellets will slowly dissolve over several years. Therefore, we anticipate void spaces will be created in each remediation boring due to the dissolution of the gypsum. If the voids collapse, ground settlement that could adversely affect the performance of the building foundation could result. Based on the estimated volume of gypsum dissolved per boring, we estimate that ground surface settlement could occur over a maximum distance of 14 feet measured from the centerline of each boring (this distance is herein referenced as the "zone of influence"). Spread footings within the zone of influence would be affected by ground settlement caused by the borings. To support building loads, we understand the project team has elected to bridge over the remediation borings with rigid slabs. Conventional spread footings will be used in areas outside of the zone of influence.

7.2 Shallow Foundations

7.2.1 Spread Footings

Conventional spread footings can be used outside the zone of influence of the remediation borings. Spread footings should extend through the fill and bear directly on native dense clayey sand or native very stiff sandy clay. Both the native dense clayey sand or native very stiff clay are considered suitable bearing layers for the spread footings. Considering the variability of fill and lack of information regarding its placement and compaction, we conclude the fill is not suitable for support of the proposed structure.

We estimate total settlement of spread footings supported on the bearing layers should be about ¼ to ½ inch. Differential settlement between adjacent columns should be about ¼ inch.

7.2.2 Rigid Slabs

Based on discussions with Hoogerwerf Engineering Group, Inc. (project structural engineer), a rigid slab will span over the cluster of the pilot study and planned environmental remediation borings near the former USTs (Area A). The magnitude of differential settlement is difficult to estimate in Area A, given the tightly spaced configuration and number of borings. However, we estimate ground settlement due to the remediation borings in this area will be on the order of 1¾ inches.

Where the remaining remediation borings are planned (Areas B and C), we understand a rigid slab-on-grade (additional reinforcing steel) will be used to span over the borings. These three areas are presented on Figure 7.

We estimate that as the loads redistribute over the rigid slabs; the amount of settlement will depend on the stiffness of the foundation system and its ability to distribute load across the rigid slabs.

7.3 Floor Slabs

In general, native dense clayey sand or native very stiff sandy clay will be exposed at finished floor subgrade areas for the proposed structure. Where the building slab is bearing on 1) native dense clayey sand or native stiff sandy clay or 2) engineered fill, the building slab may be supported at grade. If undocumented fill is exposed at subgrade, it should be removed in its entirety and placed back as engineered fill per the fill quality and compaction recommendations outlined in Section 8.1.4.

7.4 Excavation

The entire site is covered by asphaltic pavement, which will be removed. Fill, clay, silt, and sand below the pavement and existing structures can be excavated with conventional earth-moving equipment. We anticipate material that will be encountered during excavation include brick debris, foundations, and utilities. Where foundation remnants are encountered, the use of a hoe-ram may be required for removal. Existing building elements should be removed in their entirety within the proposed building footprint.

Environmental studies conducted by Langan and others indicate the excavated fill will contain concentrations of heavy metals and petroleum hydrocarbons. As part of the planning process for the new development, a soil management plan (SMP) and a health and safety (H&S) plan will be required prior to any site redevelopment activities because of the petroleum hydrocarbons detected at the site. The SMP should provide recommended measures to mitigate the long-term environmental or health and safety risks caused by the presence of petroleum hydrocarbons in the soil, and to deal with residual petroleum hydrocarbons that may be found under the slab or in piping, resulting from discharges to drains both inside and outside the buildings. The SMP should also contain contingency plans to be implemented during soil excavation if unanticipated hazardous materials are encountered. The H&S plan should outline proper soil handling procedures and health and safety requirements to minimize worker and public exposure to hazardous materials during construction.

7.5 Expansive Soil

Expansive soils are those that shrink or swell significantly with changes in moisture content[8]. The clay content and porosity of the soil also influence the change in volume. The shrinking and swelling caused by expansive clay-rich soil often results in damage to overlying structures. Atterberg Limits tests on samples of the soil near the surface indicate these materials have low expansion potential.

7.6 Temporary Shoring

The excavation for the below-grade parking area will extend to about 16 feet bgs (up to 26 feet below Webster Street). Because of space restrictions, the excavation should be shored. The primary considerations related to the selection of the shoring system are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures
- presence of fill which contains brick and wood fragments and potential rubble
- presence of old foundations and footings
- ease of installation
- proper installation of the shoring system to reduce the potential for ground movement
- cost.

On the basis of our understanding of the subsurface conditions and our experience with similar projects, we conclude a soldier-pile-and-lagging system is a feasible shoring system for this

---

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

project. Where the excavation is shallow, a cantilever shoring system may be feasible; where too deep to be cantilevered, the shoring can be tied-back. Tiebacks installed under public streets will require encroachment permits.

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

A soldier-pile-and-lagging system usually consists of steel beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds. During excavation, it may be necessary to install lagging boards with every one- or two-foot cut to prevent caving of exposed soil. If voids are created behind lagging boards, they should be filled with cement slurry prior to proceeding with excavation.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. The contractor should install surveying points to monitor the movement of shoring and settlement of adjacent structures. The monitoring should provide timely data which can be used to modify the shoring system, if needed.

7.7 Underpinning

The proposed excavation will extend about 11 to 13 feet below the bottom of the foundation supporting the existing concrete retaining wall along Webster Street. Therefore, underpinning of the wall will be required if the existing wall is to remain. Steel piles installed in slant-drilled shafts or intermittent hand-excavated piers may be used to underpin the wall. The excavation face between the underpinning piles/piers should be retained using lagging provided the existing footing can span between piles/piers. The underpinning piles/piers should be designed to resist vertical retaining wall loads, vertical tieback loads (if tiebacks are used), traffic surcharge, and lateral earth pressures.

8.0 RECOMMENDATIONS

We conclude the site can be developed as planned, from a geotechnical standpoint, provided our recommendations are incorporated into the evaluation, design and contract documents, and are implemented during construction. Recommendations for site preparation and grading, foundation design, slab-on-grade floors, shoring, below-grade walls, and seismic design are presented in this section.

8.1 Site Preparation and Grading

Prior to grading operations, existing building elements should be demolished and removed from the areas to receive the proposed improvements. Existing utility lines beneath the building may be abandoned in place or removed. In general, all existing lines within about six feet of existing grade should be removed to minimize interference with new construction. All remaining utilities below the proposed buildings may be abandoned in place provided they will not impact future utilities or building foundations. This section presents our recommendations for site preparation and grading.

8.1.1 Site Clearing

All concrete and asphalt pavements and other existing improvements within the areas to be developed should be removed during site demolition. Existing building foundations and below-grade walls should be entirely removed within five feet below final subgrade in the new building areas, and to a depth of at least three feet in areas outside the new building footprint.

Where practical, existing utilities to be abandoned should be removed. If pipes are too deep to be removed economically, we recommend they be filled with cement and sand grout or equivalent material that will prevent collapse of the pipe in the future.

8.1.2 Temporary Slopes

Excavations deeper than five feet that will be entered by workers should be shored or sloped for safety in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). Inclinations of temporary slopes should not exceed those specified in local, state or federal safety regulations. As a minimum, the requirements of the current OSHA Health and Safety Standards for Excavations (29 CFR Part 1926) should be followed. The Contractor should determine temporary slope inclinations based on the subsurface conditions exposed at the time of construction. However, temporary slopes should be no steeper than 2:1 (horizontal:vertical) in sandy soil, and 1:1 in clay. Slopes should be designed to be flatter during the wet season.

Temporary slopes should not be open for an extended period of time. If temporary slopes are open for extended periods of time, exposure to weathering and rain could result in sloughing and erosion.

All vehicles and other surcharge loads should be kept at least 10 feet away from the top of temporary slopes and the slopes should be protected from either excessive drying or saturation during construction.

8.1.3 Subgrade Preparation

The soil surface exposed at subgrade should be smooth and non-yielding. Soft or loose subgrade soil should be reworked and compacted; within building areas unsuitable soil should be removed and the excavation backfilled with lean concrete. Excavations for removal of existing foundation elements should be backfilled with engineered fill.

Subgrades for slabs-on-grade, flatwork, and pavement areas exposed by stripping and/or excavation should be:

- scarified to a minimum depth of six inches
- moisture conditioned to near optimum

- compacted to at least 90 percent relative compaction[9]

8.1.4 Fill Material and Compaction

We anticipate fill placement at the site will consist primarily of backfill for utility trenches and localized backfilling around basement walls, and preparation of subgrade for new sidewalks. Excavated on-site soil is suitable for reuse as fill or backfill provided it meets the requirements given below for general fill. All materials to be used as fill should meet the following requirements:

- be free of organic material
- contain no rocks or lumps larger than three inches in greatest dimension
- have a low expansion potential (defined by a liquid limit of less than 40 and a plasticity index lower than 12)
- be non-corrosive and non-hazardous

Fill and backfill should be placed in lifts not exceeding eight inches in loose thickness and compacted to at least 90 percent relative compaction. Fill beneath pavements and fill deeper than five feet or containing less than 10 percent fines, should be compacted to at least 95 percent relative compaction. During construction, we should check that the on-site and any proposed import material are suitable for use as fill.

Wall backfill should be compacted using light compaction equipment. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

If backfill and compaction is planned adjacent to the historic stucco façade, we recommend the fill be compacted by tamping (hand-compaction) to avoid damaging the façade with vibrations.

8.1.5 Utilities and Utility Backfill

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench

---

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

excavations should be shored and braced to prevent cave-ins and/or in accordance with safety regulations.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped to achieve the minimum relative compaction requirements given above for backfill. Trench backfill should be compacted as recommended in Section 8.1.4. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements resulting in damage to the pavement section.

8.2 Shallow Foundations

For foundation areas not affected by the environmental remediation borings, recommendations for the design of spread footings are presented in Section 8.2.1. For foundation areas affected by the environmental remediation borings (Areas A, B, and C shown on Figure 7), recommendations for the design of rigid slabs are presented in Section 8.2.2. Additional foundation recommendations are presented in Section 8.2.3.

8.2.1 Spread Footings

Isolated and/or continuous spread footings for the new structure not affected by the environmental remediation borings should bear on native dense clayey sand or native very stiff sandy clay. Shallow foundations bearing on clayey sand or sandy clay may be designed using an allowable bearing pressure of 6,000 pounds per square foot (psf) for dead plus live loads, with a one-third increase for total loads, including wind and/or seismic loads. Continuous and isolated footings should be at least 18 and 24 inches wide, respectively. Footings should be embedded at least 18 inches below the lowest adjacent soil subgrade.

For the modulus of subgrade reaction, we recommend using a value of 83 pounds per cubic inch (pci). This modulus of subgrade reaction is based on an allowable bearing pressure of 6,000 psf.

Lateral loads on footings can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along their bottoms. Passive resistance may be calculated using lateral pressures of 1,680 psf. The upper foot of soil should be ignored unless confined by a concrete slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.3. The passive resistance and base friction may be used in combination without reduction. The passive pressure and frictional resistance value includes a factor of safety of about 1.5. Excavations that extend below the level of adjacent building floor slabs or foundations should provide lateral and vertical support for the slabs and foundations to prevent movement.

8.2.2 Rigid Slabs (Areas A, B, and C)

For design using the modulus of subgrade reaction method, we recommend a value of 40 pci. This modulus is based on an allowable bearing pressure of 6,000 psf.

The edges of the rigid slabs should extend at least 14 feet from the centerline of the outermost environmental remediation boring that is clustered in a group. The recommended extents of the rigid slabs in Areas A, B, and C are shown on Figure 7. Lateral loads on rigid slabs can be resisted by passive resistance acting against the vertical faces of the slabs. Passive resistance may be calculated using lateral pressures of 1,680 psf. Base friction should not be counted on to resist lateral loads.

8.2.3 Additional Recommendations

If weak soil is encountered in the bottom of foundation excavations, the material should be removed and replaced with lean concrete. Foundations adjacent to utility trenches (or other foundations) should bear below an imaginary 2:1 plane projected upward from the bottom edge of the utility trench (or adjacent foundation).

Foundation excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. We should check foundation excavations prior to placement of reinforcing steel.

Where adjacent finished floor elevations differ, the upper foundations will impose pressure on the adjacent lower walls and foundations. Either the lower walls and foundations should be designed to accommodate these additional pressures or the foundations should bear below an imaginary 1.5:1 plane projected upwards from the bottom edge of the adjacent foundation. If the option to design the walls and foundations to accommodate additional pressures is chosen, we should be consulted to provide the surcharge pressures induced by the upper foundations.

8.3 Slab-on-Grade Floors

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

The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The particle size of the gravel/crushed rock should meet the gradation requirements presented in Table 3.

TABLE 3

Gradation Requirements for Capillary Moisture Break

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

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. The slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

8.4 Concrete (Impermeable) Pavers

We understand a paseo will be constructed along the southern side of the building and will be constructed with impermeable concrete pavers. We recommend the pavers consist of fully dentated, interlocking shapes at least 3.15 inches (80 millimeters) thick. The pavers should be placed on at least 1-inch-thick sand leveling course underlain by at least 4 inches of Class 2 aggregate base (AB) or a concrete slab. If a concrete slab is used in lieu of the Class 2 AB, we recommend it be at least five-inches-thick. The subgrade and aggregate base beneath the pavers should be compacted in accordance with the recommendations presented in Section 8.1.4. Installation of the pavers, including use of a vibratory plate to seat the pavers, should be performed in accordance with the manufacturer's recommendations.

8.5 Shoring

A soldier-pile-and-lagging system is an acceptable method to retain the excavation. Depending on the depth of the excavation, the shoring can be either cantilevered or tied-back.

If traffic occurs within 10 feet of the shoring, a uniform surcharge load of 100 psf should be applied to the top 10 feet of the shoring. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge.

Shoring should be designed and installed to allow negligible movement of the adjacent improvements. The increase in pressure due to construction equipment, stockpiles and/or adjacent building loads should be computed after the surcharge loads are known. The shoring system should be designed by a licensed engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for designing the shoring to comply with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

8.5.1 Soldier-Pile-and-Lagging System

Recommended lateral earth pressures for cantilever shoring are presented on Figure 8. For a tied-back shoring system, the recommended lateral earth pressures are shown on Figure 9. Recommended shoring pressures do not account for the presence of groundwater, since the groundwater level is assumed to be about 19 to 21 feet below the bottom of the excavation. The earth pressures shown on Figures 8 and 9 also do not include surcharge pressures from adjacent structures; if necessary; such pressures should be determined and added to Figures 8 and 9.

The soldier piles should be sufficiently embedded below the bottom of the excavation to achieve lateral stability and resist the downward loading of the tiebacks, where they exist. Recommendations for computing penetration depth of soldier piles are presented in Section 8.5.1.2.

8.5.1.1 Tieback Design Criteria and Installation Procedure

Temporary tiebacks may be used to restrain the shoring. The vertical load from the tiebacks should be accounted for in the design of the soldier piles. Design criteria for temporary tiebacks are presented on Figure 9. Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks should have a minimum unbonded length of 15 feet. All tiebacks should have a minimum bonded length of 15 feet and spaced at least four feet on center. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks. To reduce caving potential, we recommend using a smooth-cased method (such as a Klemm rig) for tieback installation.

Allowable capacities of the tiebacks will depend upon the drilling method, drillhole diameter, grout pressure, and workmanship. The friction value used for design of tiebacks will depend on the installation procedure. For estimating purposes, we recommend using the allowable skin friction values for native soil and fill presented on Figure 9. These values include a factor of safety of 1.5. The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with his installation method. The computed bond length should be confirmed by a performance- and proof-testing program under the observation of an engineer experienced in this type of work. Replacement tiebacks should be installed for tiebacks that fail the load test.

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. Recommendations for tieback testing are presented in Section 8.5.1.3. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

8.5.1.2 Penetration Depth of Soldier Piles

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient axial capacity to support the vertical load component of the tiebacks and the vertical load acting on the piles, if any. Axial loads on the shoring can be resisted by skin friction along the sides of the soldier piles below the bottom of the excavation. We recommend using an allowable skin friction value of 460 psf to compute axial capacities of the shoring, which includes a factor of safety of 1.5. End bearing should be neglected. If water is present in the shaft, concrete should be placed using a tremie system.

8.5.1.3 Tieback Testing

Each tieback should be tested. The maximum test load should not exceed 80 percent of the yield strength of the tendons or bars. The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing.

Performance Tests

The performance testing will be used to determine the load carrying capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established.

In the performance test, the load applied to the tiebacks and its movement is measured during several cycles of incremental loading and unloading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1, 2, 3, 6 and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

Proof Tests

A proof test is used to measure the total movement of the tiebacks during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the load should be maintained and the observation is continued until the creep rate can be determined. The proof test results should be compared to the performance test results. Any significant variation from the performance test results will require performance testing on the anchor.

Acceptance Criteria

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

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with a creep rate that does not exceed 0.08 inch/log cycle of time, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

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

8.5.2 Underpinning

If the existing concrete retaining wall along Webster Street is left in place, it should be underpinned. We were provided the 6 April 2015 pot-holing results along the Webster Street and Hawthorne Avenue retaining walls. The results indicate that the edge of the retaining wall foundation along Webster Street intrudes into the site approximately 48 inches and the retaining wall foundation along Hawthorne Avenue (located along the northern side of the car dealership) intrudes into the site approximately 43 inches. We understand the retaining wall foundation along Hawthorne Avenue will be removed. Steel piles installed in slant-drilled shafts or intermittent hand-excavated piers may be used to underpin the existing Webster Street wall foundation. If intermittent piles or piers are used, the excavation face between the underpinning piles/piers should be retained using lagging provided the existing footing can span between piles/piers.

The underpinning piles/piers should be designed to support the vertical retaining wall loads, vertical tieback loads (if tiebacks are used), traffic surcharge, and lateral earth pressures. The excavation cut along Webster Street will likely require multiple levels of tiebacks to restrain movement. The lateral earth pressures should be determined using the diagram presented on Figure 9. Lateral pressures may be resisted by passive resistance against the embedded portion of the piles or piers. Passive resistance is also presented on Figure 9. This value includes a safety factor of about 1.5.

Underpinning pier/piles should extend at least 2 feet below the bottom of the planned excavation. Recommendations for computing penetration depth of soldier piles are presented in Section 8.5.1.2.

Hand-excavated piers should be designed based on end bearing only. The allowable bearing pressures for spread footings (Section 8.2.1) can be used for designing underpinning piers.

To reduce movement and provide adequate foundation support during installation of the underpinning piers, adjacent piers should not be excavated concurrently. We recommend underpinning piers be preloaded prior to dry packing to reduce settlement as the foundation load is transferred to the piers.

8.6 Permanent Below-Grade Walls

Basement walls will be required for the first floor parking garage level along Webster Street and the upslope section of Hawthorne Avenue. We recommend interior building retaining walls be designed to resist lateral pressures imposed by the adjacent soil and any surcharge loads. Because the site is in a seismically active area, the design should also be checked for seismic conditions. Under seismic loading conditions, there will be an added seismic increment. We used the procedures outlined in Sitar et al. (2012) to compute the seismic active pressure. Table 4 presents the active, at-rest, and total pressures (active plus seismic pressure increment) for soil with level backfill for drained and undrained conditions. All parameters are presented as equivalent fluid weights (triangular distribution).

TABLE 4

Below Grade Wall Design Earth Pressures
(Drained Conditions)

Retained Material	Static Conditions		Seismic Conditions[1]	
	Unrestrained Walls Active Condition (pcf)	Restrained Walls At-Rest Condition (pcf)	Total Pressure (Active Plus Seismic Pressure Increment) (pcf)	
			DE[2]	MCE[3]
Clayey Sand and Fill	37	58	70	90

Notes:

1. The more critical condition of either at-rest pressure (static condition) or active pressure plus a seismic pressure increment (seismic condition) should be checked.
2. DE = Design Earthquake
3. MCE = Maximum Considered Earthquake

If surcharge loads occur above an imaginary 45-degree line (from the horizontal) projected up from the bottom of a below-grade wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the added pressure on a case-by-case basis. Where truck traffic will pass within 10 feet of below-grade walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot applied in the upper 10 feet of the walls. Minor vertical line loads other than traffic loads may be modeled as a uniform lateral pressure using 50 percent of the vertical load in the upper 10 feet of the walls.

The recommended lateral earth pressures assume drainage will be installed behind walls and below the slab. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend to a four-inch-diameter perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) and wrapped in filter fabric (Mirafi 140N or equivalent). The pipe should be connected to a sump, where water can be removed by a pump, as appropriate. We should check the manufacturer’s specifications regarding the proposed prefabrication drainage panel material to confirm it is appropriate for its intended use. All water should flow to a suitable discharge point.

Below-grade walls should be waterproofed and provided with water stops at all construction joints. The waterproofing should be placed directly against the backside of the walls.

For the design of low interior walls, such as for elevator pits, drainage may be omitted behind the walls provided the walls are designed for the hydrostatic pressure (undrained condition). Table 5 presents the active, at-rest and total pressures (active plus seismic pressure increment) for soil with level backfill for undrained conditions. All parameters are presented as equivalent fluid weights (triangular distribution).

TABLE 5

Below Grade Wall Design Earth Pressures (Undrained Conditions)

Retained Material	Static Conditions		Seismic Conditions[1]	
	Unrestrained Walls Active Condition (pcf)	Restrained Walls At-Rest Condition (pcf)	Total Pressure (Active Plus Seismic Pressure Increment) (pcf)	
			DE[2]	MCE[3]
Clayey Sand	83	95	100	110

Notes:

1. The more critical condition of either at-rest pressure (static condition) or active pressure plus a seismic pressure increment (seismic condition) should be checked.
2. DE = Design Earthquake
3. MCE = Maximum Considered Earthquake

8.7 Construction Monitoring

During excavation, the shoring system may yield and deform, which could cause surrounding improvements to settle and move laterally. The conditions of existing buildings within 100 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction. The monitoring program should include shoring movement (survey points).

8.8 Seismic Design Criteria

For the 2013 California Building Code (CBC) seismic code, we recommend the following:

- Mapped spectral acceleration values of S_s and S_1 equal to 1.899g and 0.766g respectively
- Site Class D
- Site coefficients F_a and F_v equal to 1.0 and 1.5, respectively.
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.899g and 1.148g, respectively
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.266g and 0.766g, respectively.

9.0 ADDITIONAL GEOTECHNICAL SERVICES

During final design we should be retained to consult with the design team as geotechnical questions arise. Prior to construction, we should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, we should observe site preparation, shoring installation, installation of building foundations, and fill compaction. These observations will allow us to compare the actual with the anticipated soil and bedrock conditions and to check that the contractors’ work conforms to the geotechnical aspects of the plans and specifications.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and to construction conditions as we have described them. They are the result of engineering studies and our interpretations of the existing geotechnical conditions. Actual subsurface conditions may vary. Should conditions substantially differ from those that we anticipate, some modifications to our conclusions and recommendations may be necessary.

Our firm has prepared this report for the exclusive use of our client and their representatives on this project in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by our firm during the construction phase in order to evaluate compliance with our recommendations. If we are not retained for these services, the client must assume Langan Treadwell Rollo's responsibility for potential claims that may arise during or after construction.

REFERENCES

Graymer, R.W., Brabb, E.E., Jones, D.J., Barnes D.L., Nicholson, R.S. and Stamski, R.E. (2007), "Geological Map and Map Database of Eastern Sonoma and Western Napa Counties, California", United States Geological Survey, Scientific Investigations Map 2956.

Jennings, C.W., and Bryant, W.A., 2010, Fault Activity Map of California: California Geological Survey Geologic Data Map No. 6, map scale 1:750,000.

California Building Code (2013)

California Division of Mines and Geology, 1997, "Fault Rupture Hazard Zones in California," CDMG Special Publication 42.

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

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

Langan Treadwell Rollo, (2014). "Report of Preliminary Geotechnical Investigation and Consultation, 3093 Broadway"

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

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

Working Group on California Earthquake Probabilities (WGCEP), 2003, Earthquake Probabilities in the San Francisco Bay region: 2002 to 2031 – A summary of Findings, Open-File Report 03-214.

Working Group on California Earthquake Probabilities (WGCEP), 2007, "The Uniform California Earthquake Rupture Forecast, Version 2", Open-File Report 07-1437.

Wyss, M., (1979). "Estimating maximum expectable magnitude of earthquakes from fault dimensions," Geology; v. 7; no. 7; p. 336-340.

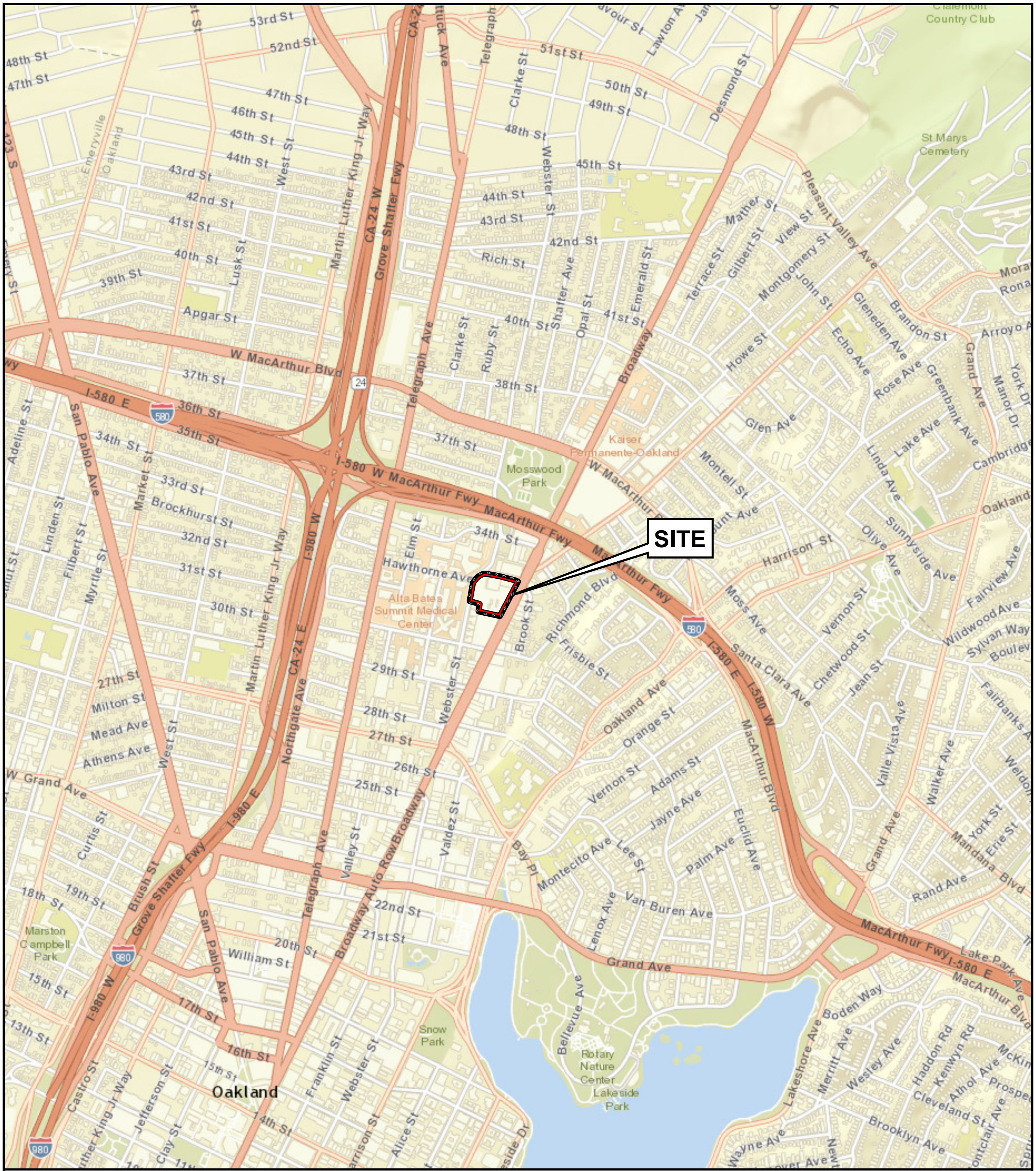
Yanev, P., 1974, "Peace of Mind in Earthquake Country" Chronicle Books, San Francisco, California.

Youd, T.L., and Hoose, S.N., (1978). "Historic ground failures in northern California triggered by earthquakes," U.S. Geological Survey Professional Paper 993.

Youd, T.L. and Garris, C.T., 1995, "Liquefaction Induced Ground-Surface Disruption", Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 121, 805-809.

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

FIGURES



Notes:

1. World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online. Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN .
2. Map displayed in California State Plane Coordinate System , Zone III, North American Datum of 1983 (NAD83) , US Survey Feet .

0 1,000 2,000
Feet

N
W E
S

3093 BROADWAY
Oakland, California

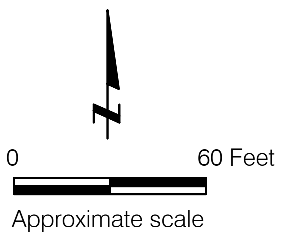
LANGAN TREADWELL ROLLO

SITE LOCATION MAP

Date 10/7/2014
Project No. 731637002
Figure 1

EXPLANATION

B-1 ⊕	Approximate location of boring by Langan Treadwell Rollo, August 2014
CPT-1 ▲	Approximate location of cone penetration test by Langan Treadwell Rollo, June 2014
MW-1 ●	Approximate location of monitoring well
— - —	Property line
53	Existing ground surface elevation (City of Oakland datum)
🟥	Existing car dealership and attached garage to be demolished
🟦	Existing two structures to be demolished
▨	Existing historic stucco facade to be retained and incorporated into new building
🟩	Proposed building footprint
▩	Existing concrete retaining wall
FFE	Finished Floor Elevation



Reference: Esri, Digital Globe, GeoEye, i-cubed, USDA, USGS AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and GIS User Community. Base map from a drawing titled "Existing Conditions Plan," by BKF, dated 2 September 2014 and "C2.0 Conceptual Grading Plan", by BKF, dated 19 August 2014, "Sheet A121, Garage Plan", by VTBS Architects, dated 22 May 2015.

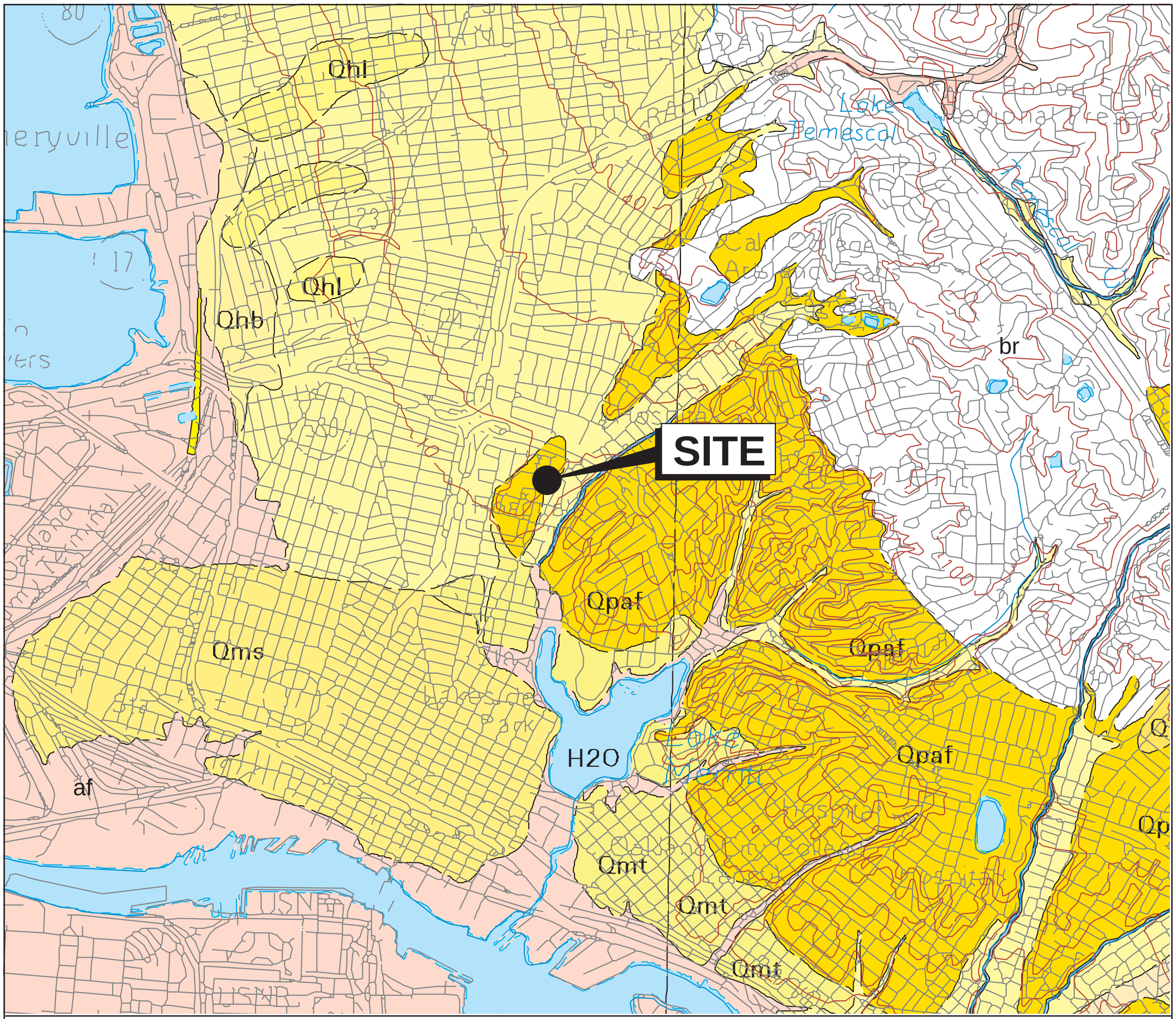
3093 BROADWAY
Oakland, California

SITE PLAN

Date 06/30/15	Project No. 731637002	Figure 2

LANGAN TREADWELL ROLLO

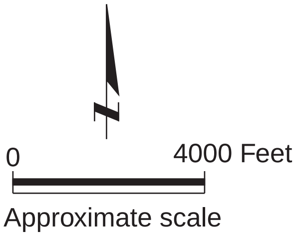
C:\Users\jvicente\appdata\local\temp\AcPublish_5292\731637002-B-SP0104.dwg 6/30/15



Base map: Quaternary Geology of Alameda County and Surrounding Areas, California: Derived from the Digital Database Open-File 97-97 by E.J. Helley and R.W. Graymer, 1997.

EXPLANATION

af	**Aritificial Fill**
Qhb	**Basin deposits (Holocene)**
Qhl	**Natural Levee deposits (Holocene)**
Qms	**Merritt Sand deposits (Holocene and Pleistocene)**
Qpaf	**Alluvial Fan deposits (Pleistocence)**
Qmt	**Marine Terrace deposits (Pleistocene)**
br	**Older alluvial fan depostis (Pleistocene)**
— — —	**Geologic contact**
—10—	**Elevation contour**



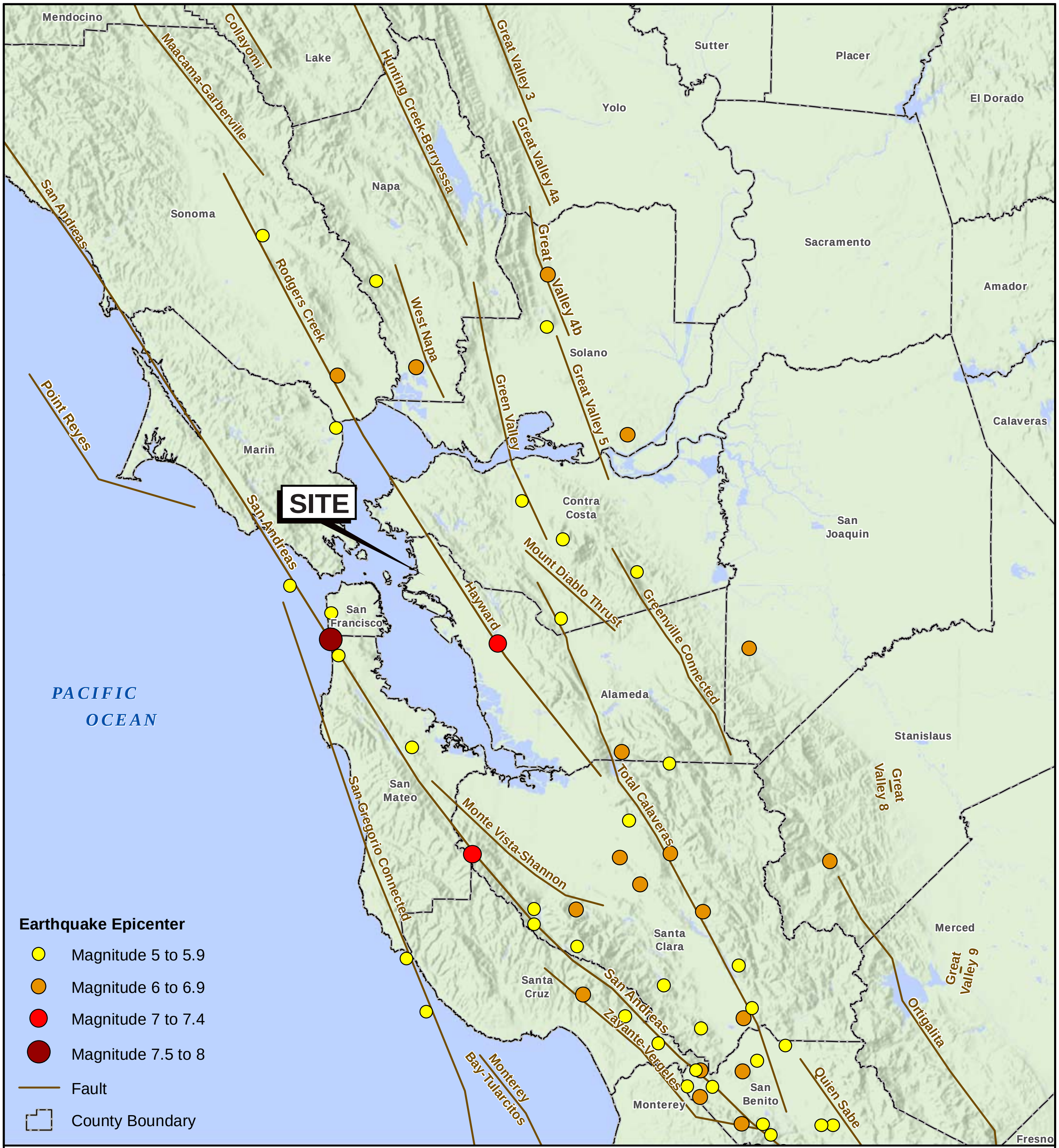
Approximate scale

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

REGIONAL GEOLOGIC MAP

Date 10/02/14 | Project No. 731637002 | Figure 3



Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 08/24/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

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

Date 10/02/14	Project No. 731637002	Figure 4

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

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

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

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

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

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

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

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

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

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

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

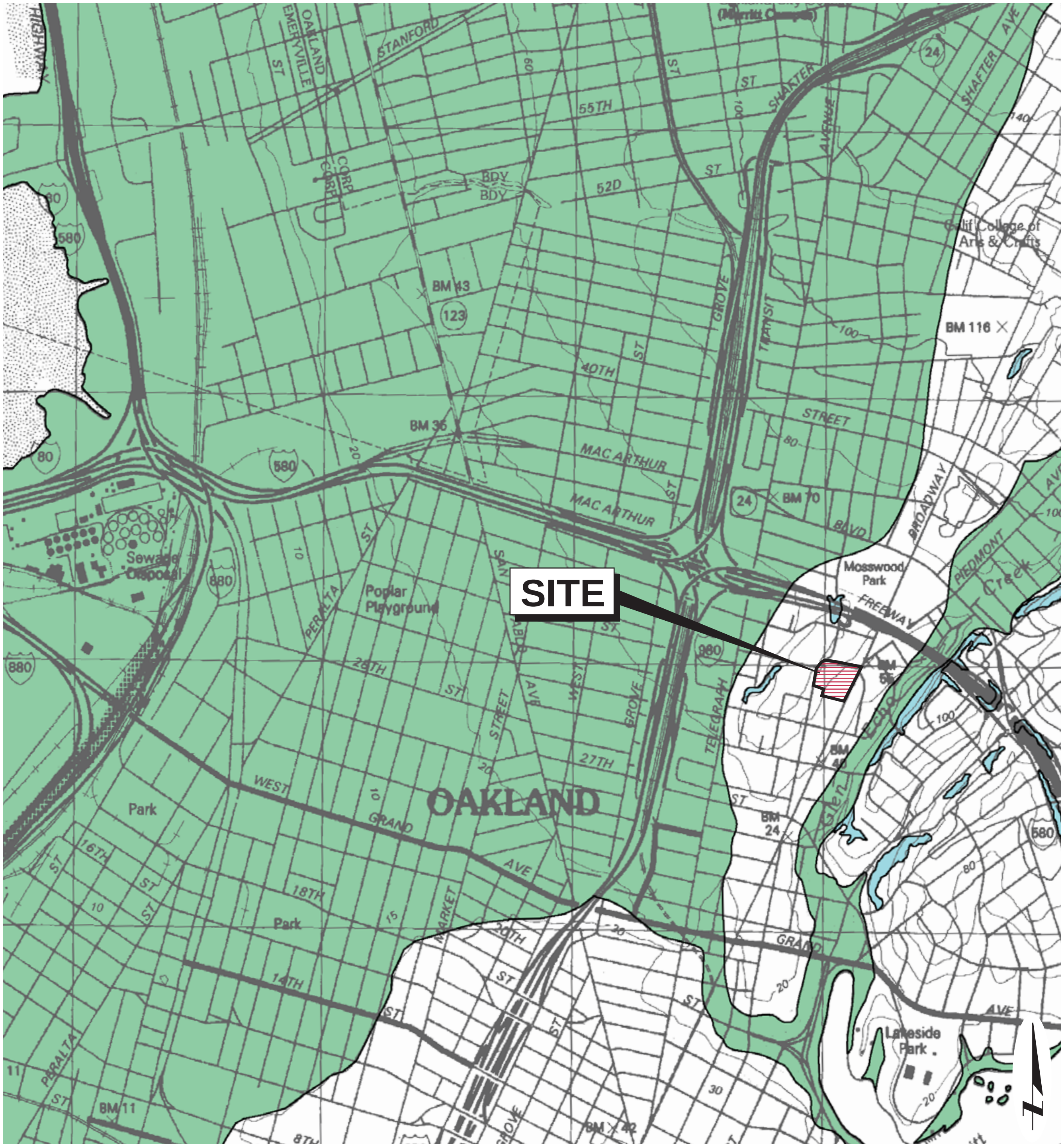
XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses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

3093 BROADWAY
Oakland, California

MODIFIED MERCALLI INTENSITY SCALE

LANGAN TREADWELL ROLLO

Date 10/02/14 | Project No. 731637002 | Figure 5



EXPLANATION

🟩 **Liquefaction;** Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

🟦 **Earthquake-Induced Landslides;** Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 2,000 4,000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones"
Oakland West Quadrangle.
Released on February 14, 2003

3093 BROADWAY
Oakland, California

REGIONAL SEISMIC HAZARD ZONES MAP

LANGAN TREADWELL ROLLO

Date 10/02/14	Project No. 731637002	Figure 6

Reference: Base map from a drawing titled "Overall Level One Garage Foundation Plan," by Hoogerwerf, dated 06/29/15. Locations of proposed remediation and pilot study from Langan Treadwell Rollo titled "Proposed Remediation Boring Locations, dated 06/10/15.

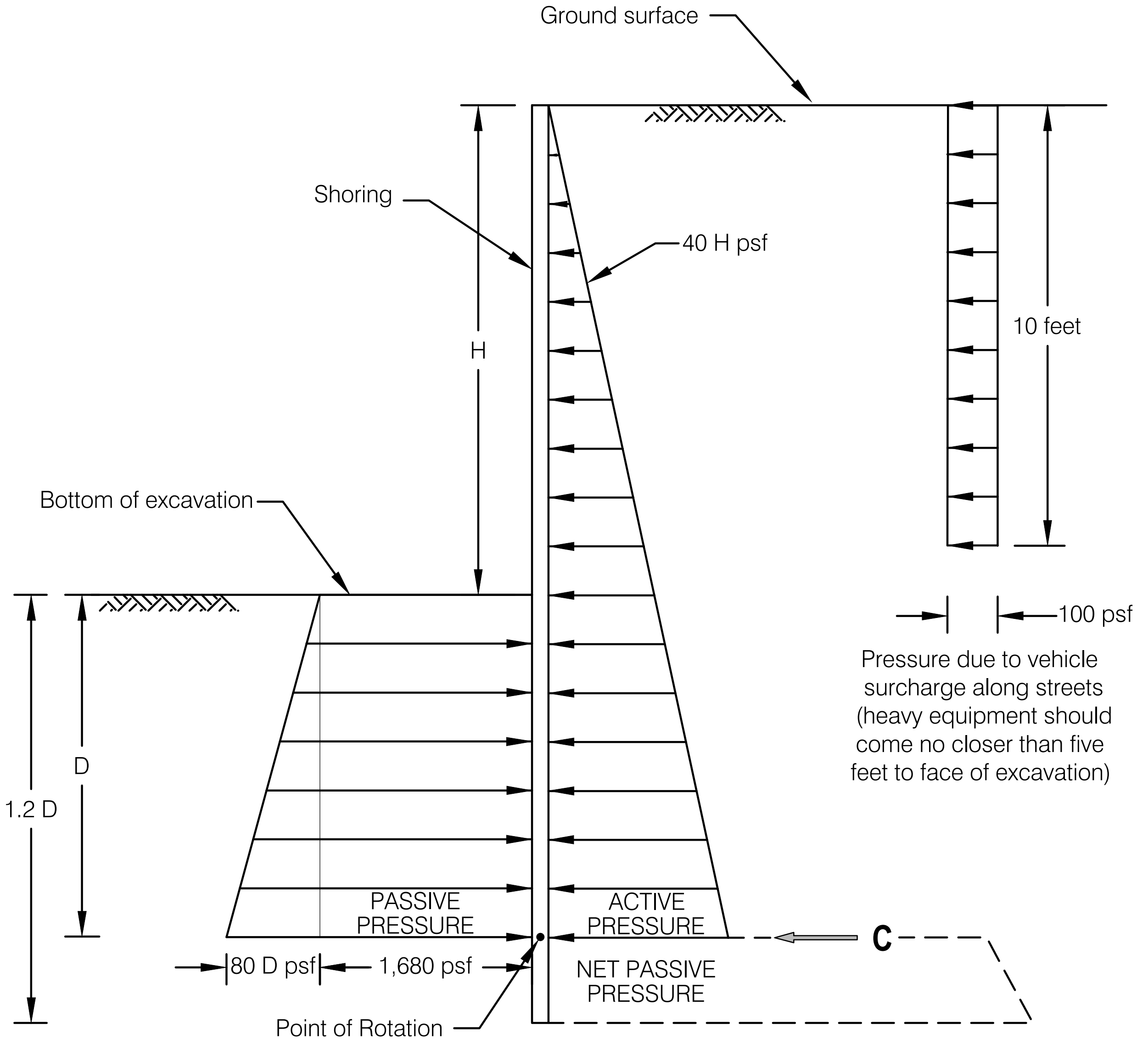
3093 BROADWAY
Oakland, California

FOUNDATION AREAS AFFECTED BY ENVIRONMENTAL REMEDIATION BORINGS

Date 07/02/15	Project No. 731637002	Figure 7

LANGAN TREADWELL ROLLO

C:\Users\vicente\appdata\local\temp\AcPublish_3624\731637002-B-SP0105.dwg 7/02/15



NOT TO SCALE

Notes and assumptions:

1. Simplified pressure diagram is presented above. The net passive pressure on the right side of shoring below the point of rotation is replaced by a concentrated force C at the point of rotation.
2. Passive pressures include a factor of safety of about 1.5.
3. Passive pressures may be assumed to act over the pier spacing or three times the pier diameter, whichever is smaller.
4. Surcharge pressure, due to construction equipment, if any, should be added to the above shoring pressure.
5. Active pressure below the excavation should be assumed to act over one pier diameter (for structural concrete).
6. Calculated embedment depth, D, should be increased by at least 20 percent to obtain the design depth of penetration.
7. pcf denoted pounds per cubic foot; psf denoted pounds per square foot.
8. Design groundwater at Elevation 33 feet (City of Oakland datum).
9. Bottom of excavation approximately Elevation 52 feet.

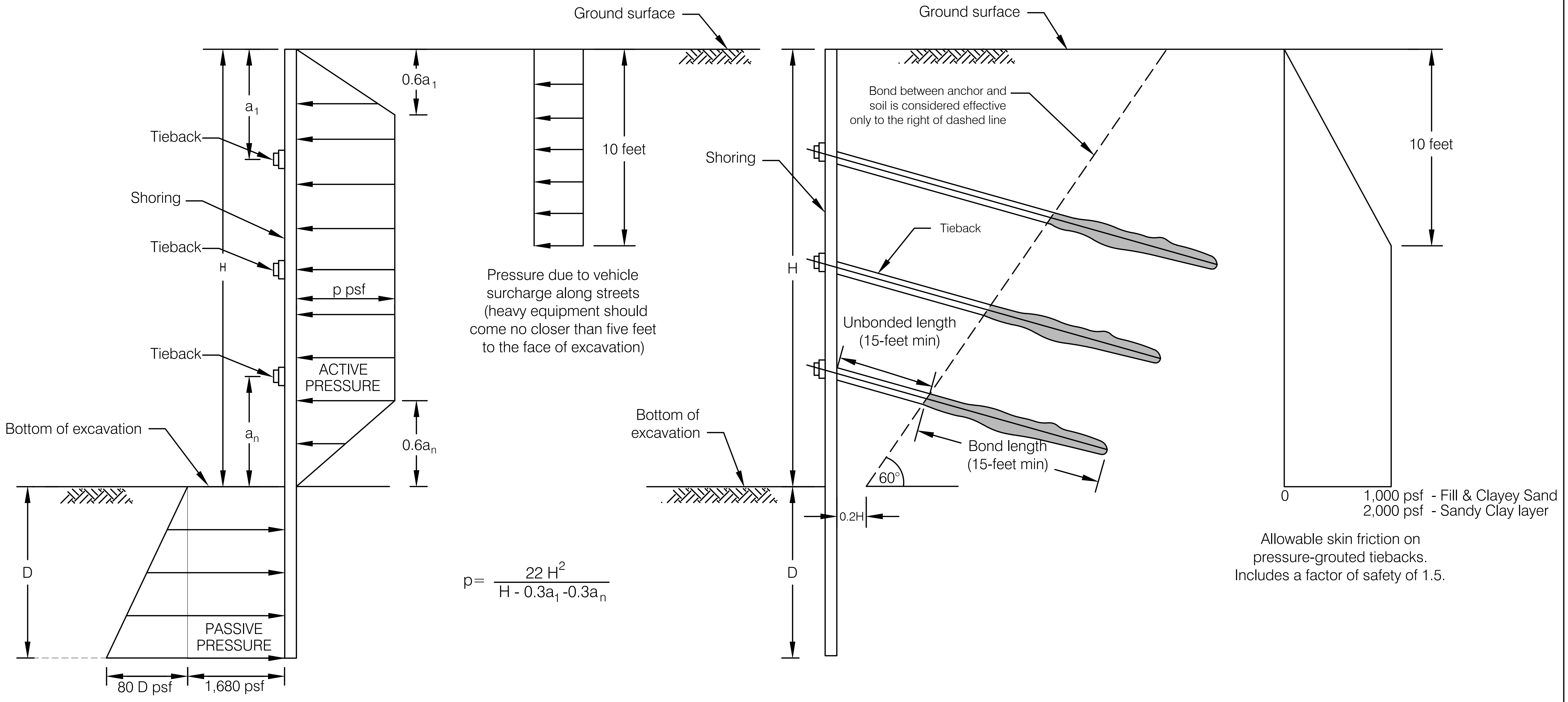
3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

LATERAL EARTH PRESSURES FOR CANTILEVER SOLDIER-PILE-AND-LAGGING SHORING SYSTEM

Date 07/02/15 | Project No. 731637002 | Figure 8

C:\Users\vicente\appdata\local\temp\AcPublish_5292\731637002-B-RW0101.dwg 6/30/15



$$p= \frac{22\ H^2}{H - 0.3a_1 -0.3a_n}$$

NOT TO SCALE

Notes:
1. Passive pressure includes a factor of safety of 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. The recommended pressures do not include surcharges from adjacent structures. Surcharge pressure from adjacent structures should be added to the above shoring pressures, as appropriate.
4. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
5. Design groundwater at Elevation 33 feet (City of Oakland datum).
6. Bottom of excavation approximately Elevation 52 feet.
7. Vertical load of existing concrete retaining wall should be incorporated into shoring design, as appropriate.

3093 BROADWAY
Oakland, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING SHORING SYSTEM WITH MULTIPLE TIEBACKS

Date 07/02/15 | Project No. 731637002 | Figure 9

LANGAN TREADWELL ROLLO

\\langan.com\data\SF\data0\731637002\Cadd Data - 731637002\2D-DesignFiles\Geotechnical\731637002-B-RW0101.dwg 6/30/15

APPENDIX A

BORING LOGS

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 8/26/14
Date finished: 8/26/14
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: N. Sherwood

LABORATORY TEST DATA

Ground Surface Elevation: 56 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ 6"	SAMPLES SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						3 inches asphalt concrete (AC)						
1–5					CL	SANDY CLAY with GRAVEL (CL) red to olive brown, moist, subrounded to subangular gravel dark-brown and gray trace wood and small brick fragments FILL						
5–6	S&H	■	3 12 21	23	CL	SANDY CLAY (CL) yellow-brown with black mottling of red-yellow oxidation stains, very stiff, moist	TxUU	500	3,518		14.5	117
10–11	S&H	■	16 50/6" 10	30/6"	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown to red-brown, very dense, moist, coarse gravel, subrounded to subangular gravel					12.6	125
12	SPT	◩	10 23 25	58	SC					19.0	13.7	
15–16	S&H	■	6 12 13	18	CL	SANDY CLAY (CL) yellow-brown mottled olive, very stiff, moist (hydrocarbon odor at 15.5 to 18 feet during drilling) LL = 31, PI = 12				50.2	23.2	
20–21	S&H	■	8 10 14	17	CL		TxUU	2,000	1,232		23.8	101
25–26	S&H	■	8 13 17	21	CL							
30												

LANGAN TREADWELL ROLLO

Project No.: 731637002
Figure: A-1a

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	■	11 28 30	41	SC	CLAYEY SAND with GRAVEL (SC) olive gray with orange oxidation staining to olive, dense, wet, subrounded to subangular gravel						
36	SPT	◩	10 18 21	47								
41	SPT	◩	6 9 10	23	CL	SANDY CLAY (CL) yellow-brown, very stiff, wet LL = 27, PI = 8 trace gravel						
46	S&H	■	10 20 39	71	SM	SILTY SAND with GRAVEL (SM) yellow-brown, very dense, wet						
51	SPT	◩	9 15 17	38	CL-ML	SILTY CLAY/CLAYEY SILT (CL-ML) olive-brown, hard, wet						

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

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

LANGAN TREADWELL ROLLO

Project No.: 731637002
Figure: A-1b

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2
Logged by: N. Sherwood
Date started: 8/26/14
Date finished: 8/26/14
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value^1		Ground Surface Elevation: 54 feet^2	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						3.5 inches asphalt concrete (AC)						
1 2 3 4					CL	SANDY CLAY with GRAVEL (CL) red to olive-brown, moist, subrounded to subangular gravel color transition to dark brown and gray, trace wood and small red brick fragments FILL						
5 6	S&H	▨	10 23 27	35	SC	CLAYEY SAND (SC) yellow-brown mottled olive brown with black spots, yellow-red oxidation stains, dense, moist					14.3	120
7 8 9												
10 11	SPT	◩	29 17 25	50	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, coarse to medium-grain sand, subrounded to subangular gravel						
12 13 14												
15 16	S&H	▨	8 15 18	23	SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist				42.3	22.9	101
17 18 19												
20 21	SPT	◩	6 8 9	20								
22												
23 24					CL	CLAY with SAND (CL) olive-gray to gray-brown, very stiff, moist, fine-grain sand						
25 26	S&H	▨	6 10 14	17			TxUU	2,500	2,103		29.3	94
27 28 29 30												

LANGAN TREADWELL ROLLO

Project No.: 731637002
Figure: A-2a

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	■	6 8 15	16		CLAY (CL) gray, very stiff, wet, trace fine-grain sand						
32					CL							
33												
34												
35												
36	S&H	■	12 25 33	41		CLAYEY SAND (SC) yellow-brown mottled olive, dense, wet, fine-grain sand						
37					SC							
38												
39												
40												
41	SPT	◩	13 26 25	61		CLAYEY SAND with GRAVEL (SC) olive brown, very dense, wet, subrounded to subangular gravel						
42					SC							
43												
44												
45												
46	S&H	■	12 23 27	35		CLAY (CL) olive-gray, hard, wet, trace sand and silt					26.6	100
47					CL							
48												
49												
50												
51	S&H	■	20 34 50	59	CL	CLAY with SAND (CL) gray, hard, wet						
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

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

[2] Elevations based on City of Oakland datum.

LANGAN TREADWELL ROLLO

Project No.: 731637002

Figure: A-2b

TEST GEOTECH LOG 731637002.GPJ TR.GDT 10/9/14

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2
Date started: 8/25/14
Date finished: 8/25/14
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: N. Sherwood

	SAMPLES					MATERIAL DESCRIPTION	LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	Ground Surface Elevation: 57.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches asphalt concrete (AC)						
1					CL	SANDY CLAY with GRAVEL (CL) red-yellow to yellow-brown, moist, trace red brick, wood fragments FILL						
2												
3						SANDY CLAY (CL) yellow-brown with black and orange, very stiff, moist						
4												
5	S&H	▨	8 15 21	25			TxUU	450	4,227		16.0	115
6					CL							
7												
8												
9												
10												
11	S&H	▨	12 28 40	48		CLAYEY SAND with GRAVEL (SC) yellow-brown, dense, moist, subrounded to subangular gravel						
12												
13					SC							
14												
15												
16	S&H	▨	7 12 14	18		CLAY with SAND (CL) yellow-brown with mottled olive, very stiff, moist LL = 36, PI = 16						
17												
18					CL							
19												
20												
21	SPT	▨	4 6 8	17		CLAY (CL) yellow-brown, very stiff, moist LL = 47, PI = 25				91.7	31.1	
22												
23					CL							
24												
25												
26	S&H	▨	11 26 37	44		CLAYEY SAND with GRAVEL (SC) yellow-brown, dense, moist, coarse to medium grain sand						
27												
28					SC							
29												
30												

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-3

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	■	10 19 24	30	CL	CLAY with SAND (CL) yellow-brown, very stiff, wet	TxUU	3,000	3,010		27.0	96
32												
33												
34												
35												
36	S&H	■	9 23 27	35		hard						
37												
38												
39												
40												
41	S&H	■	13 22 29	36							27.8	97
42												
43												
44					ML	SILT (ML) olive, wet						
45					CL-ML	SILTY CLAY/CLAYEY SILT (CL-ML) olive-gray, very stiff, wet						
46	SPT	◪	10 10 15	30								
47												
48												
49					ML	SILT with SAND (ML) olive, hard, wet						
50												
51	SPT	◪	9 14 18	38								
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

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

LANGAN TREADWELL ROLLO

Project No.: 731637002

Figure: A-3b

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-4

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 8/25/14
Date finished: 8/25/14
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: N. Sherwood

Ground Surface Elevation: 64 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						1.5 inches asphalt concrete (AC)						
1					CL	SANDY CLAY with GRAVEL (CL) dark brown, moist						
2						red brick fragments						
3						brick layer						
0–5						FILL						
5					SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, dense, moist, subrounded to subangular gravel, medium to coarse sand						
6	S&H	■	11 20 24	31						16.4	15.1	
11	SPT	◪	10 15 13	34								
14					CL	SANDY CLAY (CL) yellow-brown, very stiff, moist, fine to very fine-grain sand						
16	S&H	■	11 13 14	19						64.6	26.8	98
20					CL-ML	SILTY CLAY/CLAYEY SILT (CL-ML) yellow-brown, very stiff, moist, trace black fine grain shale gravel						
21	S&H	■	9 15 22	26			TxUU	2,000	1,264		20.2	106
26	S&H	■	11 20 23	30								
30												

TEST GEOTECH LOG 731637002.GPJ TR.GDT 10/9/14

LANGAN TREADWELL ROLLO

Project No.: 731637002
Figure: A-4a

PROJECT: **3093 BROADWAY**
Oakland, California

Log of Boring B-4

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H		10 18 22	28		SILTY CLAY/CLAYEY SILT (CL-ML) (continued)				90.9	28.1	
32												
33					CL-ML							
34												
35												
36	SPT		13 18 27	54		SAND with SILT (SM) yellow-brown, very dense, wet, medium to fine-grain						
37												
38					SM							
39												
40												
41	SPT		7 10 14	29		CLAYEY SAND (SC) olive-gray with oxidation stains, medium dense, wet, fine-grain				33.0	26.7	
42												
43												
44												
45												
46	S&H		7 8 13	15	SC	LL = 29, PI = 11 olive-gray, medium dense, wet				46.5	29.0	
47	SPT		4 12 16	34		LL = 29, PI = 10 olive-gray with oxidation stains, dense, wet, fine-grain				46.8	29.5	
48												
49												
50												
51	SPT		15 20 25	54		yellow-brown, very dense, coarse grain, trace gravel						
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

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

LANGAN TREADWELL ROLLO

Project No.: 731637002

Figure: A-4b

TEST GEOTECH LOG 731637002.GPJ TR.GDT 10/9/14

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒ Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

◩ Classification sample taken with Standard Penetration Test sampler

◨ Undisturbed sample taken with thin-walled tube

☒ Disturbed sample

⊙ Sampling attempted with no recovery

◫ Core sample

◉ Analytical laboratory sample, grab groundwater

▥ Sample taken with Direct Push sampler

▥ Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

3093 BROADWAY
Oakland, California

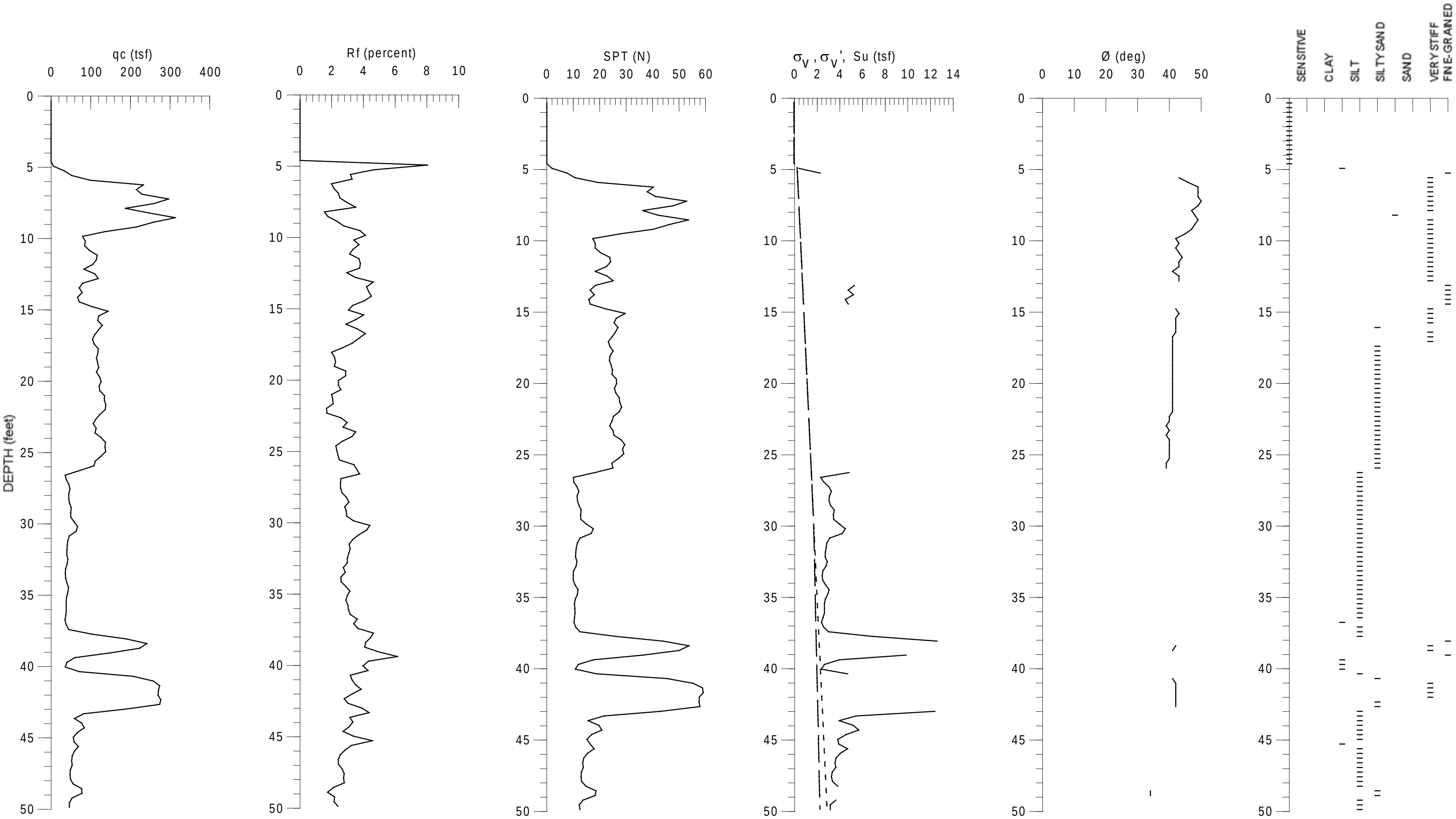
LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 09/02/14 | Project No. 731637002 | Figure A-5

APPENDIX B

CONE PENETRATION TEST RESULTS



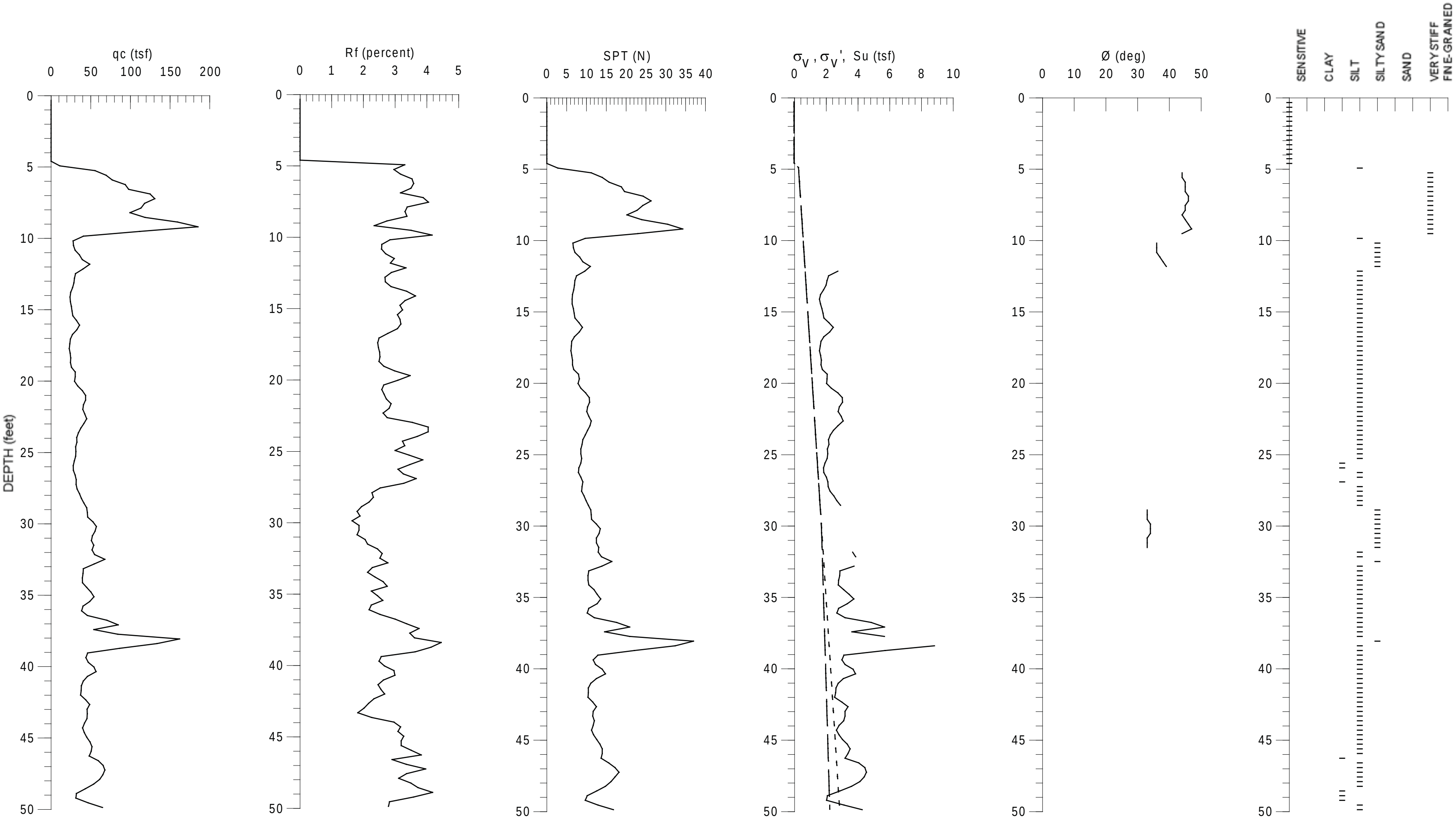
Terminated at 49.8 feet.
Date performed 06/20/14.

3093 BROADWAY
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 10/02/14	Project No. 731637002	Figure B-1

LANGAN TREADWELL ROLLO



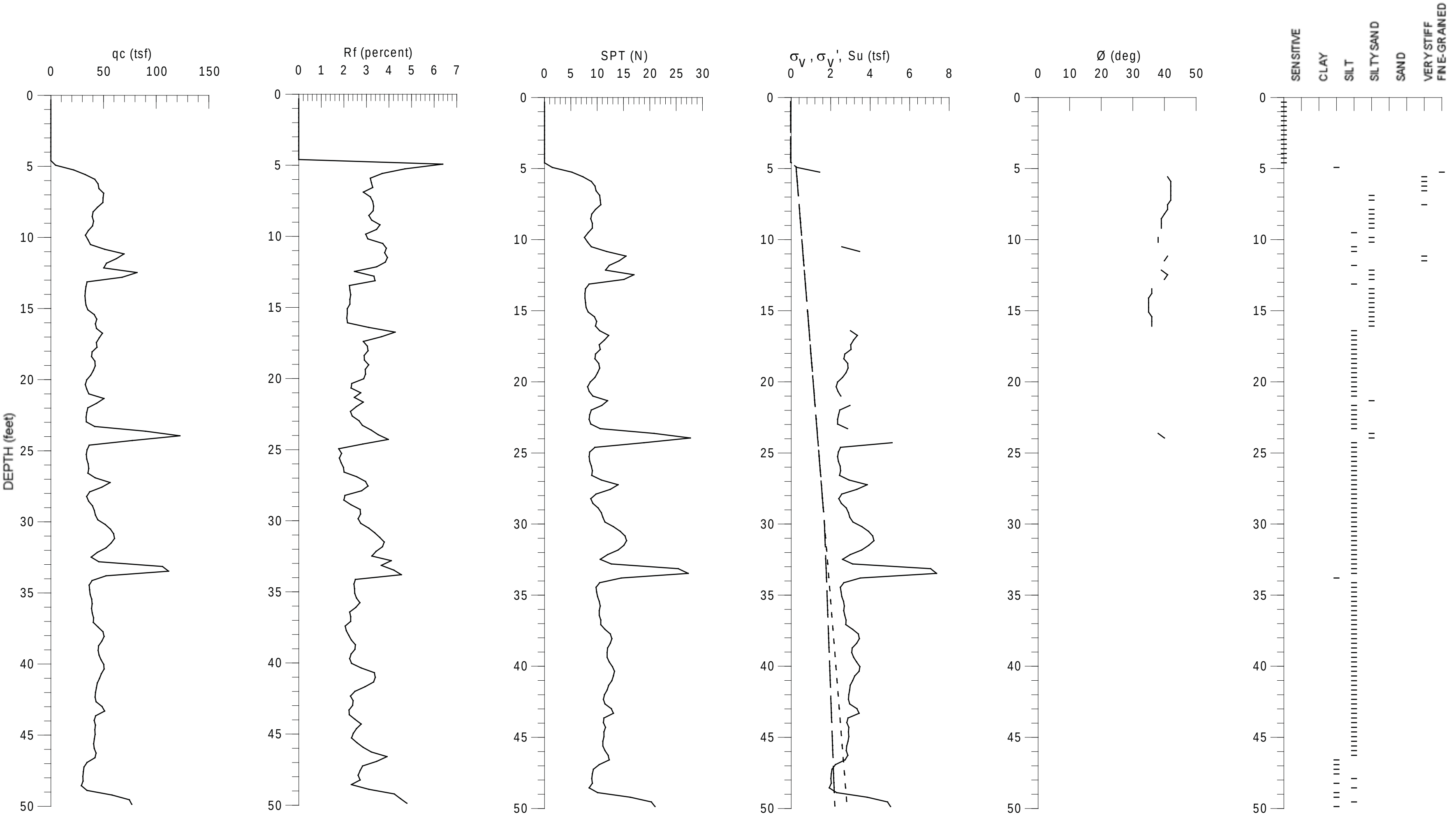
Terminated at 49.8 feet.
Date performed 06/20/14.

3093 BROADWAY
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 10/02/14	Project No. 731637002	Figure B-2

LANGAN TREADWELL ROLLO



Terminated at 49.8 feet.

Groundwater assumed at 30 feet below ground surface (based on pore pressurre dissipation test).

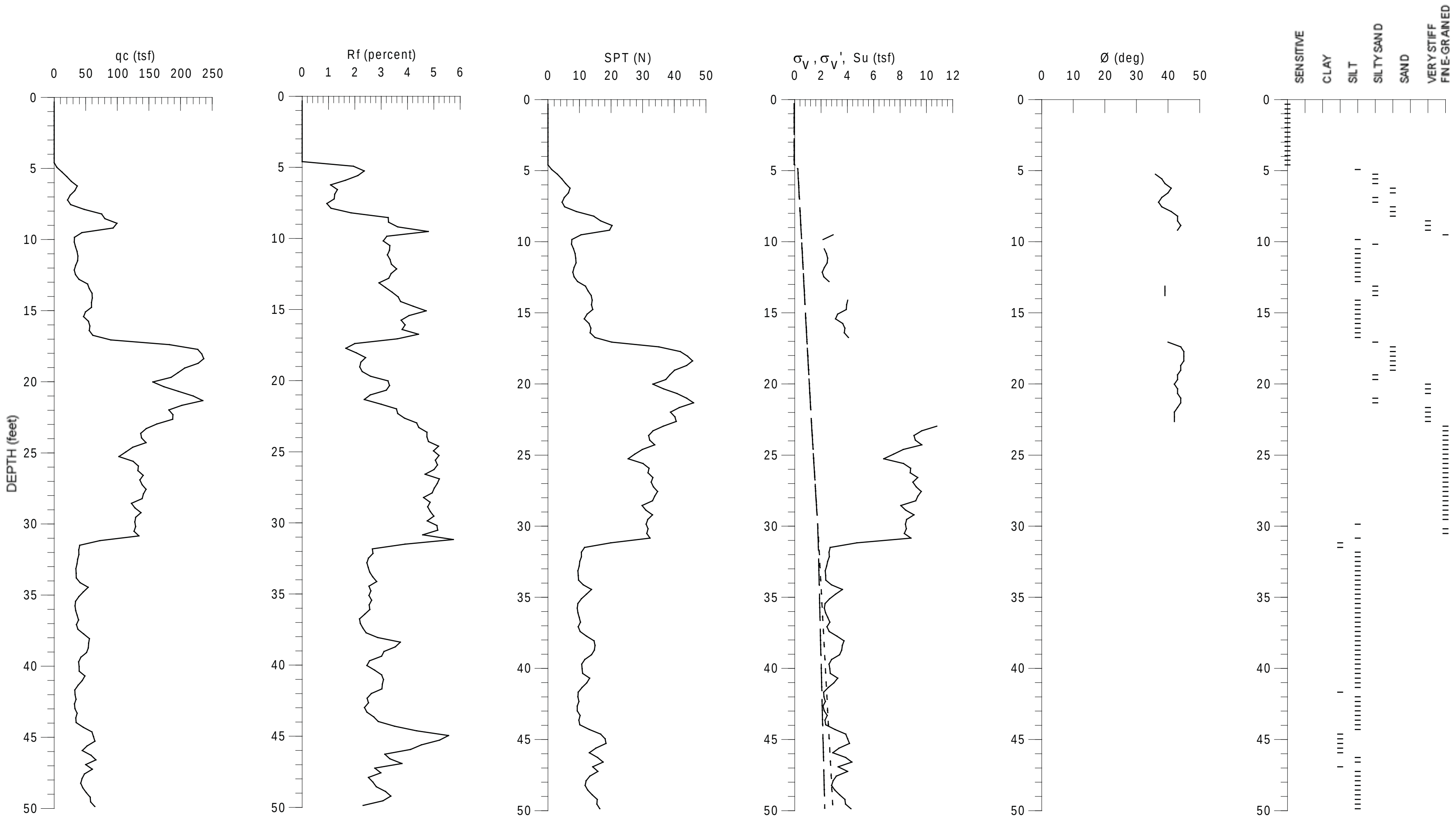
Date performed 06/20/14.

3093 BROADWAY
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 10/02/14 | Project No. 731637002 | Figure B-3

LANGAN TREADWELL ROLLO



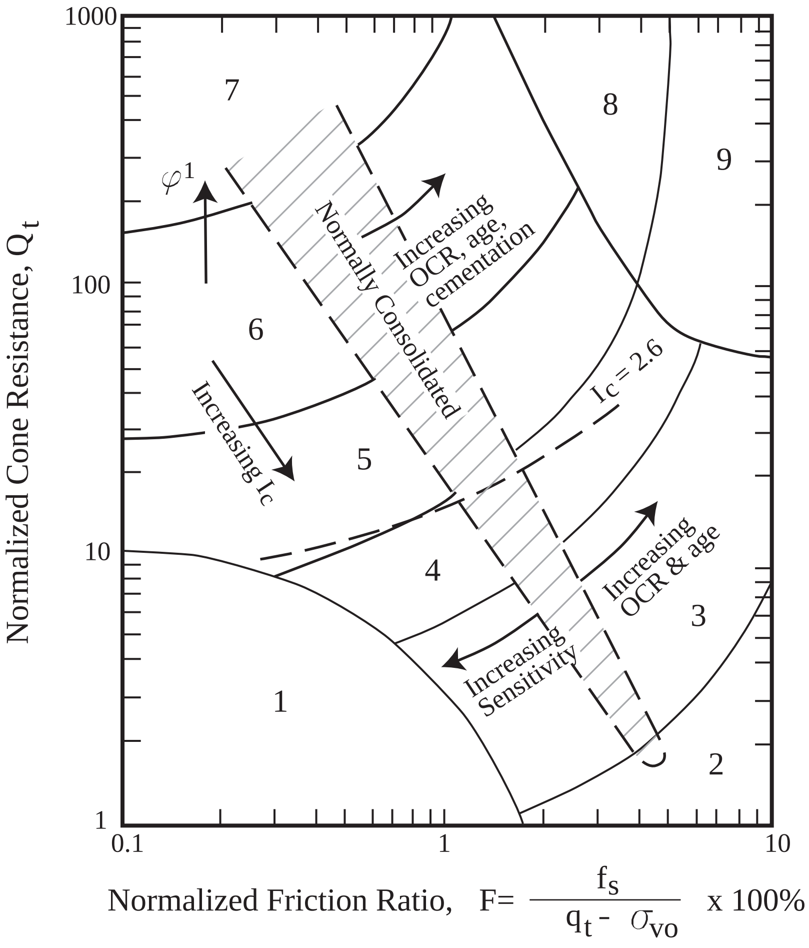
Terminated at 49.8 feet.
Date performed 06/20/14.

3093 BROADWAY
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-4

Date 10/02/14	Project No. 731637002	Figure B-4

LANGAN TREADWELL ROLLO



ZONE	SOIL BEHAVIOR TYPE
1	Sensitive Fine Grained
2	Organic Material
3	SILTY CLAY to CLAY
4	CLAYEY SILT to SILTY CLAY
5	SILTY SAND to SANDY SILT
6	SANDS to SILTY SAND
7	GRAVELLY SAND to Dense SAND
8	Very Dense SAND to CLAYEY SAND
9	Very Stiff, Fine Grained

$Q_t = (q_t - \sigma_{vo})/\sigma'_{vo}$ = Normalized Cone Resistance
$q_t = q_c + (1\text{-}a)u_2$ = Corrected Cone Resistance
q_c = Measured Cone Resistance
a = 0.8 = Area Ratio of Cone
u_2 = Pore Pressure Measured Behind Cone During Test
σ_{vo} = Total Vertical Stress
σ'_{vo} = Total Effective Vertical Stress
$F = f_s/(q_t - \sigma_{vo}) \times 100\%$ = Normalized Friction Ratio
f_s = Measured unit Sleeve Friction Resistance

Note Testing Performed in Accordance with ASTM D5778-95

Reference: Lunne, T., Robertson, P.K., and Powell, J.J.M., 1997.

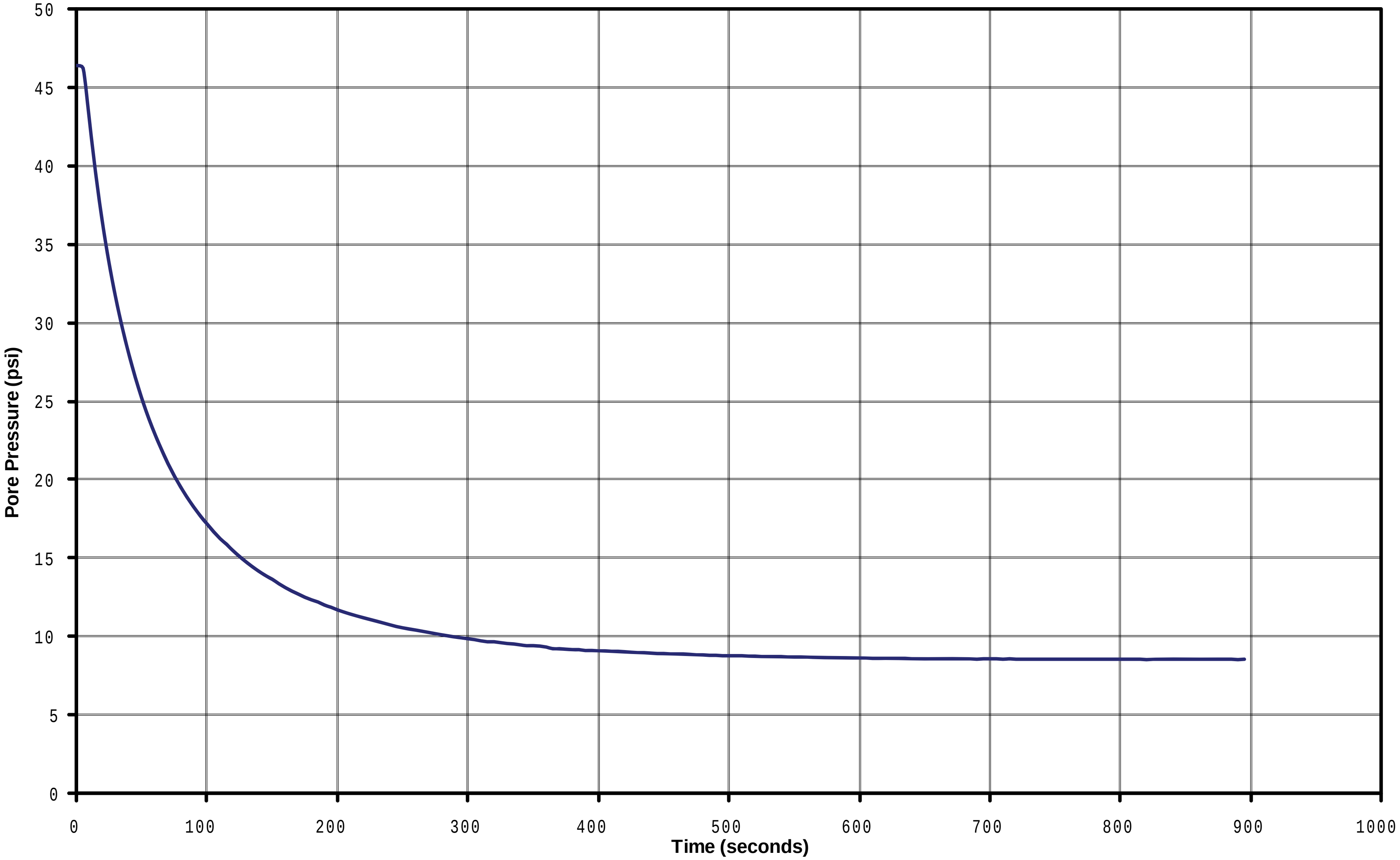
3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 10/02/14	Project No. 731637002	Figure B-5

GREGG



Terminated at 50 feet bgs
Groundwater measured at Elevation 33 feet
Date performed: 06/20/2014
Ground surface elevation: 63 feet
City of Oakland Datum

3093 BROADWAY
Oakland, California

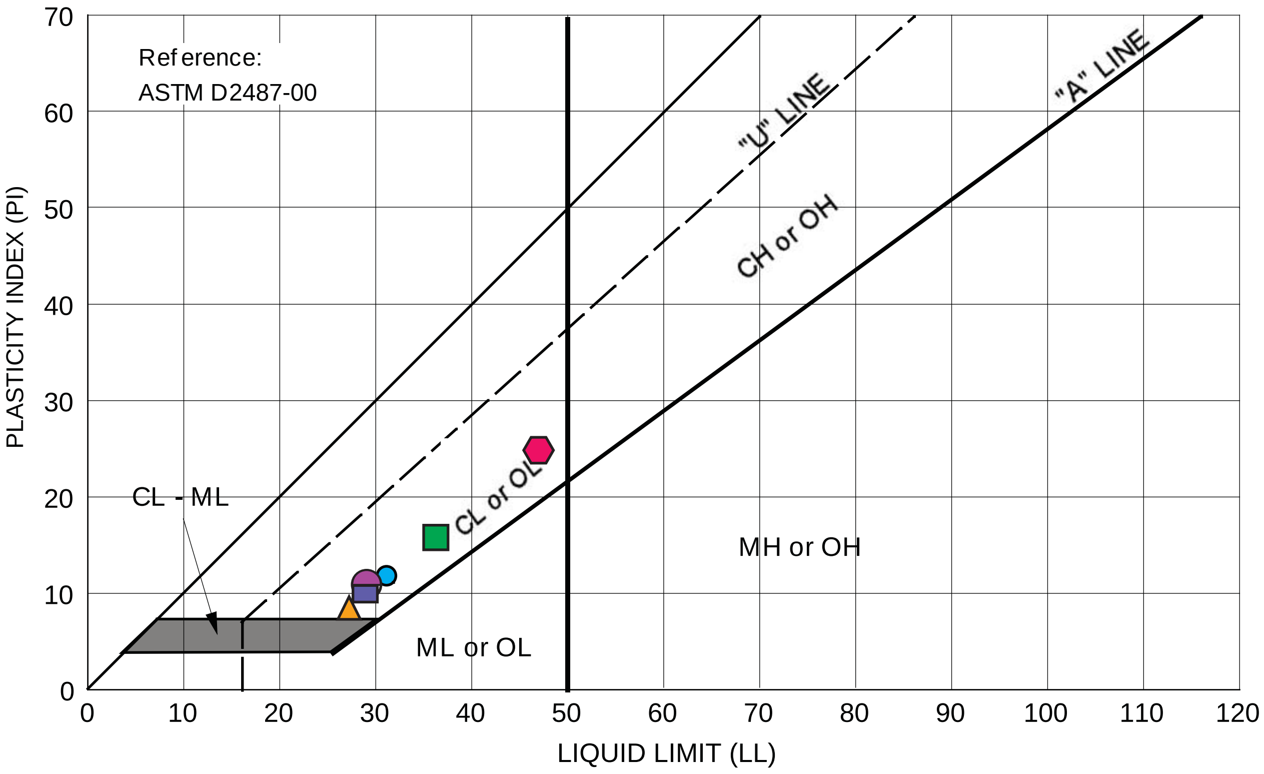
LANGAN TREADWELL ROLLO

PORE PRESSURE DISSIPATION TEST
CPT-3

Date 10/03/14 | Project No. 731637002 | Figure B-6

APPENDIX C

GEOTECHNICAL LABORATORY TEST RESULTS



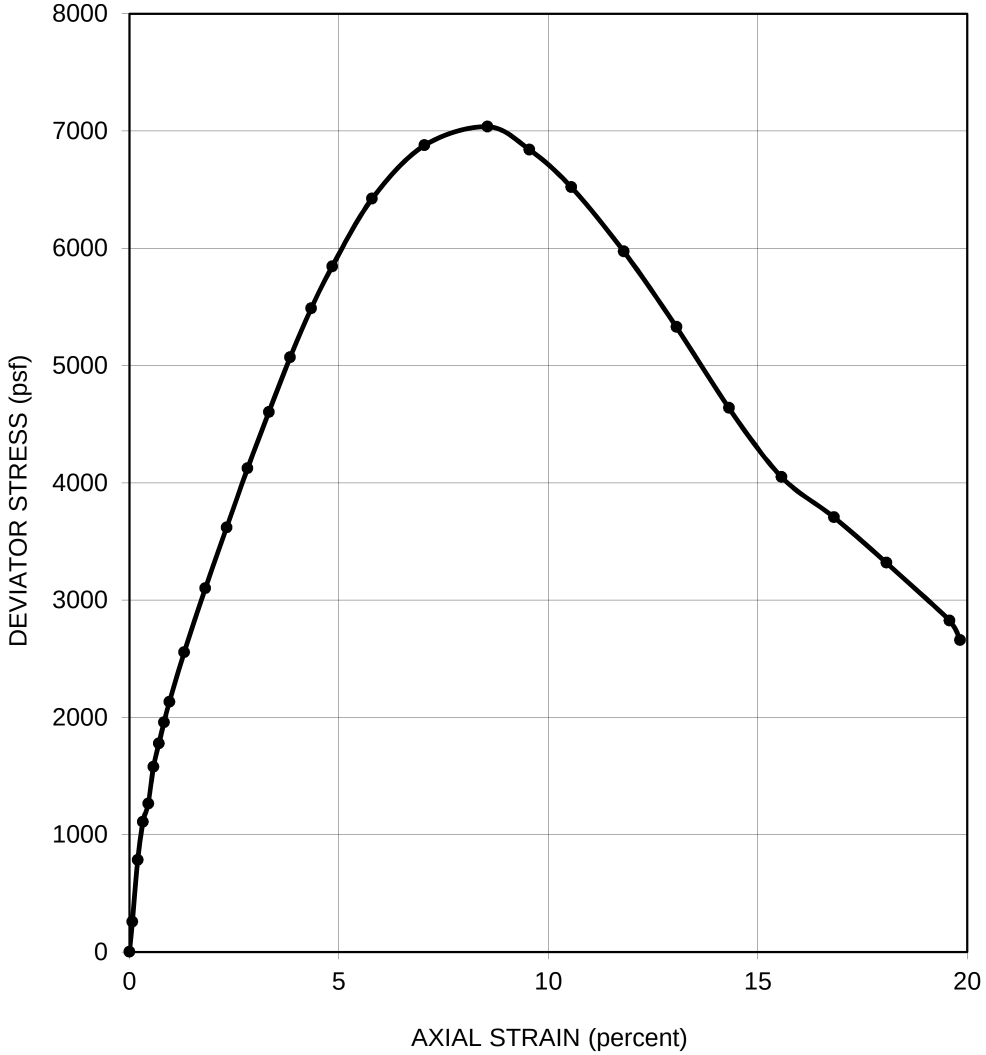
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 15.5 feet	SANDY CLAY (CL), yellow-brown mottled olive	23.2	31	12	50.2
▲	B-1 at 40 feet	SANDY CLAY (CL), yellow-brown	--	27	8	--
🟩	B-3 at 15 feet	CLAY with SAND (CL), yellow-brown with mottled olive	--	36	16	--
⬢	B-3 at 20 feet	CLAY (CL), yellow-brown	31.1	47	25	91.7
◓	B-4 at 45 feet	CLAYEY SAND (SC), olive-gray	29.0	29	11	46.5
■	B-4 at 45.5 feet	CLAYEY SAND (SC), yellow-brown	29.5	29	10	46.8

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 10/02/14 | Project No. 731637002 | Figure C-1



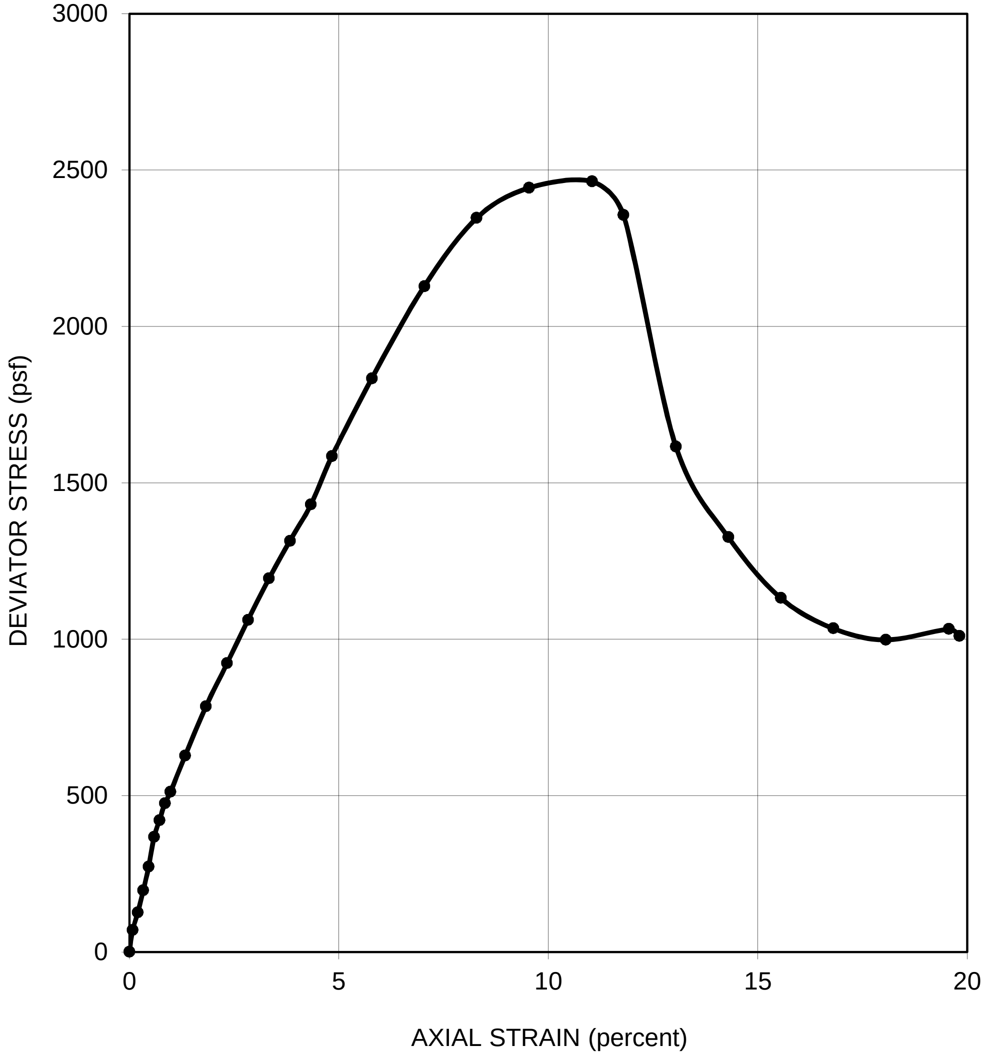
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	3,518	psf
DIAMETER (in.)	2.387	HEIGHT (in.)	4.79	STRAIN AT FAILURE	8.6	%
MOISTURE CONTENT		14.5	%	CONFINING PRESSURE	500	psf
DRY DENSITY		117	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown with black mottling			SOURCE	B-1 @ 5 feet	

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 10/02/14 | Project No. 731637002 | Figure C-2



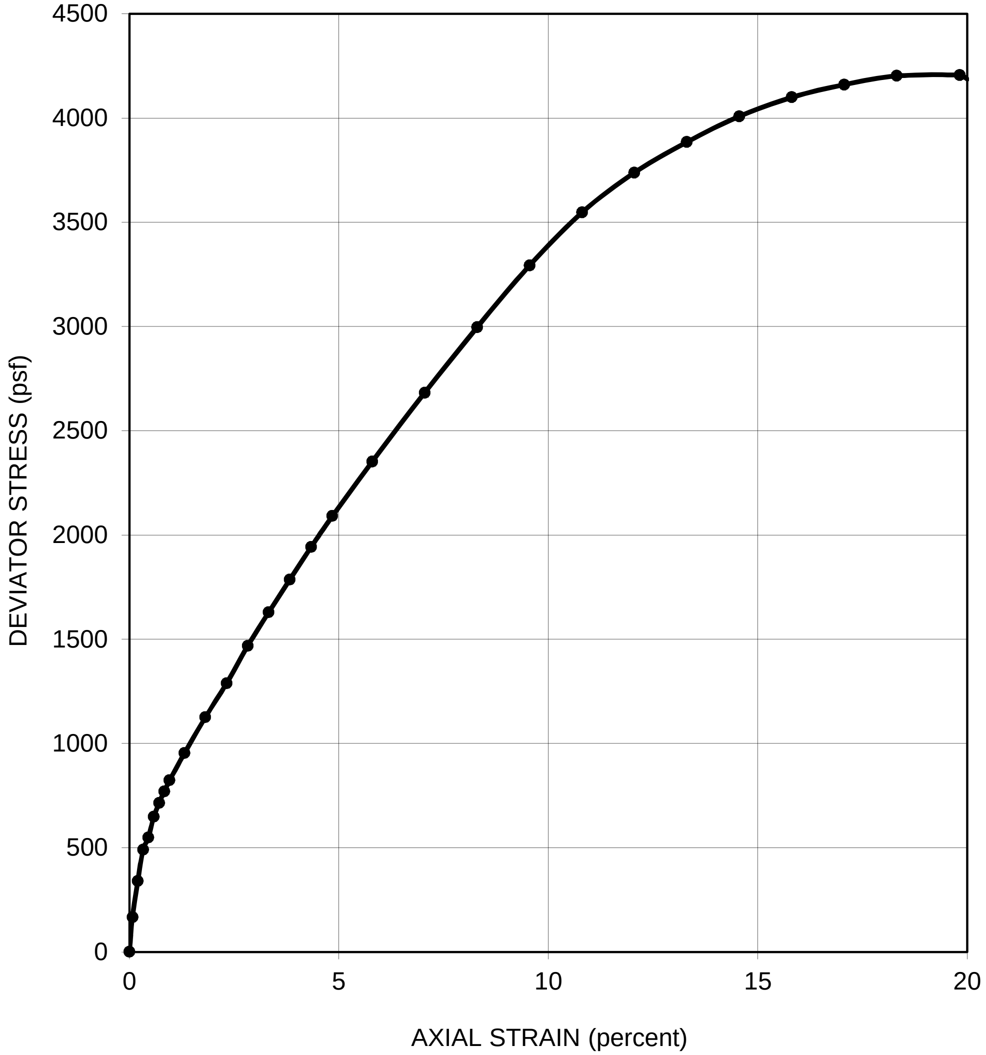
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	1,232	psf
DIAMETER (in.)	2.397	HEIGHT (in.)	5.71	STRAIN AT FAILURE	11.0	%
MOISTURE CONTENT		23.8	%	CONFINING PRESSURE	2,000	psf
DRY DENSITY		101	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown mottled olive				SOURCE	B-1 @ 20 feet

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Date	10/02/14	Project No.	731637002	Figure	C-3



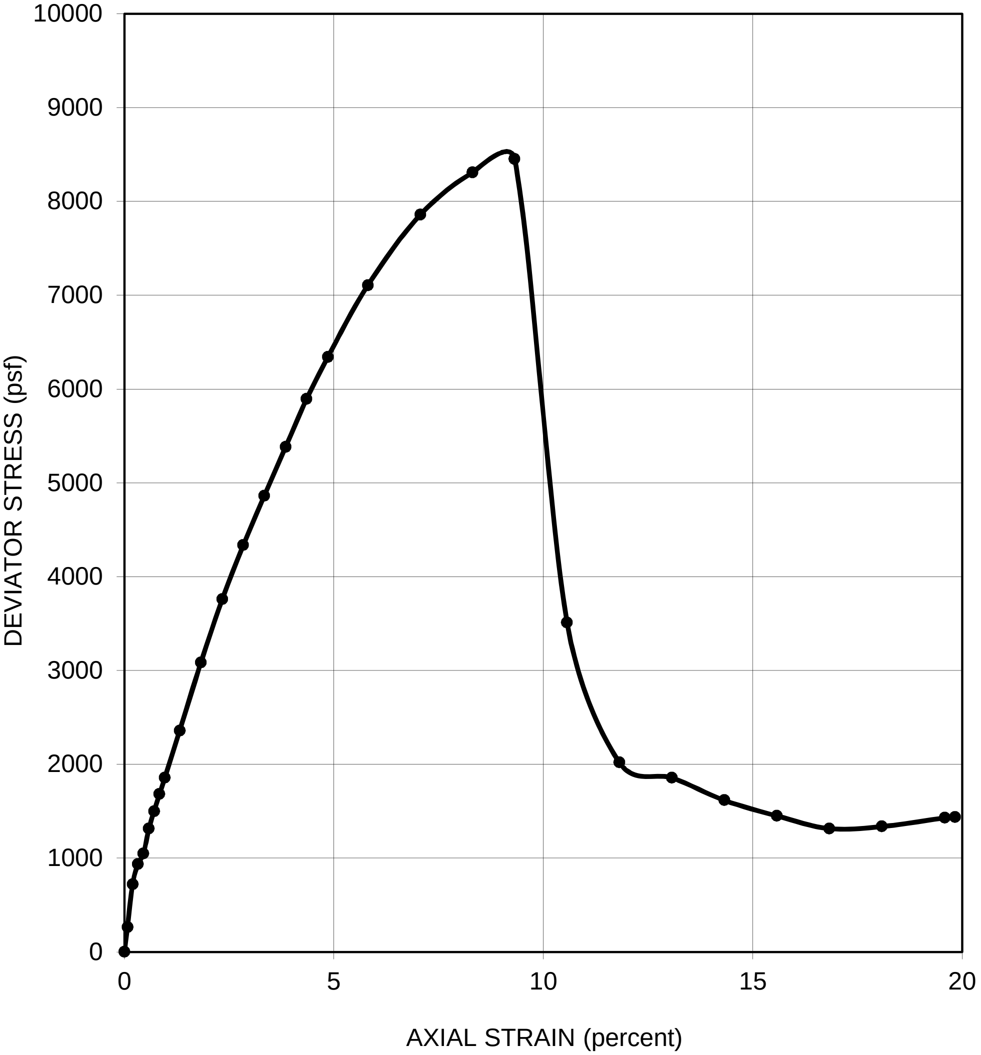
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	2,103	psf
DIAMETER (in.)	2.400	HEIGHT (in.)	5.69	STRAIN AT FAILURE	19.8	%
MOISTURE CONTENT		29.3	%	CONFINING PRESSURE	2,500	psf
DRY DENSITY		94	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY with SAND (CL), olive-gray to gray-brown				SOURCE	B-2 @ 25 feet

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Date 10/02/14 | Project No. 731637002 | Figure C-4



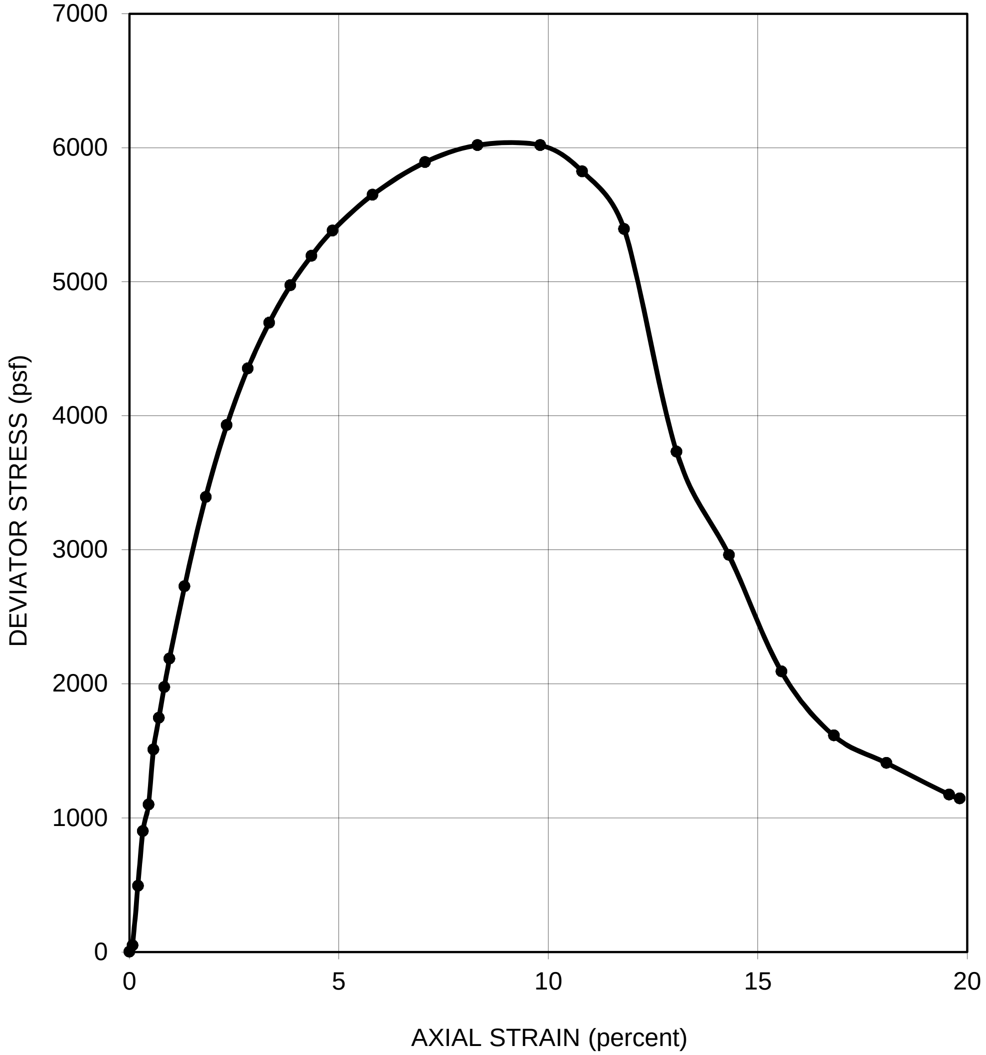
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	4,227	psf
DIAMETER (in.)	2.400	HEIGHT (in.) 5.69		STRAIN AT FAILURE	9.3	%
MOISTURE CONTENT		16.0	%	CONFINING PRESSURE	450	psf
DRY DENSITY		115	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown with black and orange			SOURCE	B-3 @ 4.5 feet	

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 10/02/14 | Project No. 731637002 | Figure C-5



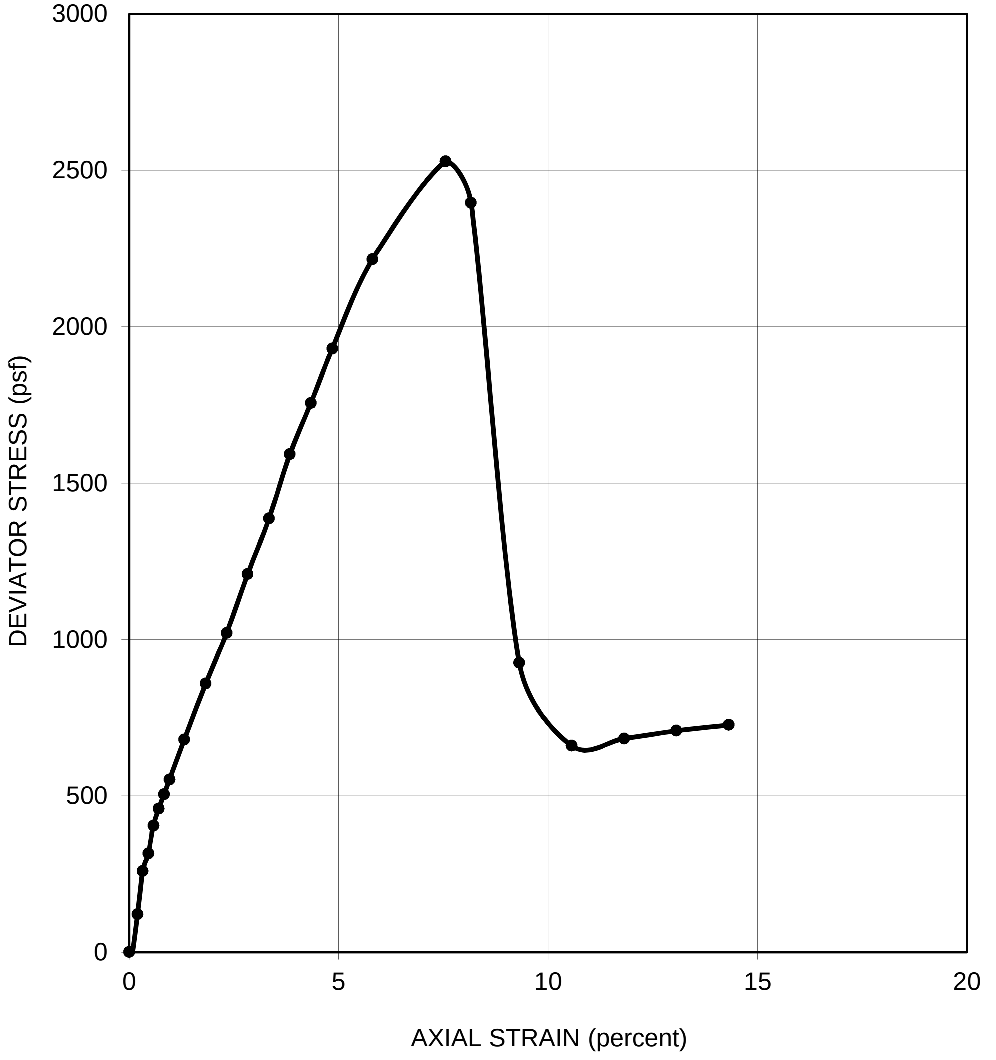
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH		3,010	psf
DIAMETER (in.)	2.397	HEIGHT (in.)	5.81	STRAIN AT FAILURE		9.8	%
MOISTURE CONTENT		27.0	%	CONFINING PRESSURE		3,000	psf
DRY DENSITY		96	pcf	STRAIN RATE		0.75	% / min
DESCRIPTION	CLAY with SAND (CL), yellow-brown				SOURCE	B-3 @ 30 feet	

3093 BROADWAY
Oakland, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 10/02/14 | Project No. 731637002 | Figure C-6



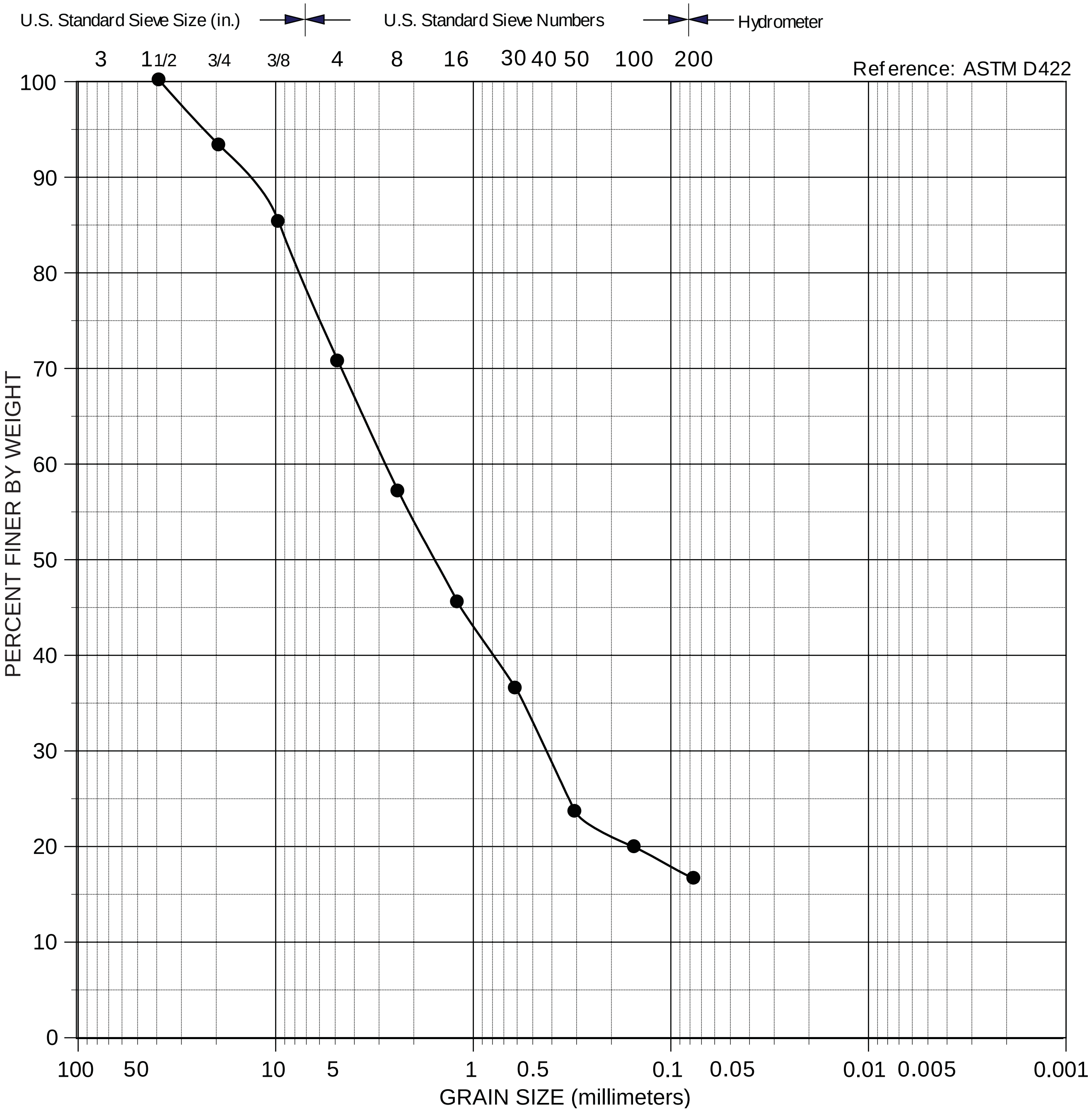
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	1,264	psf
DIAMETER (in.)	2.400	HEIGHT (in.)	5.71	STRAIN AT FAILURE	7.6	%
MOISTURE CONTENT		20.2	%	CONFINING PRESSURE	2,000	psf
DRY DENSITY		106	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	SILTY CLAY (CL-ML), yellow-brown				SOURCE	B-4 @ 20 feet

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 10/02/14 | Project No. 731637002 | Figure C-7



Symbol	Sample Source	Classification
●	B-4 at 5.5 feet	CLAYEY SAND with GRAVEL (SC), yellow-brown

3093 BROADWAY
Oakland, California

LANGAN TREADWELL ROLLO

PARTICLE SIZE ANLYSIS

Date 10/02/14 | Project No. 731637002 | Figure C-8

APPENDIX D

SOIL CORROSIVITY TEST RESULTS

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

5 September, 2014

Job No.1409000
Cust. No.10727

Mr. Nate Sherwood
Langan Treadwell Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111

Subject: Project No.: 731637002-700-0.30.0
Project Name: 3093 Broadway
Corrosivity Analysis – ASTM Methods

Dear Mr. Sherwood:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on September 02, 2014. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified "corrosive" and Sample No.002 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a detection limit of 15 mg/kg.

The sulfate ion concentrations reflect none detected with a detection limit of 15 mg/kg.

The pH of the soils ranged from 7.28 to 7.53, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 330 to 430-mV. Sample No.001 is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions, and Sample No.002 is indicative of aerobic 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 McMillen for

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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

Client: Langan Treadwell Rollo
Client's Project No.: 731637002-700-0.30.0
Client's Project Name: 3093 Broadway
Date Sampled: Not Indicated
Date Received: 2-Sep-14
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 5-Sep-2014

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1409000-001	B-2, 1 @ 5'	330	7.53	-	1,500	-	N.D.	N.D.
1409000-002	B-3, 1 @ 5'	430	7.28	-	2,300	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	3-Sep-2014	3-Sep-2014	-	2-Sep-2014	-	3-Sep-2014	3-Sep-2014

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

DISTRIBUTION

1 electronic copy:	Mr. Stephen Siri ssiri@cityview.com
1 electronic copy:	Mr. Jim Yee jyee@vtbs.com
1 electronic copy:	Mr. Brad Hoogerwerf, SE brad@hoogerwerfegi.com
1 electronic copy:	Mr. Ryan Bernal, PE, QSD, LEED AP rbernal@bkf.com
1 electronic copy:	Mr. Minh Tran mtran@bkf.com
1 electronic copy:	Mr. Sal Italiano, PE sal@spi-consulting.com
1 electronic copy:	Mr. Martin Turner mturner@johnstonemoyer.com
1 electronic copy:	Mr. Gary Woolford mturner@johnstonemoyer.com
1 electronic copy:	Mr. Ghazali Elamin melamin@johnstonemoyer.com
1 electronic copy:	Mr. Bill Johnstone bjohnstone@johnstonemoyer.com
1 electronic copy:	Mr. John Mavar, LEED AP jmavar@aol.com

QUALITY CONTROL REVIEWER:

Hadi J. Yap

Hadi J. Yap, PhD, GE
Senior Consultant