PRELIMINARY GEOTECHNICAL INVESTIGATION
PEERLESS GREENS MIXED-USE DEVELOPMENT
Berkeley, California

De Tienne Associates
San Francisco, California

23 May 2008
Project No. 4795.01

Treadwell&Rollo
Environmental and Geotechnical Consultants

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Mr. Darrell de Tienne
De Tienne Associates
3435 Cesar Chavez Street, Suite 312
San Francisco, California 94110

Subject: Preliminary Geotechnical Investigation
Peerless Greens Mixed-Use Development
Berkeley, California

Dear Mr. de Tienne:

We are pleased to submit our preliminary geotechnical investigation report for the proposed Peerless Greens mixed-use development in Berkeley, California. The project site is bound by Allston Way to the north, 5th Street to the east, Bancroft Way to the south, and Union Pacific railroad tracks to the west. Our preliminary geotechnical investigation was performed in accordance with our proposal dated 26 February 2008.

We understand the project will consist of demolishing existing buildings and constructing a partially to fully below-grade parking level. Buildings up to six stories high (75 feet) will be constructed above the parking level and will include residential, office, retail, research and development, and active art spaces. The development will also include new landscaping, courtyards, and the realignment of 4th Street to include a green park in the middle.

The site is generally blanketed by four to ten feet of high plasticity, stiff to very stiff clay fill. Beneath the fill, we encountered predominantly stiff to hard clay with varying sand and gravel content with zones of clayey sand and clayey sand with gravel to the maximum explored depths of 50 feet bgs. Several of the sand zones are potentially liquefiable in a major seismic event. Groundwater was measured at depths ranging from about Elevations 9 to 15 feet.

The primary geotechnical issues to be addressed for this project include the potential for very strong ground motion during an earthquake, the potential for excessive building settlement under static loads as well as due to liquefaction, relatively shallow groundwater, and the presence of highly expansive, undocumented fill near the surface.

On the basis of the results of our preliminary geotechnical investigation, we conclude the proposed project is feasible from a geotechnical standpoint. Our preliminary conclusion is that the proposed buildings may be supported on stiffened shallow foundations, such as mats or stiffened grid foundations. At the time this preliminary report was written, we did not have building loads from the project structural engineer. Our estimates of foundation settlement are based on assumed building loads and an approximate top-of-slab elevation of about 16.5 feet in the below-grade parking level. When actual building loads are available and if proposed floor elevations change, we should be informed so that the foundation analyses can be refined.

This report presents our preliminary recommendations regarding site preparation, fill placement, foundation design, seismic design, and other geotechnical aspects of the project. The preliminary conclusions and recommendations are based on limited subsurface exploration and laboratory testing and are not intended for final design. Final geotechnical design values should be confirmed by a detailed geotechnical investigation.

We appreciate the opportunity to work with you on this project. If you have any questions, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Treadwell & Rollo, Inc. for the proposed Peerless Greens Mixed-Use Development in Berkeley, California. The project site is approximately 630 feet by 580 feet in plan and is bound by Allston Way to the north, 5th Street to the east, Bancroft Way to the south, and Union Pacific railroad tracks (3rd Street) to the west, as shown on Figure 1.

The site is currently occupied by one- to two-story commercial and warehouse buildings of the former Peerless Lighting Corporation, the Davlin Paint Company, parking lots, one single-family home, and 4th Street, which runs through the middle of the site and is included in the proposed development. The Site Plan, Figure 2, shows the approximate location of existing structures and parking lots. The northeast corner of the site, which is occupied by a concrete-paved parking lot and the Vik Distributors building, is not currently included in the proposed development.

According to the drawing titled “Preliminary Topographic Survey”, prepared by Aliquot Planners, Civil Engineers, Surveyors, dated 19 March 2008, the existing ground elevation at the site varies between about 24.5 feet[1] in the southeast corner (5th Street and Bancroft Way) of the site, 22.5 feet at the northeast corner, 12.5 feet at the northwest corner, and 19 feet in the southwest corner. Also, there is a graded slope at the southern and southwestern portions of the site that decreases from about elevation 19 feet to about elevation 12 to 15 feet along the existing railroad right-of-way.

We understand the proposed mixed-use development will consist of two large development areas, each to be constructed over a concrete podium parking structure and consist of up to six-story buildings. One development area will be located east of 4th Street and the second will occupy the area west of 4th Street. The footprint of each proposed podium parking structures has an irregular shape, as indicated on Figure 2. The proposed development will be constructed above the podium parking level and will include a mixture of residential, office, retail, research and development, and active art spaces, as well as

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[1] All elevations reference City of Berkeley Datum and are based on the information described on the drawing titled “Preliminary Topographic Survey” prepared by Aliquot Civil Engineers, dated 19 March 2008.

additional parking at several locations. The proposed eastern structure will have a ground level finish floor elevation of about 26.5 feet. The proposed development will also include new landscaping, courtyards, and the realignment of 4th Street to include a green park in the middle.

The podium parking structure planned along the eastern portion of the property will have a top-of-slab finish floor elevation of 16.5 feet and building foundation subgrade level that may extend several feet below the podium floor. The existing site grades within proposed eastern podium parking footprint vary from about elevation 20 to 24.5 feet; therefore, we estimate that excavations on the order of 6 to 12 feet deep may be required to accommodate the proposed eastern podium structure.

The finish floor elevation of the western podium parking level is also planned at elevation 16.5 feet. However, the existing ground surface within the western portion of the site varies between about elevation 20 and 15.5 feet. Therefore, the western podium parking structure will require excavations of up to about six feet in depth.

At the time this report was being prepared, building loads and column spacing are not available. However, in performing our preliminary settlement estimates, we have assumed approximate column loads of 1,800 kips and a garage column spacing of 40 feet. Once the building loads and column spacing have been established, we should re-evaluate foundation settlement estimates.

The site is located within an area of “historic occurrence of liquefaction” as defined on the State of California Seismic Hazard Zone Map of the Oakland West Quadrangle, dated 14 February 2003. California Geological Survey (CGS) has provided recommendations for procedures and report content for site investigations performed within seismic hazard zones in Special Publication 117 (SP 117), entitled “Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California”, dated 13 March 1997. Our geotechnical investigation was performed in general accordance with SP 117.

The site has a known history of commercial and industrial uses dating back to the 1940s. Fugro West, Inc. (Fugro) has performed environmental studies of portions of the site, including the property at 2220 Fourth Street. Studies by Fugro, as well as by others, have confirmed that releases of solvents, heavy metals, and petroleum hydrocarbon compounds have impacted the soil and groundwater in various locations in the site vicinity. Some remedial efforts have been undertaken in the past, and ongoing monitoring and further environmental studies are currently being performed by Fugro.

2.0 SCOPE OF SERVICES

Our preliminary geotechnical investigation was performed in accordance with our proposal dated 26 February 2008. Our scope of services consisted of exploring subsurface conditions at the site by drilling borings and performing cone penetration tests (CPTs), performing laboratory tests and engineering analyses, and developing preliminary conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- appropriate foundation type(s) for proposed buildings
- preliminary capacities for foundation type(s), including vertical and lateral capacities
- preliminary estimates of foundation settlement
- preliminary design parameters for temporary shoring and permanent below-grade walls
- site grading, including criteria for fill quality and compaction
- flexible pavement design criteria
- 2007 California Building Code (CBC) site class and seismic design parameters
- soil corrosivity
- construction considerations.

3.0 PRELIMINARY FIELD INVESTIGATION AND LABORATORY TESTING

We explored the subsurface conditions at the site by drilling four borings (designated B-1 through B-4) and performing six cone penetration tests (designated CPT-1 through CPT-6). The approximate locations of the borings and CPTs are presented in the Site Plan, Figure 2.

Prior to performing the field investigation, we:

- obtained a soil boring permit from the City of Berkeley Toxics Management Division (TMD)
- obtained a street use permit from the City of Berkeley Engineering Department
- notified Underground Service Alert (USA)

- verified the boring and CPT locations were clear of underground utilities using an independent utility locating contractor.

3.1 Borings

Borings B-1 through B-5 were drilled by Exploration Geoservices of San Jose, California on 13 March 2008 using a truck-mounted Mobile B-42 rig equipped with eight-inch-outside-diameter, hollow-stem flight augers. The borings were advanced to a depth of 50 feet below the existing ground surface (bgs). During drilling, our field engineer logged the borings and obtained samples of the soil encountered for further classification and laboratory testing. Logs of B-1 through B-4 are presented in Appendix A as Figures A-1 through A-4. The soil was classified in accordance with the classification system shown on Figure A-5.

Soil samples were obtained using a Sprague and Henwood (S&H) split-barrel sampler with 3.0-inch outside and 2.5-inch inside diameter, as well as a Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners. The S&H sampler contained 2.43-inch inside diameter brass liners. Both samplers were driven with a 140-pound, hydraulic wire-line hammer falling about 30 inches. The blow counts required to drive the S&H and SPT samplers the final 12 inches of an 18-inch drive were converted to approximate Standard Penetration Test (SPT) N-values using conversion factors of 0.6 and 1.0, respectively, and are shown on the boring logs. Upon completion of drilling, the borings were backfilled with cement grout in accordance with the requirements of TMD.

The soil cuttings generated during drilling were temporarily stored on site in 55-gallon drums. Samples of the drummed cuttings were obtained and tested for chemical contaminates. The soil cuttings were characterized as non-hazardous and subsequently disposed at an off-site facility.

3.2 Cone Penetration Tests

On 13 March 2008, Gregg Drilling and Testing, Inc. of Martinez, California, performed CPT-1 through CPT-6 by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe into the ground to a depth of 50 feet bgs. Load cells and a pore pressure transducer within the cone continuously measured soil parameters during the entire depth of each probing. The accumulated data was processed by computer to provide engineering information, such as the soil behavior type and approximate strength characteristics of the soil encountered.

The CPT logs showing tip resistance, friction ratio, equivalent SPT N-value, shear strength, internal friction angle, and soil behavior type are presented in Appendix A on Figures A-6 through A-11. A classification chart for the CPTs is included as Figure A-12.

3.3 Laboratory Testing

We re-examined the soil samples obtained from our borings to confirm field classification and selected representative samples for geotechnical laboratory testing. Laboratory tests were performed to evaluate the engineering properties of onsite soil, including:

- moisture content and dry density
- fines content
- Atterberg limits
- undrained shear strength
- resistance value (R-value)
- consolidation characteristics.

The laboratory test results are presented on the boring logs and in Appendix B.

3.4 Soil Corrosivity Testing

Corrosivity testing was performed on samples obtained from boring B-2 at a depth of six feet bgs and boring B-3 at a depth of nine feet bgs. The corrosivity test results were evaluated by Environmental Technical Services of Petaluma, California and the results and evaluation are presented in Appendix B.

4.0 SUBSURFACE CONDITIONS

The results of our preliminary subsurface investigation indicate the site is generally blanketed by 4 to 10 feet of stiff to very stiff clayey fill. Atterberg limits tests indicate this near-surface fill has a moderate to high expansion potential. Expansive soil undergoes changes in volume with changes in moisture content (i.e., it shrinks when dried and swells when wetted).

Beneath the fill, we encountered predominantly stiff to hard clay with varying sand and gravel content to the maximum explored depths of 50 feet bgs, with zones of clayey sand and clayey sand with gravel in some of the borings and CPTs. The sand layers generally contain relatively high clay content.

Four groundwater monitoring wells, designated as MW-1 through MW-4 in this report, are present at the site and were installed as part of previous investigations by others. The approximate locations of the wells are shown on the Site Plan, Figure 2. Water level measurements were taken in wells MW-3 and MW-4 on 17 April 2008 and in wells MW-1 and MW-2 on 15 May 2008. A pore pressure dissipation test was performed in CPT-3 on 13 March 2008. Table 1 presents the estimated groundwater elevations at the site measured during our investigation.

TABLE 1
Groundwater Level Measurements

Well	Date of Measurement	Approximate Elevation at Top of Well Box (feet)	Approximate Elevation of Groundwater (feet)
MW-1	05/15/2008	19.4	9.2
MW-2	05/15/2008	20.5	10.4
MW-3	04/17/2008	21.5	12.5
MW-4	04/17/2008	24.5	15
CPT-3	03/13/2008	--	13

Additional water level measurements were taken in each test hole immediately after performing our borings and CPTs. However, these holes were not left open long enough for groundwater to fully recharge and stabilize sufficiently. Therefore, these water levels, as presented on the boring and CPT logs, may not represent stabilized groundwater levels.

On the basis of our evaluation of groundwater levels at various locations across the site, we conclude that the groundwater surface appears to slope down toward the west and southwest at gradients of 1.0 to 1.5 percent. Additional monitoring points and further long-term groundwater level measurements will be useful for defining groundwater contours and seasonal fluctuations of the groundwater level at the site.

5.0 REGIONAL SEISMICITY

The major active faults in the area are the Hayward, Rodgers Creek, Calaveras, and San Andreas Faults. These and other faults of the region are shown on Figure 3. For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[2] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2007) and Cao et al. (2003)] are summarized in Table 2.

TABLE 2
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
North Hayward	4	Northeast	6.5
Total Hayward	4	Northeast	6.9
Total Hayward-Rodgers Creek	4	Northeast	7.3
South Hayward	9	East	6. 7
Mt Diablo	23	East	6.7
San Andreas - 1906 Rupture	25	West	7.9
San Andreas - Peninsula	25	West	7.2
San Andreas- North Coast South	26	West	7.5
Total Calaveras	26	East	6.9
Concord/Green Valley	26	East	6.7
Rodgers Creek	28	Northwest	7.0
Northern San Gregorio	30	West	7.2
Total San Gregorio	30	West	7.4
West Napa	34	North	6.5
Greenville	41	East	6.9
Monte Vista-Shannon	48	South	6.8
Point Reyes	48	West	6.8
Great Valley 6	49	East	6.7

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

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 99 km from the site.

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

The 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 3.

TABLE 3
WGCEP (2007) 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 DISCUSSION AND PRELIMINARY CONCLUSIONS

On the basis of our preliminary geotechnical investigation, we conclude the proposed development is feasible from a geotechnical standpoint. The primary geotechnical issues to be addressed for the project include:

- the potential for strong to very strong ground motions at the site during an earthquake
- the presence of potentially liquefiable deposits beneath the project site
- the presence of moderately to highly expansive, undocumented near-surface fill
- foundation settlement
- relatively shallow groundwater.

6.1 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failures, such as those associated with soil liquefaction,[3] lateral spreading,[4] and cyclic densification.[5] We used the results of the subsurface investigation to perform a preliminary evaluation of these phenomena occurring at the project site. The results of our preliminary seismic hazards evaluation are presented below.

6.1.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward Fault. However, ground shaking from future earthquakes on any of the nearby faults may be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, magnitude and duration of the earthquake, and specific subsurface conditions. We estimate that ground shaking at the site during a large earthquake on one of the nearby active faults discussed in Section 5.0 will be strong to very strong.

6.1.2 Soil Liquefaction and Associated Hazards

When saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength due to a transient rise in excess pore pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. In general, most of the soil encountered was cohesive and not prone to liquefaction, with the exception of a soil layer encountered in CPT-2 between the depth of 16 and 26 feet bgs, where several one- to three-foot-thick layers were identified as potentially liquefiable. The material within this zone was assigned an interpreted soil behavior type of sandy silt and silty sand and was characterized as being liquefiable using the proceedings of the NCEER workshop on the evaluation of liquefaction resistance of soils (NCEER 1997). Based on our preliminary evaluation, we conclude this zone may liquefy during a major seismic event on one of the nearby faults and we estimate liquefaction-induced ground settlement from this zone will be about two inches.

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[3] Liquefaction is a phenomenon in which saturated (submerged), cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand and silt of low plasticity that is relatively free of clay.

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

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

Also, thin potentially liquefiable seams were encountered in CPT-1, CPT-3, CPT-4, CPT-5, and CPT-6, generally between depths of 17 and 28 feet bgs. Based on our preliminary evaluation, we conclude these seams may liquefy during a major seismic event on one of the nearby faults, resulting in liquefaction-induced ground settlement of about 1/4 to 1/2 inch.

We evaluated the potential for lateral spreading to occur at the site by evaluating whether continuous layers of potentially liquefiable soil are present at the boring and CPT locations. The results of our preliminary evaluation indicate that liquefiable soil layers were not present at the boring locations, and the potentially liquefiable soil layers encountered at the CPT locations are generally thin, discontinuous, and have relatively high fines content. Based on our evaluation of the subsurface data, we preliminarily conclude the potential for lateral spreading to occur at the site is low.

6.1.3 Cyclic Densification

Seismically-induced compaction or cyclic densification of non-saturated (i.e. above the groundwater table), loose to medium dense, cohesionless sand and/or gravel due to earthquake vibrations may cause differential settlement. We did not encounter cohesionless sand or gravel above the water table at the site. Therefore, we conclude that the potential for settlement from cyclic densification is nil.

6.1.4 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 building damage resulting from fault offset at the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is also low.

6.2 Expansive Near-Surface Fill

The site is blanketed by moderately to highly expansive clay fill. Expansive near-surface soil is subject to high volume changes during seasonal fluctuations in moisture content, which can cause cracking of foundations, floor slabs, and exterior slabs that are supported near the surface. These effects can be mitigated by moisture conditioning the expansive soil, placing non-expansive fill below slabs and foundations, supporting foundations below the zone of severe moisture change, and/or designing foundations and slabs to resist ground movements associated with the volume changes. The non-expansive fill layer may consist of a variety of materials, including granular soil or lime-treated fill. Preliminary recommendations for foundation support, subgrade preparation, and placement and compaction of non-expansive fill are presented in Section 7.0.

6.3 Foundations and Differential Ground Movement

The primary geotechnical issues affecting the selection of an appropriate foundation system for support of the proposed buildings include:

- erratic settlement associated with the upper 4 to 10 feet undocumented fill at the site
- shrinking and swelling of the near-surface expansive fill
- total and differential settlement associated with the consolidation of moderately compressible soil underlying the site when subjected to new fill and building loads
- total and differential settlement resulting from post-liquefaction densification of granular soil layers following a major seismic event.

The proposed building sites are currently blanketed by a 4- to 10-foot-thick layer of undocumented fill. At three of four boring locations, the undocumented fill was found to be highly expansive. Considering that an approximate 6- to 12-foot-deep excavation will be required to accommodate the easternmost podium parking structure, we preliminarily conclude that new foundations for the eastern development area will gain support in native soil below the fill, and differential ground movements associated with undocumented and expansive fill should not adversely impact the building foundations within the eastern development area.

At the western portion of the proposed development, we estimate that up to six feet of excavation will be required to accommodate the proposed podium structure. Based on our evaluation of the subsurface conditions, we preliminarily conclude that the western portion of the development area will be partially underlain by expansive undocumented fill, and the proposed foundations for the western podium structure could be adversely impacted by the presence of moderately to highly expansive clayey fill. To reduce the potential for differential ground movement associated with the undocumented clayey fill, we preliminarily conclude that clayey fill beneath the bottoms of the proposed podium foundations and slab-on-grade floors should be overexcavated and replaced with non-expansive fill. Non-expansive fill may consist of import select fill or lime-treated on-site clay fill.

If the proposed buildings over podium parking structures are supported on conventional, shallow spread-type footings that gain support in either native soil or compacted fill, we estimate that total and differential foundation settlements will occur under static conditions as the moderately compressible soil underlying the site consolidates under new fill and building loads. We estimate the consolidation-related settlement will be about two inches and differential settlement will be on the order one inch between columns. These settlement estimates are based on an assumed bottom-of-footing elevation of about 13.5 feet. As discussed in Section 6.1.2, additional foundation settlement associated with liquefaction-induced reconsolidation should be expected to occur during and immediately after a large earthquake on one of the nearby active faults.

The governing factor for foundation design is the potential for total and differential settlement across the proposed building sites. The most suitable foundation type for the proposed buildings depends on the cost of the foundation system, as well as, the amount of earthquake-induced damage that would be acceptable. For this project, we preliminarily conclude that a stiffened, shallow foundation system bearing in native soil or on a layer of compacted non-expansive fill may be used for support the proposed buildings. Acceptable stiffened shallow foundation systems include a grid of interconnected, well-reinforced continuous footings or a reinforced concrete mat. The building floors would either consist of concrete slabs-on-grade if a grid foundation system is used, or the mat. Both stiffened foundation systems would reduce the potential for differential settlement by transferring loads over “soft spots” in the underlying subgrade. Some settlement-related damage to the building superstructure should be expected during a major earthquake, as well as, some cracking of the floor slab; however, we believe this damage would be repairable. We estimate total settlement of the proposed buildings supported on properly designed stiffened foundations bearing on native soil or compacted non-expansive fill will be on the order of two inches for static loading conditions. Differential settlement will be controlled by the column spacing and the stiffness of the foundation system.

Alternatively, to further reduce the potential for differential foundation settlement, we conclude the proposed buildings may be supported on shallow foundations that gains support on soil-cement columns or compacted aggregate piers (such as Geopiers), or the proposed structures may be supported on deep foundations, such as driven piles, drilled piers, or a proprietary deep foundation system (such as torque down piles) that are acceptable to the project geotechnical engineer.

Based on our preliminary discussions with the design team, we understand a stiffened shallow foundation system resting on native soil or compacted non-expansive fill is preferable. However, if requested, we can provide design recommendations for supporting the proposed buildings on shallow foundations bearing on either soil-cement columns or compacted aggregate piers, or on deep foundations.

6.4 Waterproofing and Moisture Proofing Considerations

At the eastern portion of the site, the groundwater level may rise above the bottom of the stiffened shallow foundation system (estimated at about Elevation 13.5 feet) and may potentially rise above the podium finish floor elevation of 16.5 feet. A preliminary design groundwater level at elevation 17 feet should be used for the eastern podium structure. To reduce the potential for water and/or water vapor to rise into the eastern podium parking area, we preliminarily conclude that the walls and floor of the eastern podium structure should be waterproofed and designed to resist horizontal and vertical hydrostatic forces associated with a preliminary design groundwater elevation at 17 feet. Also, we conclude that further studies should be performed to define the groundwater contours across the site and monitor the groundwater level for seasonal fluctuations and annual variations.

At the western development area, a preliminary design groundwater elevation of 12 feet should be for this project. This preliminary design groundwater level is about 1.5 feet below the proposed bottom of the proposed foundation subgrade and about 4.5 feet below the podium finish floor elevations. Therefore, if water vapor or moisture intrusion into the proposed northwestern podium parking area is a concern, a water vapor retarder system should be used beneath the podium floor slab and podium walls should be moisture-proofed.

6.5 Construction Considerations

Based on the results of our subsurface investigation, we preliminarily conclude that the soil at the eastern and western development areas can be excavated using standard earth moving equipment. If site grading is scheduled for rainy season, the native soil may be too wet to achieve adequate compaction during fill placement. To facilitate the placement and compaction of fill at the site, either the wet soil should be moisture-conditioned (dried) or lime-treated to improve the engineering characteristics of the wet clayey soil.

Based on an estimated foundation subgrade elevation of 13.5 feet and an assumed preliminary design groundwater elevation of 17 feet for the eastern development area, we believe groundwater may be encountered during the mass grading of the eastern podium structure. A temporary dewatering system will likely be needed to facilitate the construction of the below-grade portions of the buildings. The exposed subgrade is likely to be saturated and easily disturbed by construction traffic. The water should be pumped down below the foundation subgrade and protected prior to the placement of waterproofing material and steel and concrete for the proposed foundation system.

7.0 PRELIMINARY RECOMMENDATIONS

Preliminary recommendations for grading, foundation design, pavement design, seismic design, and other geotechnical aspects of this project are presented in the following sections. These recommendations are for planning purposes only and should be re-evaluated based on the results of a final geotechnical investigation.

7.1 Site Preparation and Fill Placement

7.1.1 Site Preparation

At the start of construction, the existing buildings should be demolished and removed, and site should be stripped of all vegetation and pavement. Organic soil generated during stripping should be removed from site or stockpiled for later use in landscape areas, if approved by the architect. Existing pavement materials, such as Class 2 aggregate base, may be segregated and reused at the site. From a geotechnical standpoint, Portland cement concrete from the former buildings and exterior slabs, and asphalt concrete may be crushed, processed, and used as a granular select fill or recycled aggregate base, provided it meets the gradation requirements. The environmental aspects of reusing crushed Portland cement concrete and asphalt concrete at the site should be discussed with the project architect and environmental consultant.

Any foundation elements and former utilities should be removed. Underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the proposed construction, they may be abandoned in-place, provided the lines are filled with lean concrete or cement grout to the property line. Abandoned underground utility lines should be excavated and removed from the site and the resulting excavations should be properly backfilled as described in subsequent sections of this report.

After clearing and stripping activities are completed, the proposed development areas should be excavated to remove existing expansive undocumented fill, and accommodate the below-grade portions of the concrete podium parking structures. We estimate that the eastern development site will require an approximately 6- to 12-foot-deep excavation and the western development area will require up to about six feet of excavation.

The proposed excavations can be constructed using a cut slope with a maximum inclination of 1-1/2:1 (horizontal to vertical). If the excavation for the proposed below-grade level cannot be sloped due to space limitations, shoring will be required to laterally restrain the sides of the excavation and limit the movement of adjacent improvements. At the northwestern corner of the site, the planned podium footprint lies immediately adjacent to existing neighboring buildings. Further investigation will be necessary at this location to investigate for the presence of undocumented and expansive fill and evaluate whether underpinning of the neighboring building foundations will be necessary to accommodate the excavation for the proposed podium structure. Preliminary shoring and underpinning design recommendations are presented in Section 7.4.

7.1.2 Subgrade Preparation

We should check the condition of the soil subgrade at the bottom of the planned foundation excavations to check that suitable native soil is present in the excavations. If undocumented expansive fill is present at the foundation level, it should be overexcavated to a depth of 3 feet below foundation subgrade and replaced with compacted non-expansive fill, such as select fill or lime-treated on-site fill. Below the 3-foot overexcavation, if localized soft areas are encountered, it should be overexcavated and removed and the resulting excavations should be filled with either lean concrete or a combination of geogrid (Mirafi BasXgrid 11 or equivalent) and compacted granular fill. Subsequently, the exposed foundation soil subgrade should be kept moist and protected from damage from constructed equipment. A 4-inch-thick layer of lean concrete mud slab may be used to protect the soil subgrade and facilitate the placement of waterproofing (at the eastern development area only) and reinforcing steel and concrete for the podium foundations and floor slabs.

In areas to receive new fill or site improvements (excluding podium foundation locations where suitable native soil is exposed), the soil subgrade should be scarified to a depth of at least 12 inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to at least 90 percent relative compaction.[6] The soil subgrade should be kept moist until it is covered by fill.

For exterior concrete slabs, site grading should accommodate at least a 12-inch-thick layer of non-expansive soil beneath the proposed slabs. If a grid-type foundation with concrete slab-on-grade is used for the western building, grading should accommodate for least two feet of potential excavation of undocumented fill and replacement with non-expansive soil beneath the proposed podium structures. The non-expansive soil may consist of select fill or lime-treated onsite soil.

7.1.3 Fill Quality and Compaction

If the native soil is to be used as general site fill, it should be moisture-conditioned to at least three percent above optimum moisture content, placed in horizontal lifts not exceeding eight inches in loose thickness, and compacted to at least 90 percent relative compaction. Select fill should consist of soil that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and approved by the geotechnical engineer. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above the optimum moisture content, and compacted to at least 90 percent relative compaction. The upper six inches of the soil subgrade in pavement areas that will receive vehicular traffic should be compacted to at least 95 percent relative compaction. Where expansive clay is exposed at the pavement subgrade elevation, it should be moisture-conditioned to at least three percent above optimum moisture content and compacted to at least 90 percent relative compaction.

Samples of proposed imported fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material.

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[6] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-07 laboratory compaction procedure.

7.1.4 Utility Trenches

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. 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.

Where utilities enter the buildings, an impermeable plug consisting of lean concrete should be constructed. The purpose of the impermeable plug is to reduce the potential for groundwater from entering beneath the building pads.

7.2 Foundations

Stiffened shallow foundations, such as interconnected continuous footings or a reinforced concrete mat may be used for support of the proposed buildings and podium parking structures. The foundation should be constructed directly above the waterproofing membrane (eastern development area only) and mud slab that is used to protect the soil subgrade. The proposed foundations should gain support on suitable native soil, lean concrete, or non-expansive fill that has been compacted to at least 90 percent relative compaction. To limit excessive settlement, our preliminary recommendation is the stiffened shallow foundations should be designed for an allowable bearing pressure on the order of 2,000 to 3,000 psf, for dead plus live loads, with a one-third increase for total design loads, including wind or seismic forces. For elastic analyses of the foundation system and determination of deflections, we recommend a preliminary modulus of vertical subgrade reaction of 20 kips per cubic foot (kcf). The modulus will vary depending on the stress distribution and estimated settlement. After the foundation stresses on the soil have been calculated throughout the building footprint by the structural engineer, we should check the moduli to confirm they are still appropriate. Several iterations may be needed.

Lateral loads may be resisted by a combination of passive pressure on the embedded vertical faces of the stiffened shallow foundation system and friction along the bottom of the foundation elements. Passive resistance may be calculated using a uniform pressure of 2,000 psf. The upper foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.25. Where the mat is underlain by a vapor retarder or waterproofing membrane, a base friction of 0.15 should be used. The allowable passive resistance and base friction coefficient include a factor of safety of about 1.5

The foundation subgrade exposed by excavation should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the excavations should be maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will eventually heave, which may result in cracking and distress. In order to minimize disturbance to the foundation subgrade and to provide a working surface for placement of waterproofing and reinforcing steel, we recommend pouring a mud slab shortly after preparing the soil subgrade. A waterproofing expert should be consulted to determine the best method of protecting the waterproofing material.

The top of the mat foundation may be used as the parking garage floor, or a thin layer of concrete (topping slab) may be placed directly above the mat to provide a smooth wearing surface.

7.3 Waterproofing and Moisture Proofing

The foundation beneath the eastern half of the site should be designed to resist a hydrostatic uplift force associated with the design groundwater level of Elevation 17 feet. The foundation above western half of the site will likely be above the static groundwater level, based on our current measurements. The below grade walls and foundation of the eastern building should be waterproofed as recommended by the project architect or waterproofing consultant. It will likely not be necessary to waterproof the western building foundation. If water vapor transmission through the floor slab is undesirable (e.g., where floor covering will be placed), a capillary moisture break and a water vapor retarder may be installed beneath the floor. A capillary moisture break and vapor retarder are generally not required below parking slabs-on-grade because there is sufficient air circulation to limit condensation of moisture on the slab surface. However, it may be appropriate beneath storage, mechanical, electrical rooms, or any occupied spaces.

A capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. It would be appropriate for the vapor retarder to meet the requirements for Class C vapor retarders stated in ASTM E1745-97 and for the vapor retarder to be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. Design parameters for the gravel/crushed rock and sand are presented in Table 4. A materials testing inspector should be retained to observe the condition of the vapor retarder.

TABLE 4
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

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

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, we judge that one design parameter for the floor slab concrete be that it have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability may be increased by adding plasticizers.

Before the floor covering is placed, the contractor may check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.4 Temporary Shoring and Underpinning

Soldier-pile-and-lagging is an acceptable method to retain the excavation both below the sidewalks as well as adjacent buildings, provided it is designed to limit vertical and lateral displacements to acceptable levels. Where the shoring will be near existing neighboring buildings, underpinning of the adjacent foundations may be necessary. The shoring and underpinning should be designed by a shoring engineer.

We recommend the cantilevered soldier-pile-and-lagging shoring system should be designed using the lateral earth pressures presented in Figure 5. In locations where there are no adjacent buildings and sensitive underground utilities, the shoring system should be designed to resist an active equivalent fluid weight of 40 pcf. In locations where minimizing lateral deflections is critical, such as beneath an adjacent building or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 60 pcf above. These pressures are appropriate assuming groundwater is pumped down 3 feet below the bottom of excavation during construction. If traffic is planned within 10 feet of the top of the shoring system, a traffic increment consisting of a uniform (rectangular distribution) lateral pressure of 100 psf should be applied to the upper 10 feet of the wall. The recommended passive pressures on the embedded portion of the soldier piles are presented in Figure 5.

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The temporary shoring system should be designed by a shoring engineer. We should have the opportunity to review the geotechnical aspects of the proposed shoring system to ensure that it meets our requirements. During construction, we should have the opportunity to observe the installation of the shoring system and check the condition of the soil encountered during excavation.

7.5 Permanent Below-Grade Walls

We recommend permanent basement walls be designed to resist lateral pressures imposed by the adjacent soil and surcharge loads, such as vehicles. The permanent basement walls should be designed for the more critical of the following conditions:

- at-rest equivalent fluid weight of 60 pcf above the design groundwater level (Elevation 17 feet for the eastern podium and Elevation 12 feet for the western podium structure) and 95 pcf below
- active pressure of 40 pcf above the design groundwater elevation, 85 pcf below, plus a seismic increment of 14 times the height of the wall in psf, uniform pressure distribution.

Where traffic is expected within a distance equal to the height of the walls, the walls should be designed for an additional uniform lateral pressure of 100 psf to be applied over the entire height of the wall or the upper 10 feet, whichever is less.

The lateral earth pressures recommended for the wall above the water table are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe placed approximately one foot above the design water level. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or drain rock wrapped in filter fabric (Mirafi 140N or equivalent). We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. The pipe should be connected to a suitable discharge point. As an alternative to a collector pipe, the drainage panels can extend down below the groundwater. A one-foot cap of clay soil should be compacted at the top of the below-grade walls, in order to separate surface drainage from subsurface water.

If it is undesirable to install a drainage system behind building walls from an environmental standpoint, the walls may be designed for undrained conditions. Below-grade walls that are not drained should be designed for the pressures given for below groundwater throughout their entire depth.

To protect against water infiltration, below-grade walls should be waterproofed and water stops placed at all construction joints.

7.6 Exterior Concrete Flatwork

Exterior concrete slabs, not subject to vehicular loading, should be supported on at least 12 inches of select fill or lime/cement-treated soil; the upper six inches of select fill should consist of Class 2 aggregate base. If the subgrade is lime/cement-treated, we recommend supporting exterior concrete slabs on 12 inches of lime-treated soil (no aggregate base required). The subgrade and aggregate base should be compacted to at least 90 percent relative compaction and provide a smooth, non-yielding surface for support of the concrete slabs. Even with 12 inches of select fill, these slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to buildings, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

7.7 Preliminary Asphalt Concrete Pavement Design

The State of California resistance value (R-value) method for flexible pavement design was used to develop recommendations for pavement sections. We expect the soil subgrade in asphalt-paved areas may consist of the existing highly plastic clay. An R-value test was performed on a near-surface sample from boring B-4, which was moderately plastic with a plasticity index of 17. However, near surface soil in borings B-1 and B-3 are highly plastic, with plasticity indices of 35 and 33, respectively. Based on our experience with highly plastic soil, we selected an R-value of 5 for our preliminary asphalt pavement design analysis. If the subgrade soil will be lime/cement-treated, the R-value may be increased for pavement design.

Since the traffic demand is not known at this time we have assumed traffic indices (TI) of 7.5 for truck traffic in the city street, 5.5 for driveways and 4.5 for parking areas. The driveway traffic index assumes passenger car traffic and light to moderate truck traffic for a 20-year period. The exterior parking areas assume only passenger vehicles and occasional trucks for a 20-year period. Three recommended pavement design alternatives for these traffic indices are presented in Table 5.

TABLE 5
Preliminary Recommendations for Asphalt Pavement Sections
(assumed R-value of 5)

	Parking Areas (TI = 4.5)	**Driveways (TI = 5.5)**	**Truck Traffic (TI = 7.5)**
Asphalt Concrete	2.5	3	4.5
Class 2 Aggregate Base	10.0	12	6.0
Aggregate Subbase	-	-	12.0

Refer to Section 7.1 for our parking and roadway subgrade preparation recommendations. The aggregate base and subbase materials should conform to the current Caltrans Standard Specifications (Section 25 and 26) and be compacted to at least 95 percent relative compaction.

7.8 Seismic Design

For seismic design in accordance with the provisions of 2007 California Building Code (CBC) we recommend the following:

- Maximum Considered Earthquake (MCE) Ss and S1 of 1.65g and 0.61g, respectively.
- Site Class D
- Site Coefficients FA and FV of 1.0 and 1.5
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, SMS, and at one-second period, SM1, of 1.65g and 0.91g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, SDS, and at one-second period, SD1, of 1.10g and 0.61g, respectively.

Several layers of potentially liquefiable material were encountered in the borings and CPTs. These zones generally appeared to be relatively thin and marginally liquefiable, as discussed in Section 6.1. However, additional subsurface exploration will be necessary to better characterize the vertical and lateral extents of these zones. There is a potential for encountering additional liquefiable materials beneath the proposed buildings during subsequent field investigations. If this is the case, the site class may need to be re-evaluated.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations provided in this report are preliminary in nature as only preliminary development plans are available and limited subsurface exploration was performed for this investigation. A final geotechnical investigation, including additional subsurface exploration and laboratory testing, should be performed prior to final design. Information required for this final investigation includes final building footprints, building loads, estimated site grades, and allowable settlement tolerances. Once the development plans have been provided and additional geotechnical sampling has been performed, we can provide final recommendations for foundation design, site grading, and seismic design, as required.

9.0 LIMITATIONS

The purpose of this preliminary geotechnical investigation is to address geotechnical issues regarding the proposed development at the site and to provide preliminary geotechnical recommendations for the project. The preliminary geotechnical conclusions and recommendations presented in this report are not intended for final project design. Final geotechnical design values should be confirmed by a detailed geotechnical investigation for the site.

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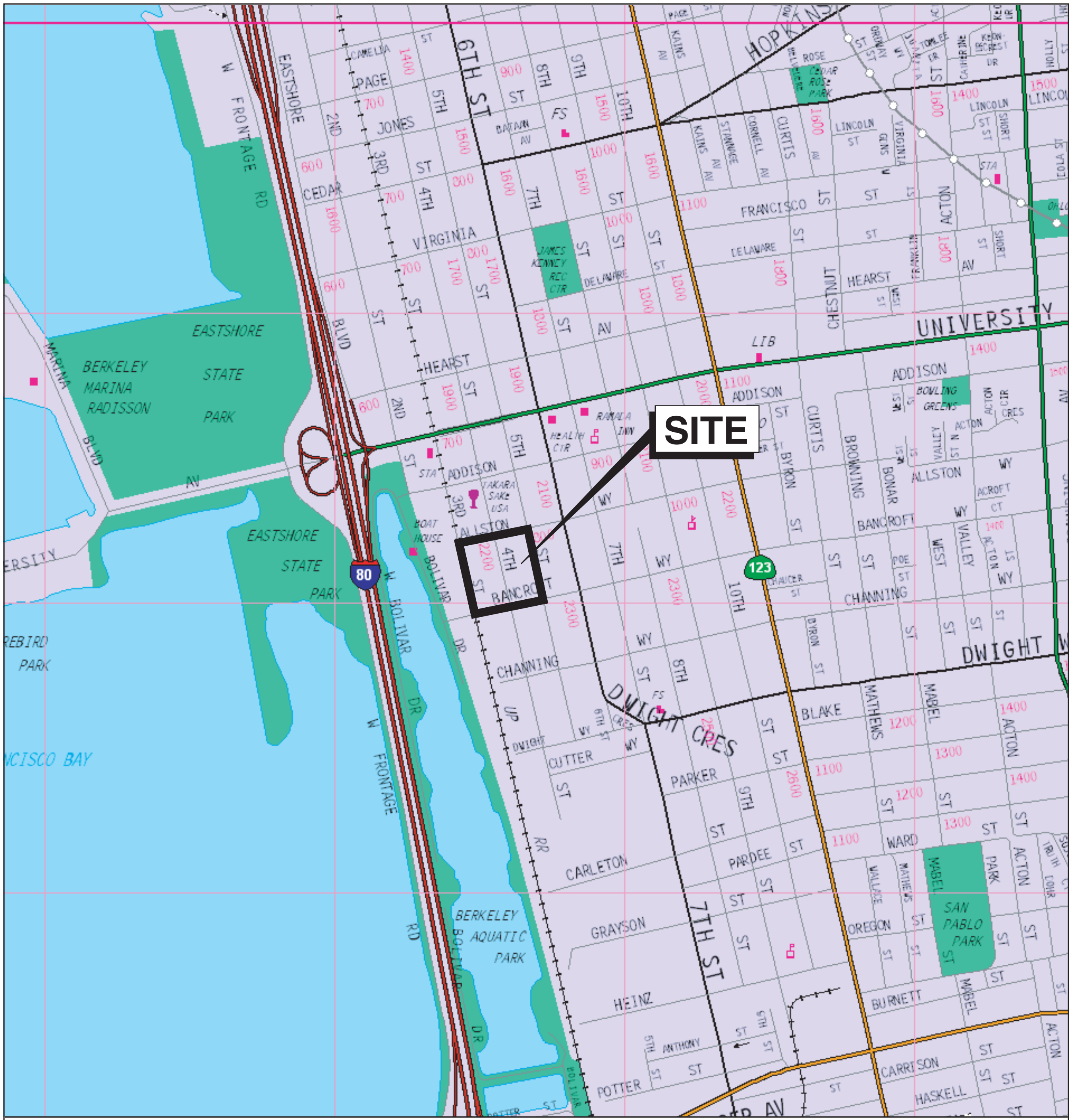
FIGURES



Base map: The Thomas Guide
Alameda County
2002

0 1/4 1/2 Mile
Approximate scale

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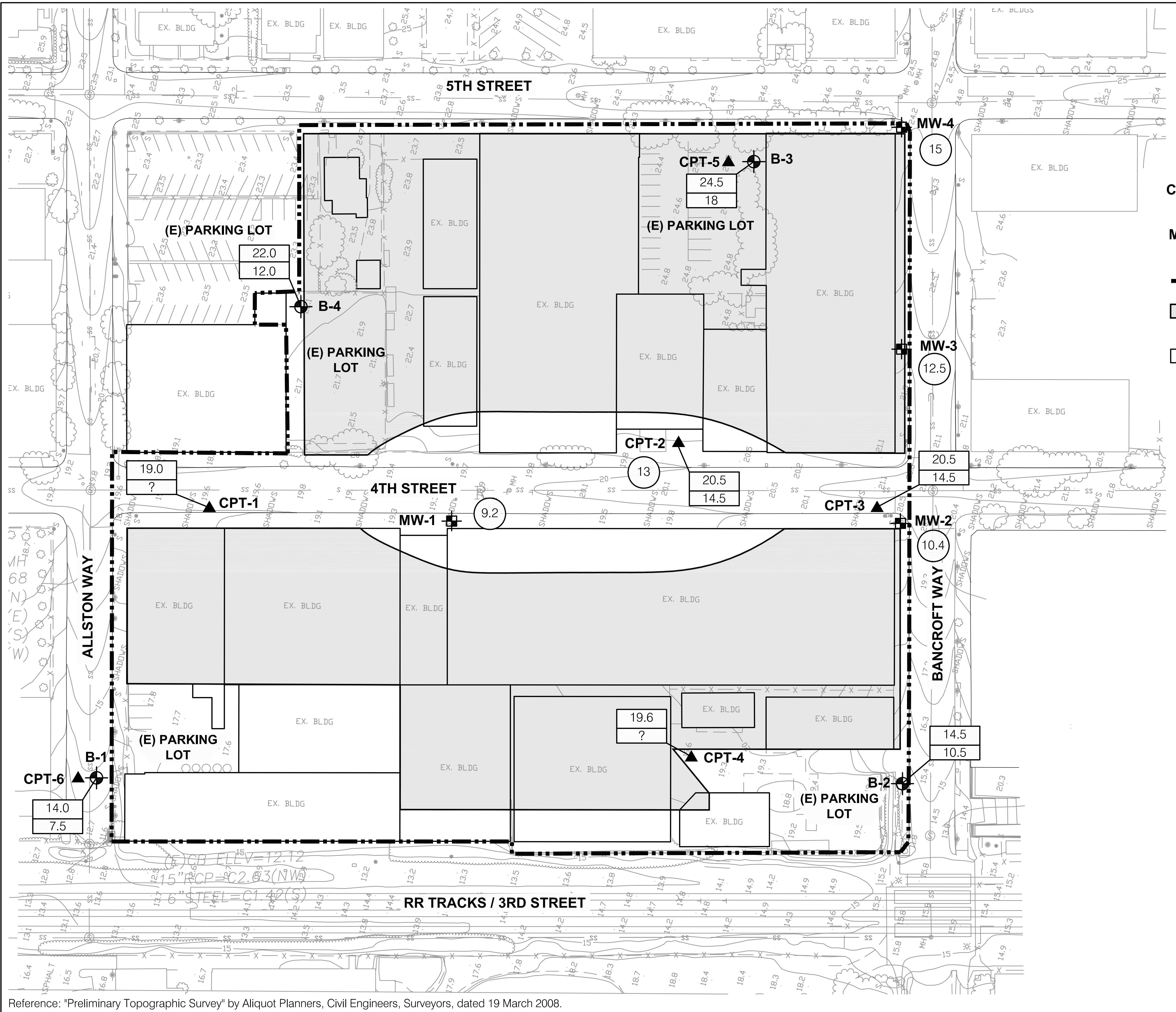
SITE LOCATION MAP

Date 05/22/08 | Project No. 4795.01 | Figure 1



Reference: "Preliminary Topographic Survey" by Aliquot Planners, Civil Engineers, Surveyors, dated 19 March 2008.

S:\Trgraphics–Oak\4700's\4795.01\OAK–4795.01_SITE PLAN.dwg 5/23/08

EXPLANATION

B-1 ⊕	Approximate location of boring by Treadwell & Rollo, Inc., March 2008
CPT-1 ▲	Approximate location of cone penetration test by Treadwell & Rollo, Inc., March 2008
MW-1 ⊞	Approximate location of groundwater monitoring well by others
━ · · ━	Approximate limits of project site
▨	Approximate location of proposed development areas
☐	Approximate location of existing building
14.0 7.5	Approximate ground surface elevation at test location Approximate elevation of bottom of existing fill material (Elevations reference City of Berkeley Datum)
(12.5)	Approximate elevation of groundwater level measured during this investigation

0 80 Feet

Approximate scale

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Berkeley, California

SITE PLAN

Date 05/21/08	Project No. 4795.01	Figure 2

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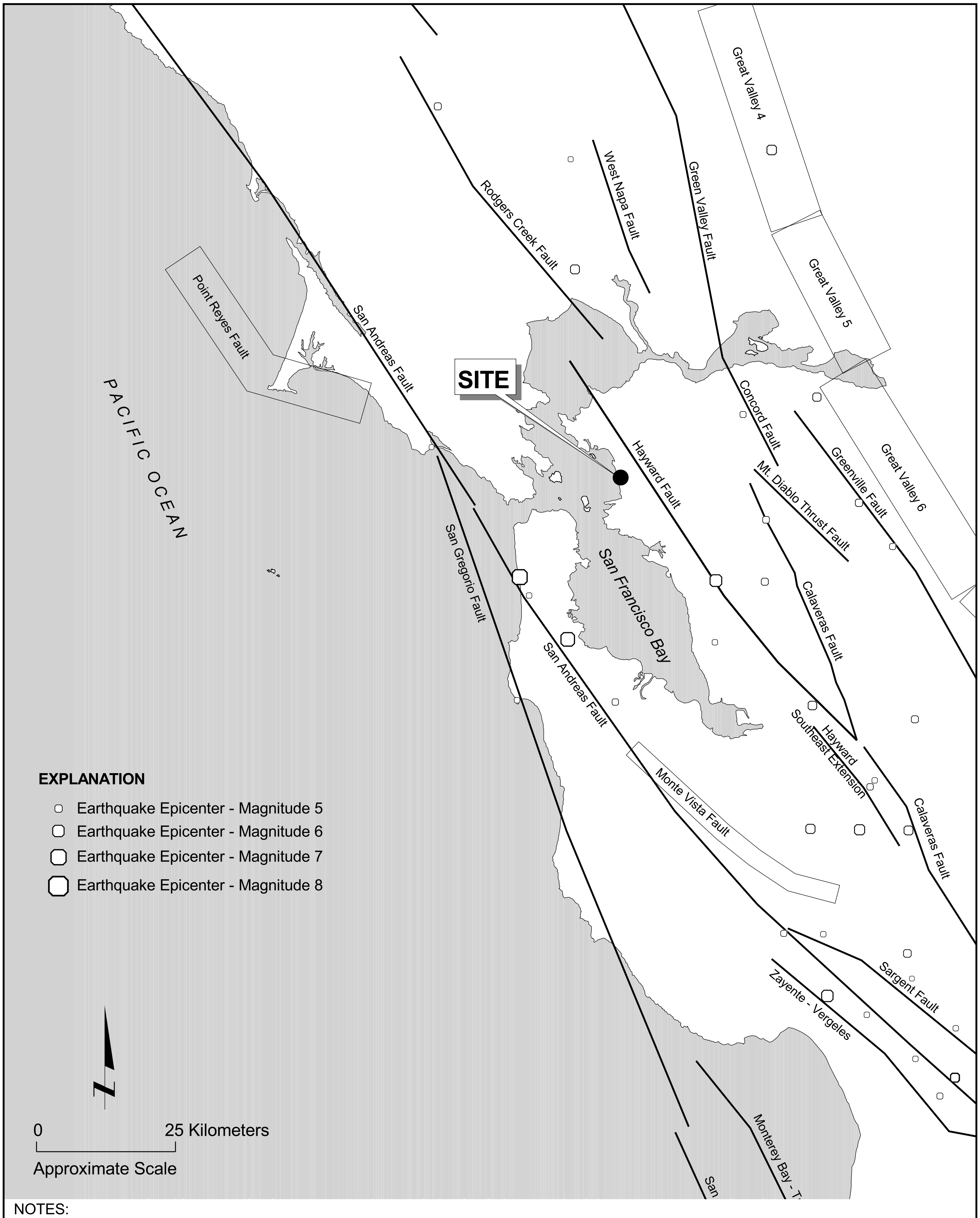

NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

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

Date: 04/18/08	Project No. 4795.01	Figure: 3

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.

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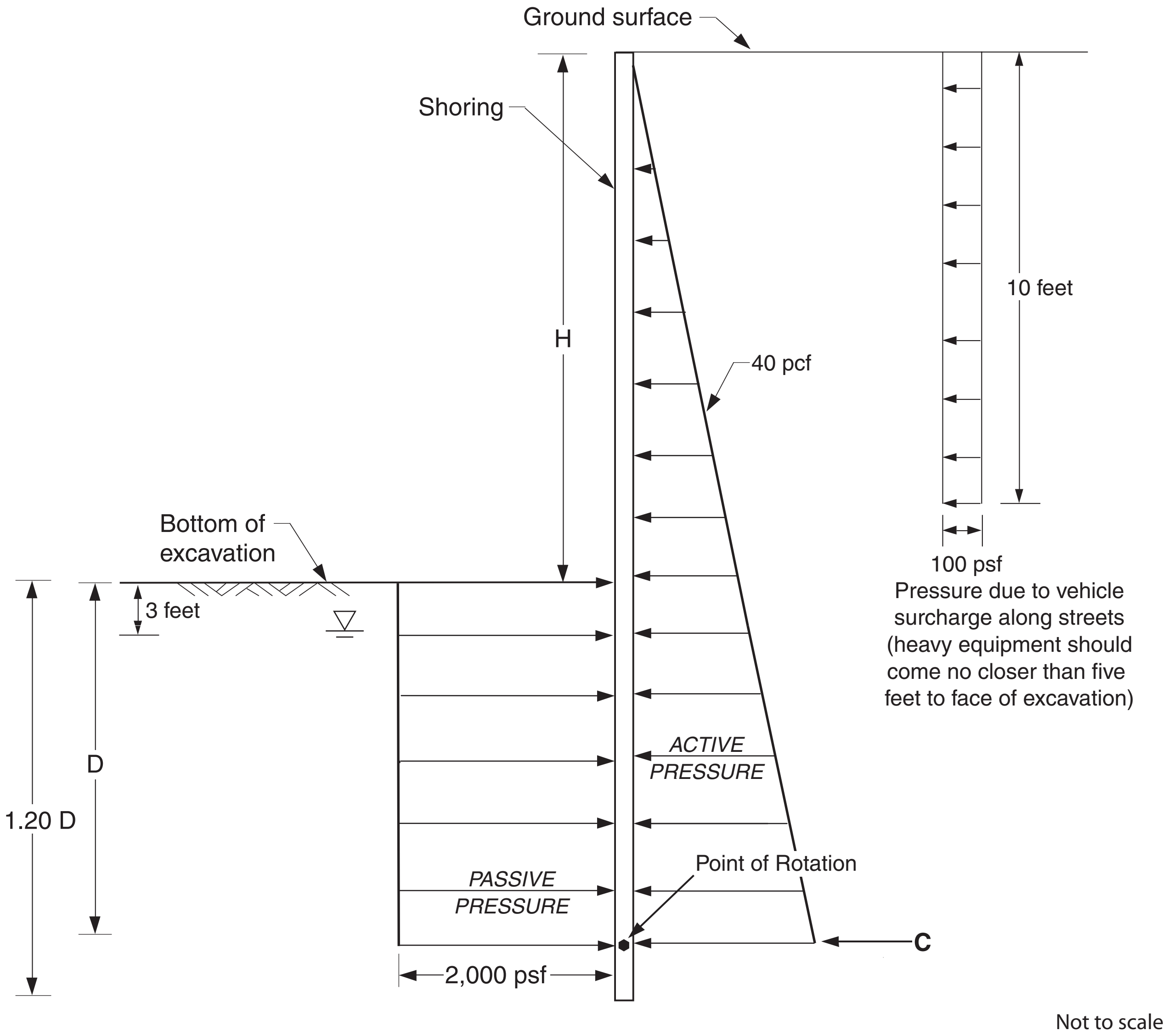
MODIFIED MERCALLI INTENSITY SCALE

Date: 04/18/08	Project No. 4795.01	Figure: 4



Notes:
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.
2. Passive pressures include a factor of safety of about 1.5.
3. For piles that are backfiled with structural concrete, passive pressures may be assumed to act over the pile spacing or three times the pile 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 pile diameter.
6. Calculated embedment depth, D, should be increased by at least 20 percent to obtain the design depth of penetration.
7. The recommended pressures do not include surcharges from adjacent buildings. Where shoring system is adjacent to a building or sensitive underground utility, at-rest lateral pressures should be used and surcharge pressure from footings should be added to the above shoring pressures.
8. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
9. Pressure diagram assumes shoring is pervious and groundwater table is lowered down to 3 feet below the bottom of excavation.

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PRELIMINARY DESIGN CRITERIA FOR CANTILEVERED SOLDIER-PILE-AND-LAGGING SHORING SYSTEM

Date 05/20/08	Project No. 4795.01	Figure 5

APPENDIX A
Logs of Borings and Cone Penetration Tests

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Date started: 3/13/08
Date finished: 3/13/08
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Schmidt
Coordinates: Northing:
Easting:

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 14 feet[2]		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						3 inches of asphalt concrete							
1					CH	CLAY (CH) olive-brown, very stiff, moist LL = 51, PI = 35; see Appendix B	FILL						
3	S&H	▨	11 15 17	19								22.0	104
6	S&H	▨	15 18 24	25		occasional subrounded gravel up to 1/4 inches in greatest dimension							
7					SC	CLAYEY SAND with GRAVEL (SC) brown, medium dense, wet, coarse sand, subrounded to subangular gravel up to 1/2 inches in greatest dimension							
9	S&H	▨	16 21 23	26								16.9	112
12	S&H	▨	11 18 23	25									
14	S&H	▨	7 12 10	13							44		
16	SPT	◪	7 6 10	16	CL	CLAY (CL) brown, very stiff, wet,							
19	S&H	▨	12 16 25	25				PP		3,500			
21						▽ 2:15 PM; 3/13/08 (not allowed to stabilize)							
24	S&H	▨	7 10 15	15		stiff to very stiff Consolidation Test; see Appendix B						26.0	97
29	S&H	▨	15 21 33	32		occasional black mottles very stiff to hard							
30													

Treadwell&Rollo

Project No.: 4795.01
Figure: A-1a

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, California

Log of Boring B-1

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
						CLAY (CL) (continued)						
31												
32												
33												
34	S&H	■	17 19 23	25		very stiff						
35					CL							
36												
37												
38												
39	S&H	■	15 27 40	40		occasional orange mottles, hard						
40												
41						SANDY CLAY (CL) brown with orange to black mottles, very stiff to hard, wet, fine to medium sand						
42												
43												
44	S&H	■	10 14 36	30								
45					CL							
46												
47												
48						hard						
49	S&H	■	16 27 30	34		occasional subangular gravel up to 1/2 inch in greatest dimension						
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet.
Boring backfilled with grout.
Groundwater measured at a depth of 21 feet immediately after drilling. Water level was not allowed to stabilized.
PP = Pocket Penetrometer.

[1] S&H blowcounts converted to equivalent SPT blowcounts using a factor of 0.6.

[2] Elevation references the City of Berkeley Datum.

Treadwell&Rollo

Project No.: 4795.01

Figure: A-1b

TEST GEOTECH LOG 479501.GPJ TR.GDT 5/23/08

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 3/13/08
Date finished: 3/13/08
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Schmidt
Coordinates: Northing:
Easting:

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 14.5 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY (CH) olive-brown, very stiff, moist, with seams of white clay						
2		■			CH	FILL						
3	S&H	■	9 16 22	23								
4						CLAY (CL) brown with orange mottles, very stiff, moist occasional subangular gravel up to 1/2 inch in greatest dimension Corrosion Test; see Appendix B						
5		■										
6	S&H	■	11 16 28	26							17.8	111
7												
8		■										
9	S&H	■	11 14 17	19		no sand or gravel, no orange mottles, occasional black mottles	PP		2,800			
10												
11		■				∇ 11:30 AM; 3/13/08 (not allowed to stabilize)						
12	S&H	■	11 12 14	16		yellow-brown, wet	TxUU	800	1,900		27.1	97
13		■				brown, stiff, occasional subangular gravel up to 1/2 inch, in greatest dimension Consolidation Test; see Appendix B						
14	S&H	■	8 9 12	13							31.8	92
15												
16												
17					CL							
18												
19	S&H	■	15 19 25	26		very stiff	PP		1,500			
20												
21												
22												
23												
24	S&H	■	14 15 30	27		no gravel or sand, no mottles	PP		2,300			
25												
26												
27												
28												
29	S&H	■	7 8 12	12		stiff Consolidation Test; see Appendix B					25.8	98
30												

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, 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
						CLAY (CL) (continued)						
31												
32												
33					CL	occasional orange mottles, very stiff, occasional subrounded gravel up to 1/4 inch in greatest dimension						
34	S&H	■	15 20 26	28								
35												
36												
37						CLAYEY SAND with GRAVEL (SC) brown, very dense, wet, medium sand, rounded gravel to 1/2 inch in greatest dimension						
38					SC							
39	S&H	■	27 47 48	57								
40	SPT	◩	14 15 18	33		SANDY CLAY (CL) brown, hard, wet, fine sand						
41					CL							
42						CLAY (CL) brown, very stiff, moist, low plasticity clay, orange mottles throughout						
43												
44	S&H	■	10 15 32	28								
45												
46					CL							
47												
48												
49	S&H	■	7 15 31	28		with gray mottles						
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet.
Boring backfilled with grout.
Groundwater measured at a depth of 11 feet immediately after drilling. Water level was not allowed to stabilized.
PP = Pocket Penetrometer.

[1] S&H blowcounts converted to equivalent SPT blowcounts using a factor of 0.6.
[2] Elevation references the City of Berkeley Datum.

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Project No.: 4795.01

Figure: A-2b

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 3/13/08
Date finished: 3/13/08
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H)
Logged by: K. Schmidt
Coordinates: Northing:
Easting:

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 24.5 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
					3 inches concrete						
1					CLAY (CH) black, stiff to very stiff, moist organic odor						
2											
3	S&H	8 11 14	15	CH	LL = 51, PI = 33; see Appendix B	TV		1,500		24.0	100
4					FILL						
5											
6	S&H	4 6 11	10		olive-gray, stiff, no sand						
7					CLAY (CL) gray-brown, stiff, moist						
8											
9	S&H	8 11 13	14		Corrosion Test; see Appendix B						
10					▽ 9:00 AM; 3/13/08 (not allowed to stabilize)						
11											
12	S&H	14 24 47	43		hard, wet with gravel up to 1/2 inch in greatest dimension						
13					SANDY CLAY (CL) orange-brown, very stiff, moist, fine to coarse sand						
14											
15				CL							
16	S&H	8 14 18	19								
17					CLAY with GRAVEL (CL) orange-brown, stiff, wet, subrounded gravel up to 1/2 inch in greatest dimension						
18											
19	S&H	5 8 10	11			TxUU	1,100	2,150		20.9	107
20											
21											
22				CL							
23											
24	S&H	17 15 16	19		very stiff						
25											
26											
27											
28				CL	SANDY CLAY (CL) orange-brown, stiff to very stiff, wet, fine to coarse sand, with seams of silty sand and gravel						
29	S&H	7 11 14	15								
30											

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Project No.: 4795.01
Figure: A-3a

PROJECT: **PEERLESS GREENS MIXED-USE DEVELOPMENT**
Berkeley, California

Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SANDY CLAY (CL) (continued)						
31												
32												
33												
34	S&H	■	5 12 20	19		orange and black mottles, very stiff						
35					CL							
36												
37												
38												
39	S&H	■	19 28 25	32		fewer orange and black mottles, hard, gravel up to 3/4 inch in greatest dimension						
40												
41						CLAY (CL) orange-brown, very stiff, wet						
42												
43												
44	S&H	■	8 18 25	26								
45					CL							
46												
47												
48												
49	S&H	■	4 17 11	17		gray-brown with orange and black mottles						
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet.
Boring backfilled with grout.
Groundwater measured at a depth of 10 feet immediately after drilling. Water level was not allowed to stabilized.
TV = Torvane.

[1] S&H blowcounts converted to equivalent SPT blowcounts using a factor of 0.6.

[2] Elevation references the City of Berkeley Datum.

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Project No.: 4795.01

Figure: A-3b

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, California

Log of Boring B-4

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 3/13/08
Date finished: 3/13/08
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Schmidt
Coordinates: Northing:
Easting:

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 22 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
						4 inches concrete						
1						CLAY (CL) brown, stiff, moist, trace subangular, yellow and dark brown gravel up to 1/2 inch in greatest dimension LL = 33, PI = 17; see Appendix B Resistance Value Test; see Appendix B						
2												
3	S&H	■	7 11 12	14							20.6	106
4												
5					CL	FILL						
6	S&H	■	17 28 30	35		orange-brown, hard, with pockets of gray clay, no gravel						
7												
8												
9	S&H	■	16 25 25	30		black and red-brown mottles, very stiff to hard, fewer pockets of gray clay						
10						SANDY CLAY (CL) brown with black mottles, very stiff, wet						
11					CL							
12	S&H	■	11 18 22	24			TxUU	800	2,600		21.5	106
13						CLAY with GRAVEL (CL) orange-brown, very stiff, wet, subangular gravel up to 1 inch in greatest dimension						
14	S&H	■	14 18 23	25								
15					CL							
16												
17						CLAY (CL) brown, very stiff, wet, clay, black and orange mottles throughout						
18					CL							
19	S&H	■	12 12 14	16		Consolidation Test; see Appendix B	PP TV		1,750 1,500		22.4	103
20						5:30 PM; 3/31/08 (not allowed to stabilize)						
21						∇						
22						CLAYEY SAND with GRAVEL (SC) yellow-brown, medium dense, wet, with subrounded gravel up to 1-1/2 inch in greatest dimension						
23												
24	S&H	■	11 11 27	23	SC	LL = 30, PI = 11; see Appendix B				33	17.4	114
25												
26												
27												
28						CLAYEY SAND (SC) brown, dense, wet, medium sand, trace subangular gravel up to 1/4 inch in greatest dimension						
29	S&H	■	10 23 28	31	SC					16		
30												

TEST GEOTECH LOG 479501.GPJ TR.GDT 5/23/08

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Project No.: 4795.01
Figure: A-4a

PROJECT: **PEERLESS GREENS**
MIXED-USE DEVELOPMENT
Berkeley, California

Log of Boring B-4

PAGE 2 OF 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
31	SPT	◩	4 4 7	11	CL	CLAY (CL) orange-brown, stiff, moist						
32												
33												
34	S&H	■	14 23 25	29		very stiff, with subangular gravel up to 1/2 inch in greatest dimension, orange mottles, around gravel						
35												
36												
37												
38					CL	very stiff, decreased gravel content						
39	S&H	■	12 14 20	20							23.8	102
40												
41												
42												
43						orange mottles very stiff, no gravel						
44	S&H	■	14 23 25	29			PP		1,800			
45												
46												
47						CLAYEY SAND (SC) brown with orange and black mottles, medium dense, wet, fine sand						
48					SC							
49	S&H	■	17 20 28	29						45		
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet.
Boring backfilled with grout.
Groundwater measured at a depth of 21 feet immediately after drilling. Water level was not allowed to stabilized.
PP = Pocket Penetrometer.
TV = Torvane.

[1] S&H blowcounts converted to equivalent SPT blowcounts using a factor of 0.6.
[2] Elevation references the City of Berkeley Datum.

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Project No.: 4795.01

Figure: A-4b

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample
⊙	Sampling attempted with no recovery
◫	Core sample
◉	Analytical laboratory sample
▥	Sample taken with Direct Push 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

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

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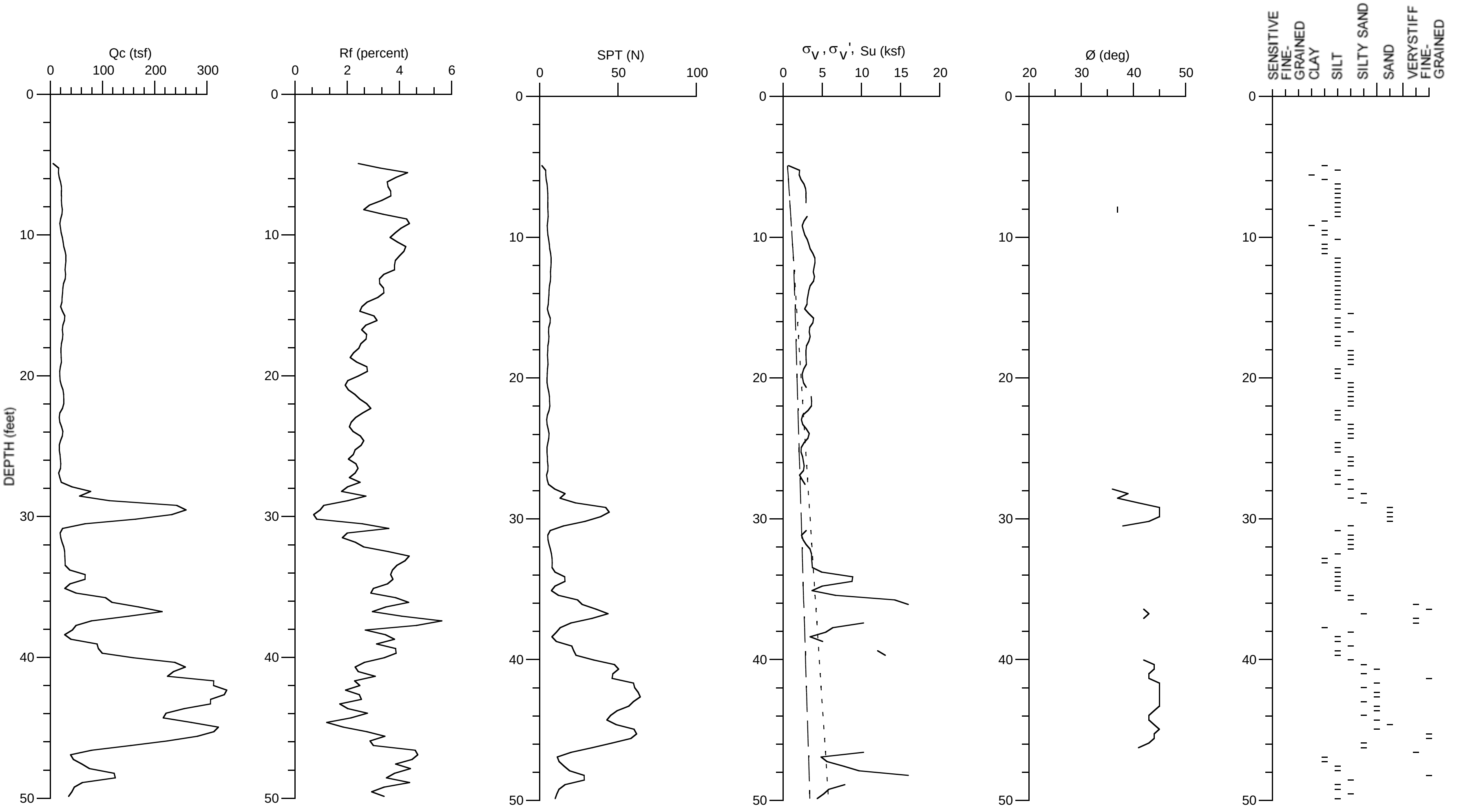
CLASSIFICATION CHART

Date 03/27/08 | Project No. 4795.01 | Figure A-5



— Effective vertical stress, σ_v'

- - - - Total vertical stress, σ_v

——— Undrained Shear Strength, s_u

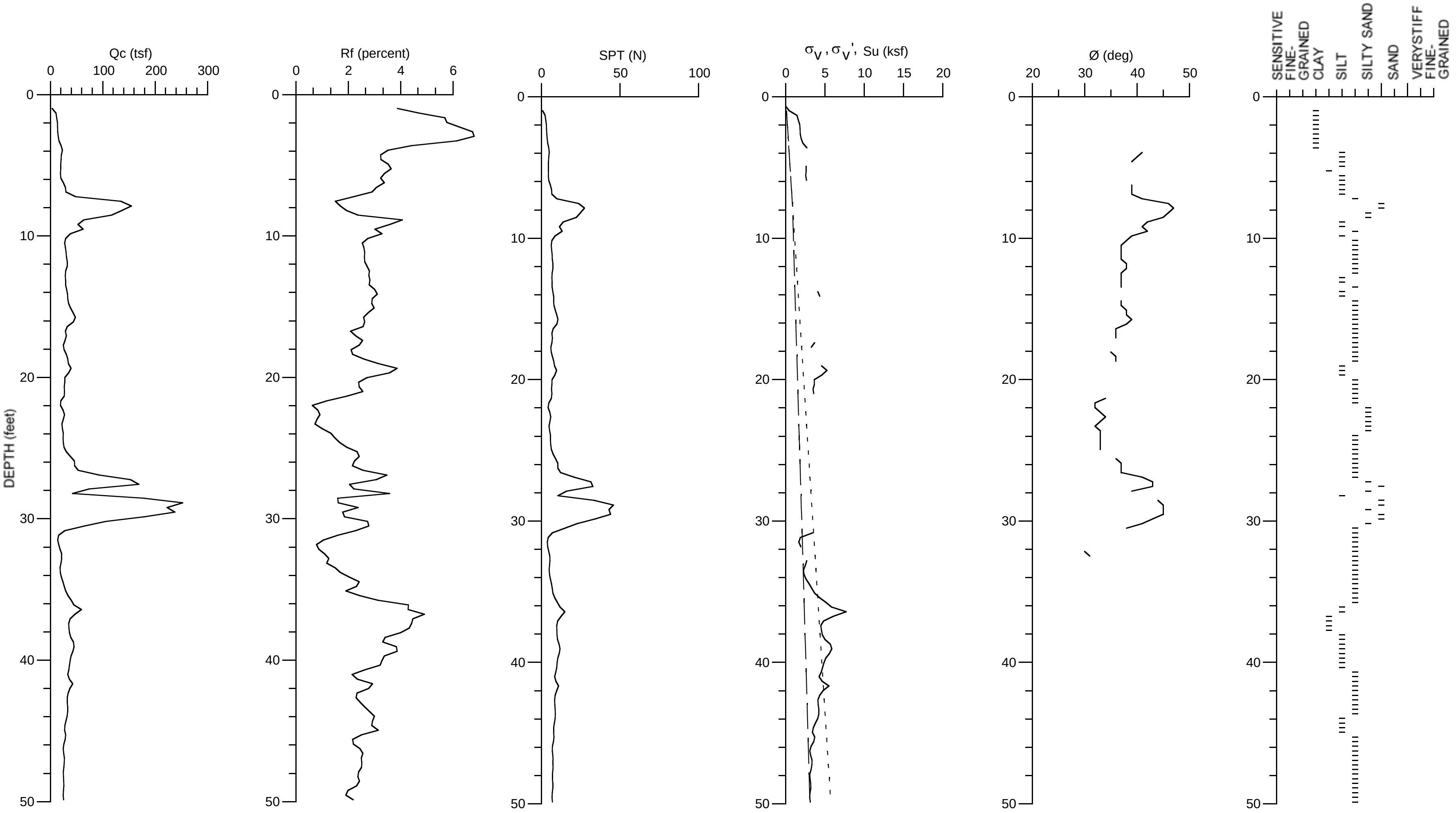
Terminated at 50 feet.
Groundwater was measured at a depth of 12 feet immediately after test.
Date performed: 03/13/08
Elevation and datum: 19 feet, City of Berkeley Datum.

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 05/22/08	Project No. 4795.01	Figure A-6

Treadwell&Rollo



Terminated at 50 feet.
Groundwater was measured at a depth of 7.7 feet immediately after test.
Date performed: 03/13/08
Elevation and datum: 20.5 feet, City of Berkeley Datum.

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 03/28/08	Project No. 4795.01	Figure A-7

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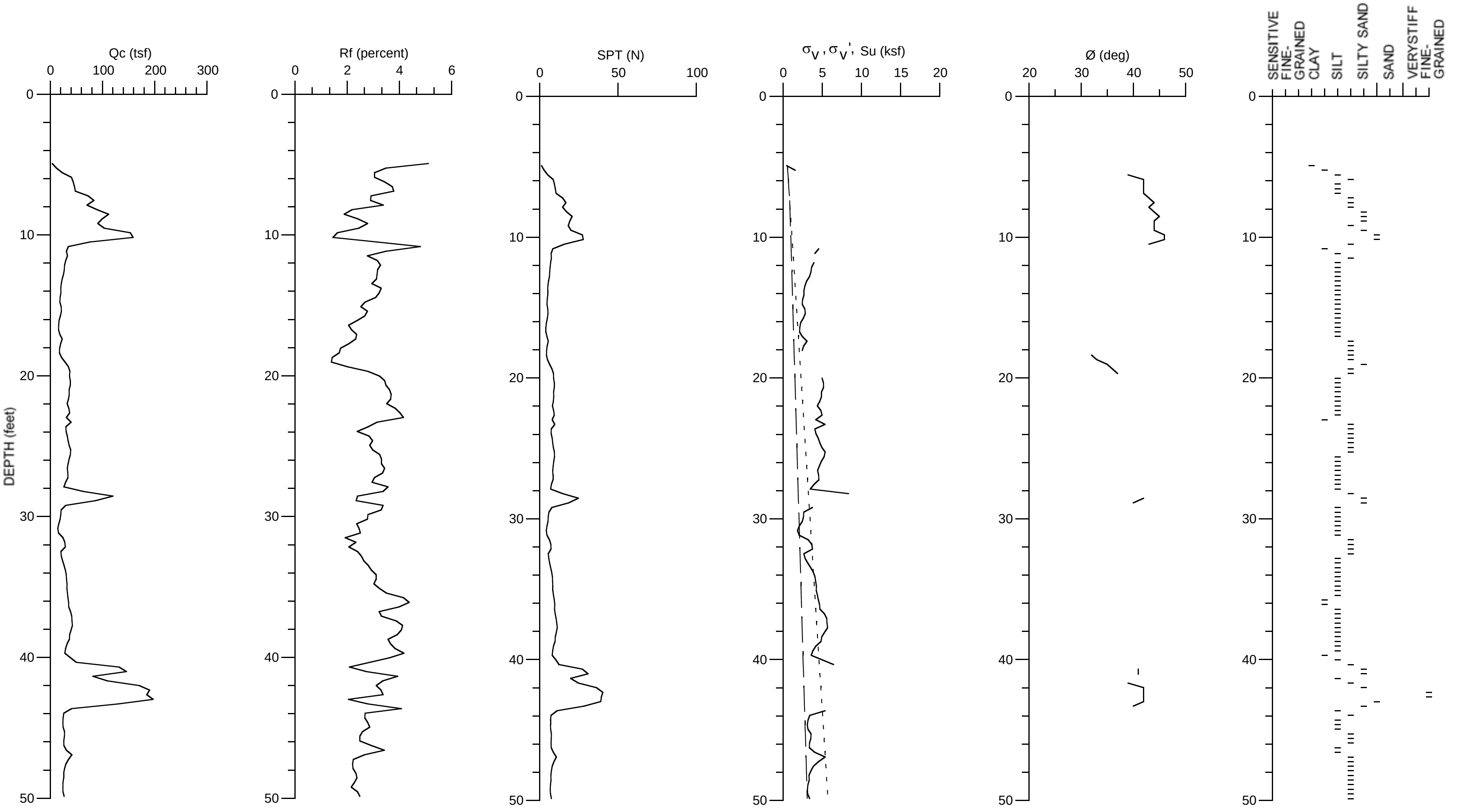


Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, s_u

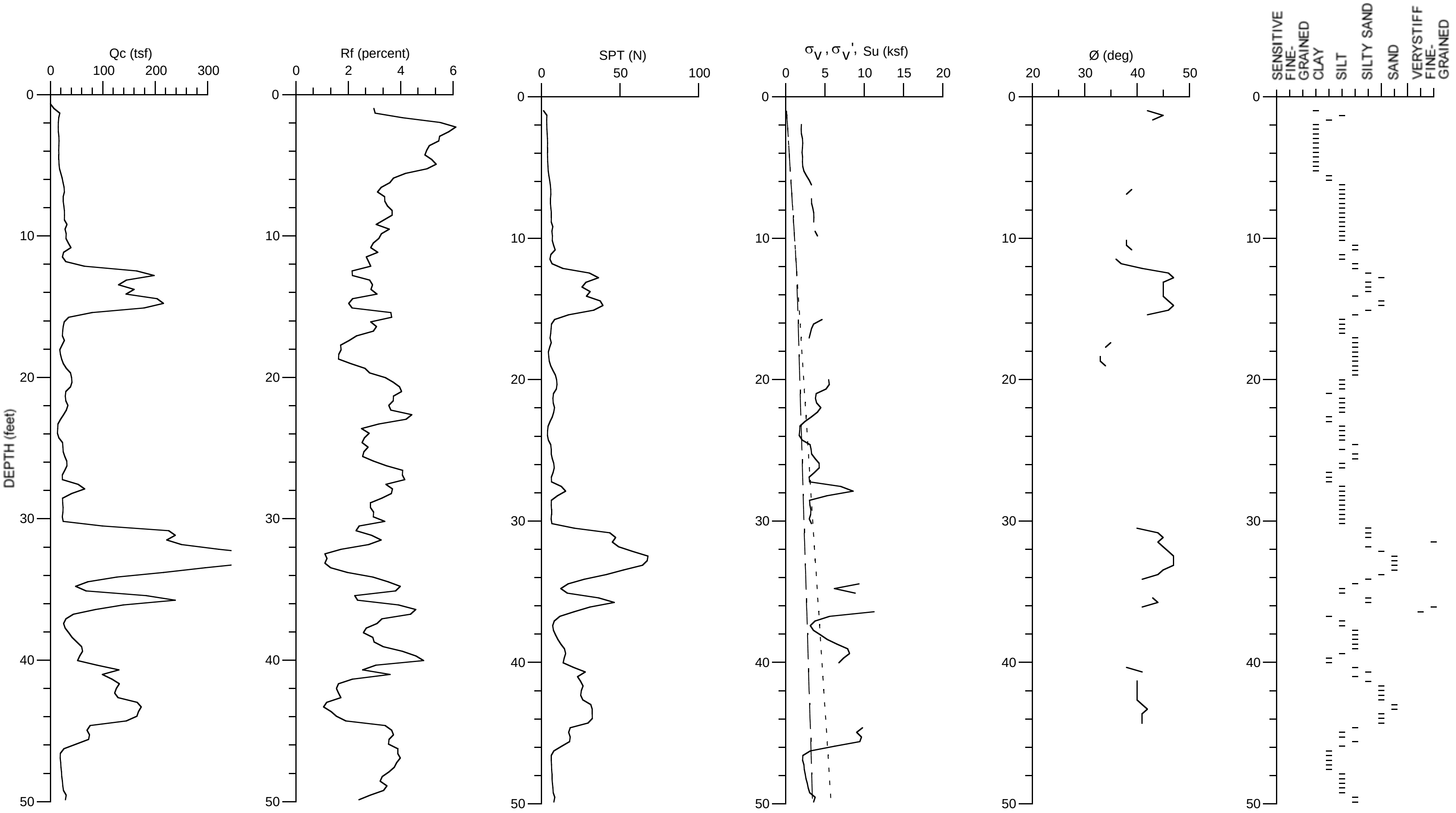
Terminated at 50 feet.
Groundwater was measured at a depth of 8.4 feet immediately after test.
Date performed: 03/13/08
Elevation and datum: 20.5 feet, City of Berkeley Datum.

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 03/28/08	Project No. 4795.01	Figure A-8

Treadwell&Rollo



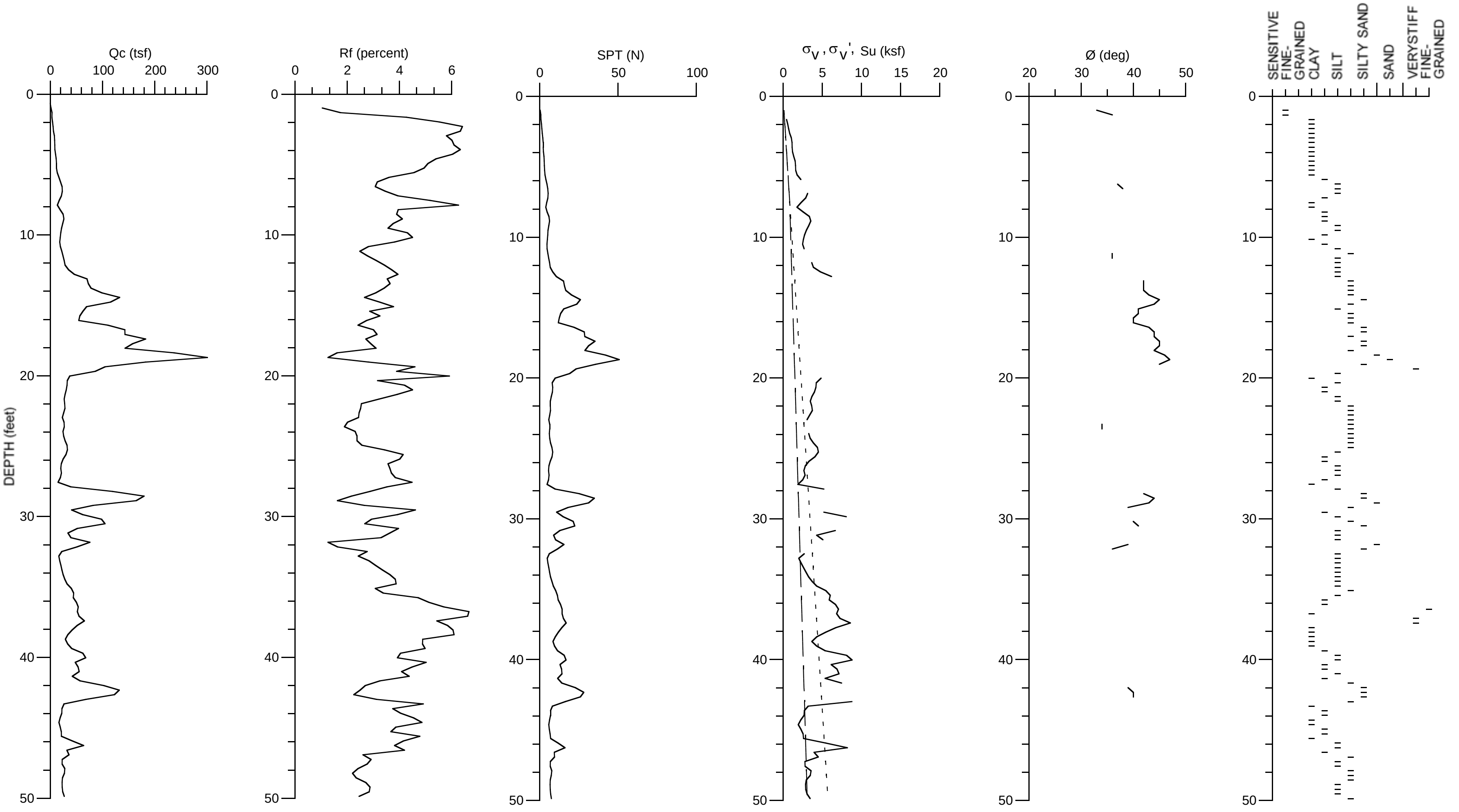
Terminated at 50 feet.
Groundwater was measured at a depth of 12.3 feet immediately after test.
Date performed: 03/13/08
Elevation and datum: 19.6 feet, City of Berkeley Datum.

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

CONE PENETRATION TEST RESULTS
CPT-4

Date 05/22/08	Project No. 4795.01	Figure A-9

Treadwell&Rollo



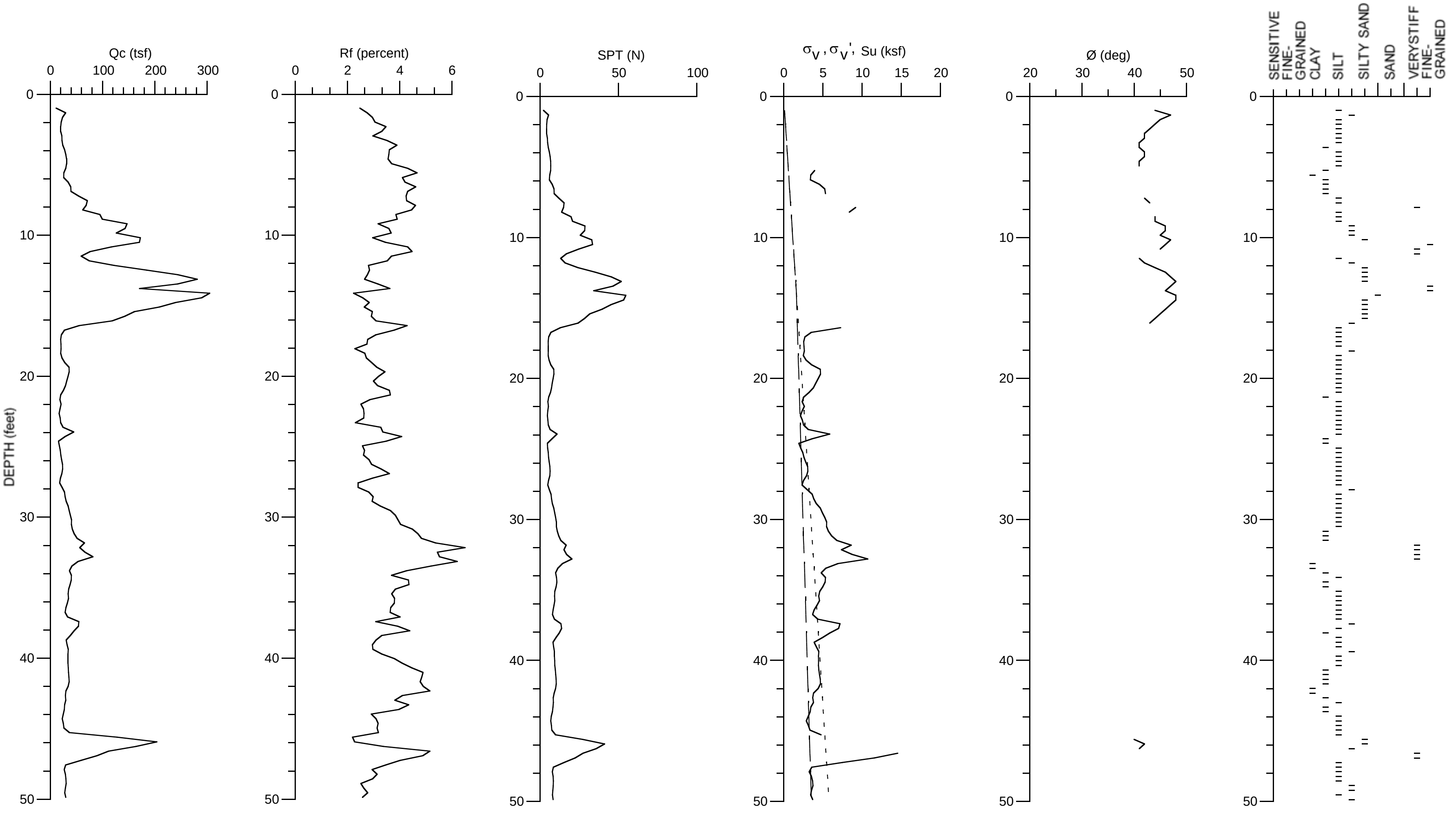
Terminated at 50 feet.
Groundwater was measured at a depth of 8.5 feet immediately after test.
Date performed: 03/13/08
Elevation and datum: 24.5 feet, City of Berkeley Datum.

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

CONE PENETRATION TEST RESULTS
CPT-5

Date 03/28/08	Project No. 4795.01	Figure A-10

Treadwell&Rollo



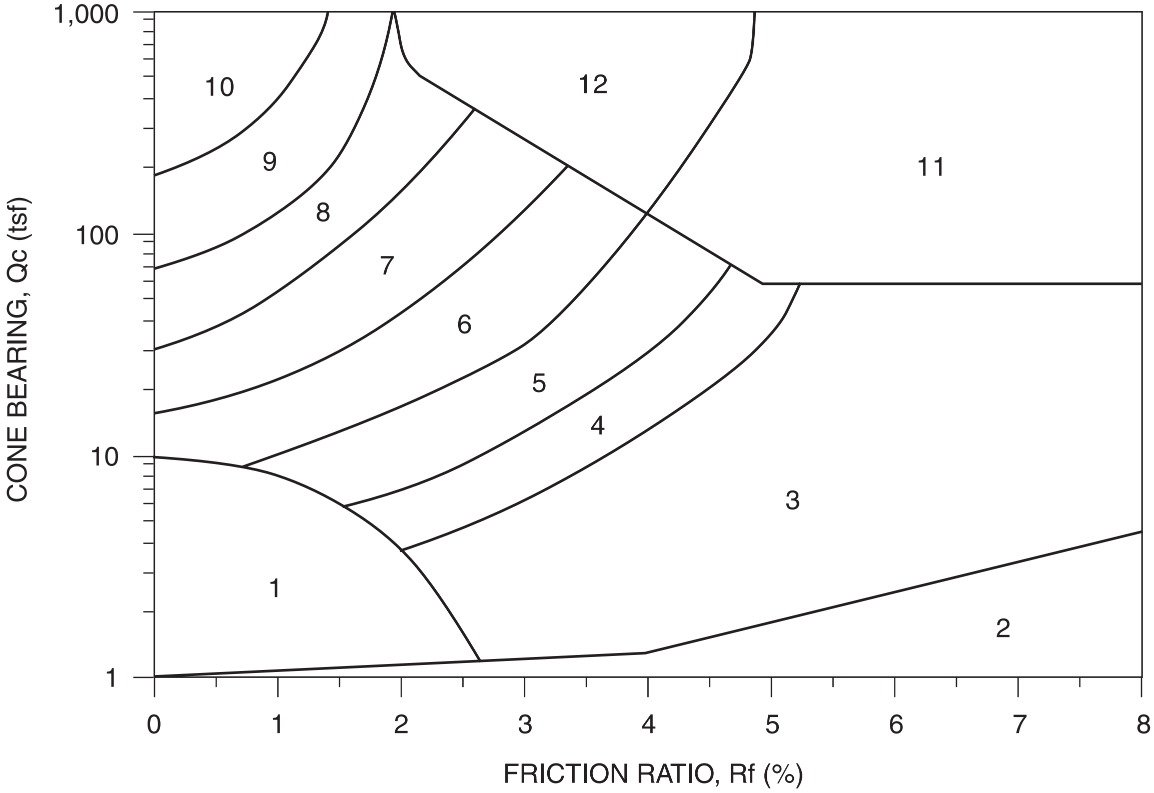
PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

CONE PENETRATION TEST RESULTS
CPT-6

Date 05/22/08	Project No. 4795.01	Figure A-11

Treadwell&Rollo

Terminated at 50 feet.
Groundwater was measured at a depth of 14 feet immediately after test.
Date performed: 03/13/08
Elevation and datum: 14 feet, City of Berkeley Datum.



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

(*) Overconsolidated or Cemented
Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

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

PEERLESS GREENS
MIXED-USE DEVELOPMENT
Berkeley, California

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CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 03/27/08 | Project No. 4795.01 | Figure A-12

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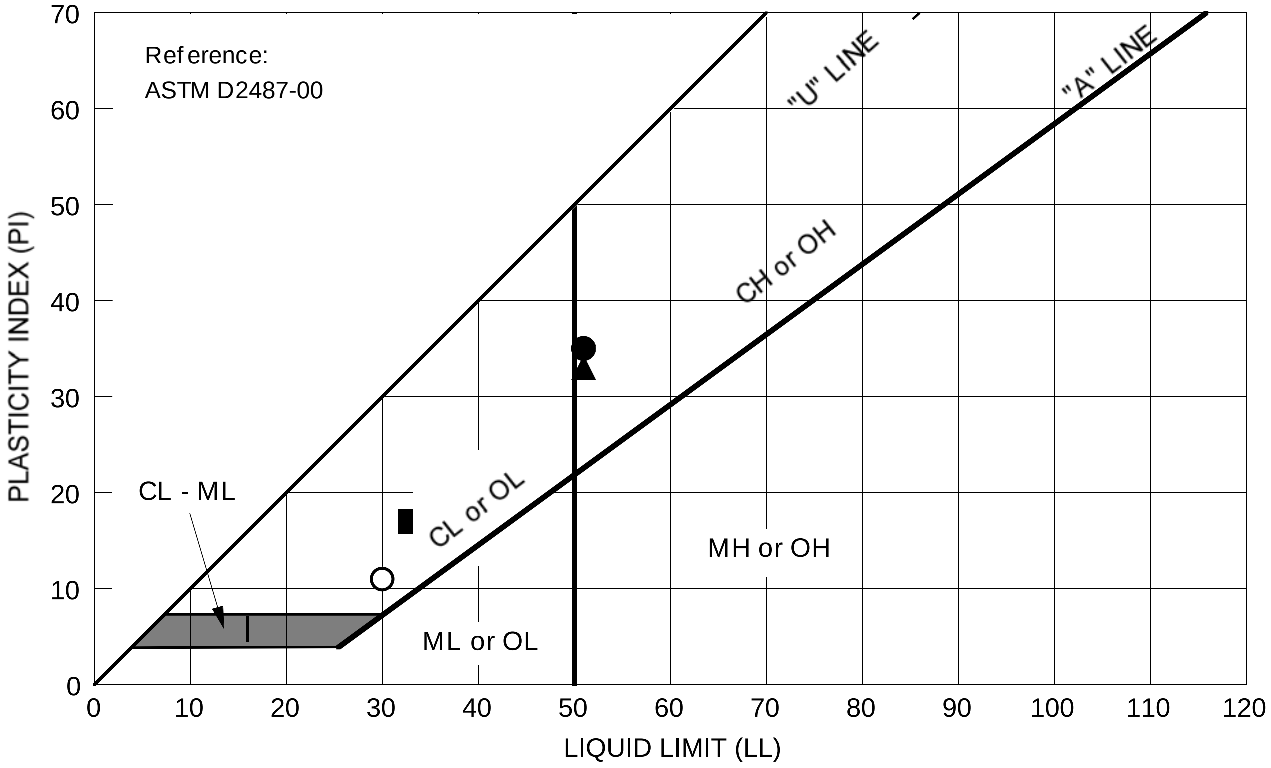
APPENDIX B
Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 2.5 feet	CLAY (CH), olive-brown	22.0	51	35	--
▲	B-3 at 3.0 feet	CLAY (CH), black	24.0	51	33	--
■	B-4 at 3.0 feet	CLAY (CL), brown	20.6	33	17	--
○	B-4 at 24.5 feet	CLAYEY SAND with GRAVEL (SC), yellow-brown	17.4	30	11	33

PEERLESS GREENS
MIXED-USE DEVELOPMENT
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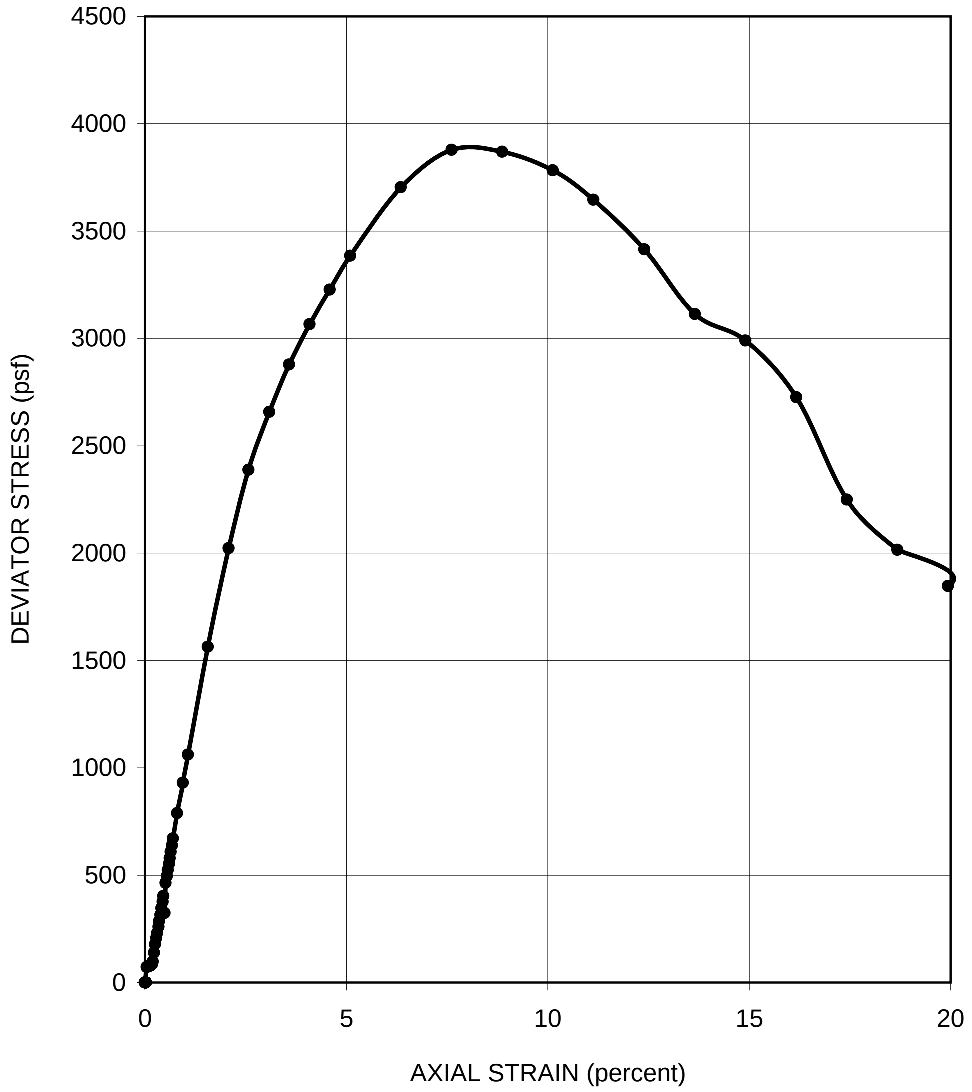
PLASTICITY CHART

Date 04/25/08 | Project No. 4795.01 | Figure B-1



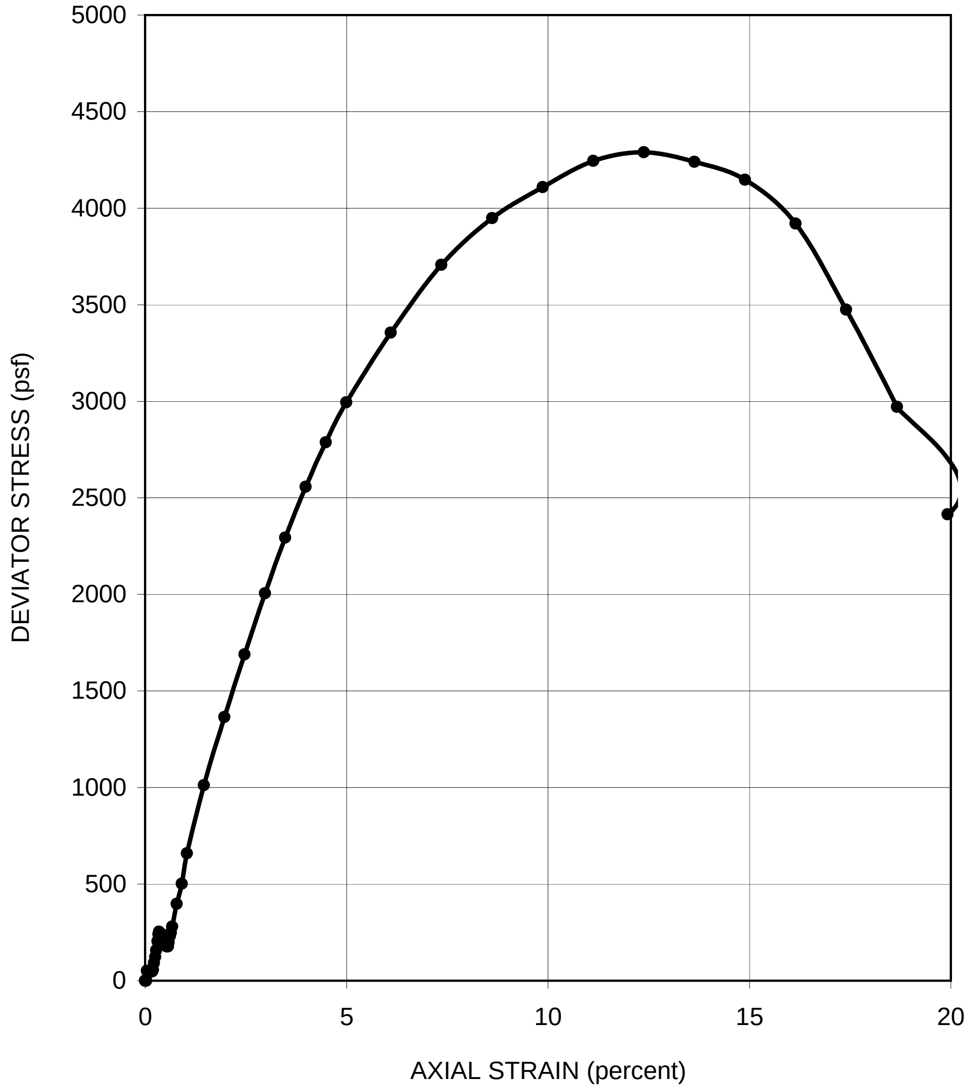
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	1,900	psf
DIAMETER (in.)	2.390	HEIGHT (in.) 6		STRAIN AT FAILURE	7.6	%
MOISTURE CONTENT		27.1	%	CONFINING PRESSURE	800	psf
DRY DENSITY		97	pcf	STRAIN RATE	1.00	% / min
DESCRIPTION	CLAY (CL), yellow-brown				SOURCE	B-2 @ 12 feet

PEERLESS GREENS MIXED-USE DEVELOPMENT
Berkeley, California

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Date	05/23/08	Project No.	4795.01	Figure	B-2



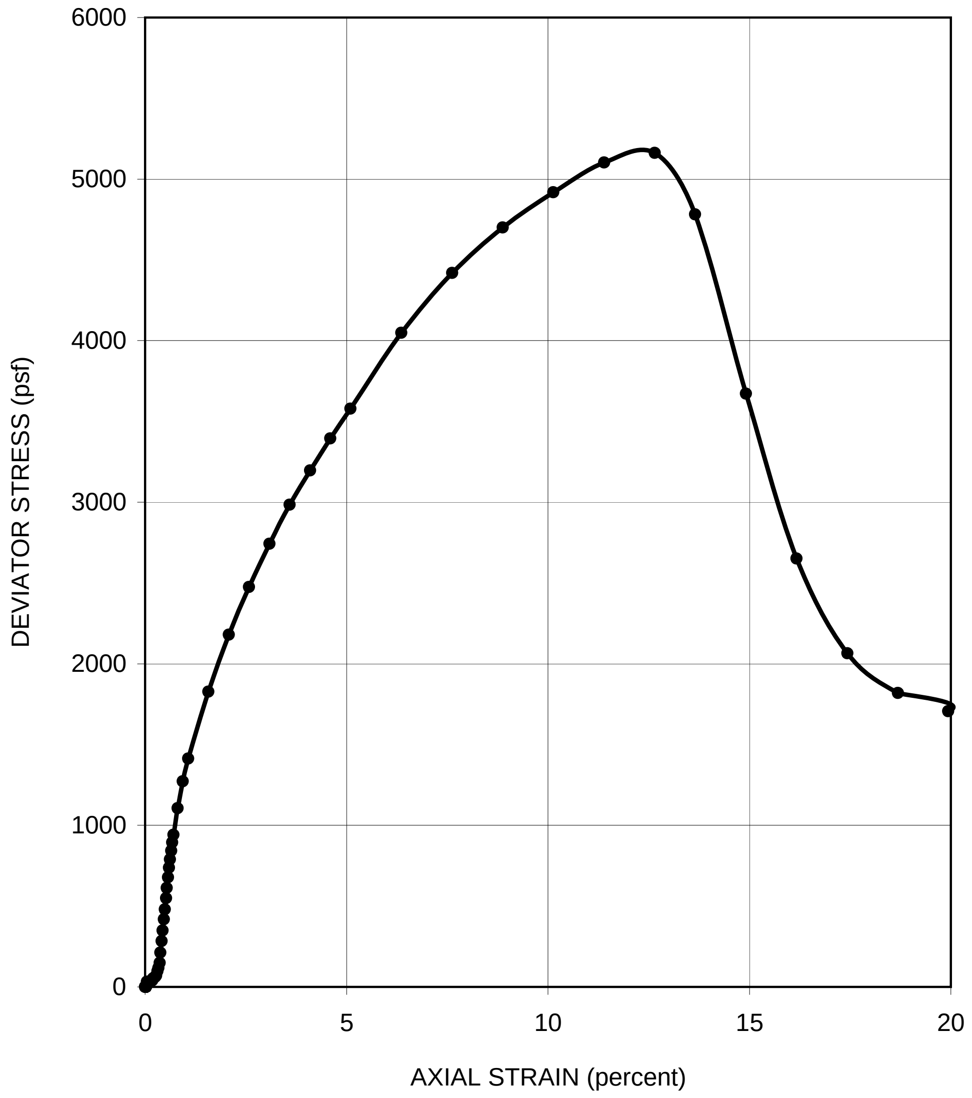
SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	2,150	psf
DIAMETER (in.)	2.390	HEIGHT (in.) 6	STRAIN AT FAILURE	12.4	%
MOISTURE CONTENT	20.9	%	CONFINING PRESSURE	1,100	psf
DRY DENSITY	107	pcf	STRAIN RATE	1.00	% / min
DESCRIPTION	CLAY with GRAVEL (CL), orange-brown			SOURCE	B-3 @ 19 feet

PEERLESS GREENS MIXED-USE DEVELOPMENT
Berkeley, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 05/23/08 | Project No. 4795.01 | Figure B-3



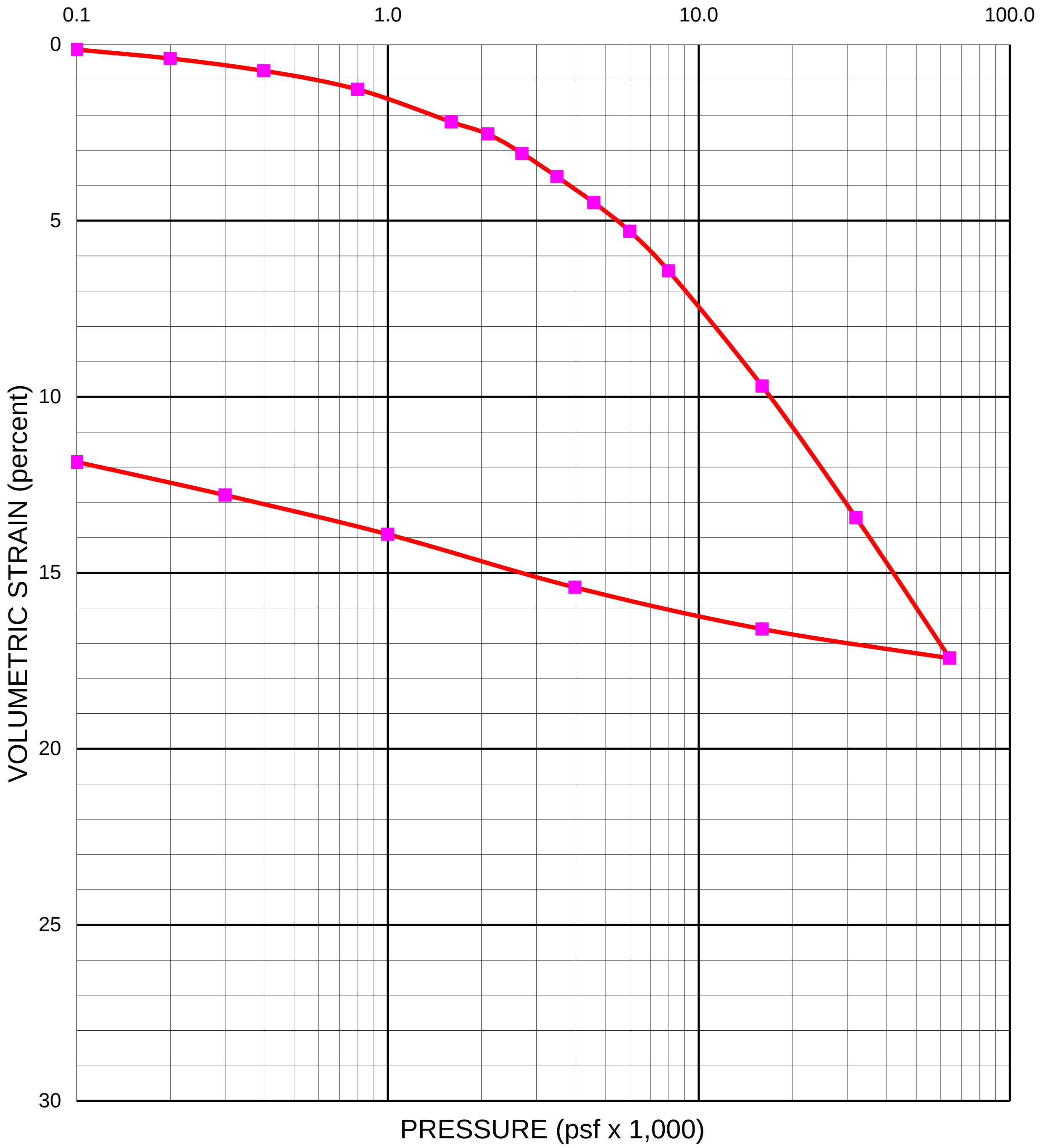
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	2,600	psf
DIAMETER (in.)	2.413	HEIGHT (in.) 5.88		STRAIN AT FAILURE	12.6	%
MOISTURE CONTENT		21.5	%	CONFINING PRESSURE	800	psf
DRY DENSITY		106	pcf	STRAIN RATE	1.00	% / min
DESCRIPTION	SANDY CLAY (CL), brown with black mottles				SOURCE	B-4 @ 12 feet

PEERLESS GREENS MIXED-USE DEVELOPMENT
Berkeley, California

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Date 05/23/08 Project No. 4795.01 Figure B-4



Sampler Type: Sprague & Henwood			Condition		Before Test		After Test
Diameter (in) 2.42	Height (in)	1.00	Water Content	w_o	26.0 %	w_f	19.6 %
Overburden Pressure, p_o	2,100	psf	Void Ratio	e_o	0.73	e_f	0.53
Preconsol. Pressure, p_c	4,500	psf	Saturation	S_o	96 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.14		Dry Density	γ_d	97 pcf	γ_d	110 pcf

Recompression Ratio, $C_{\varepsilon r}$ 0.02	LL --	PL --	PI --	G_s 2.70 (assumed)

Classification CLAY (CL), brown

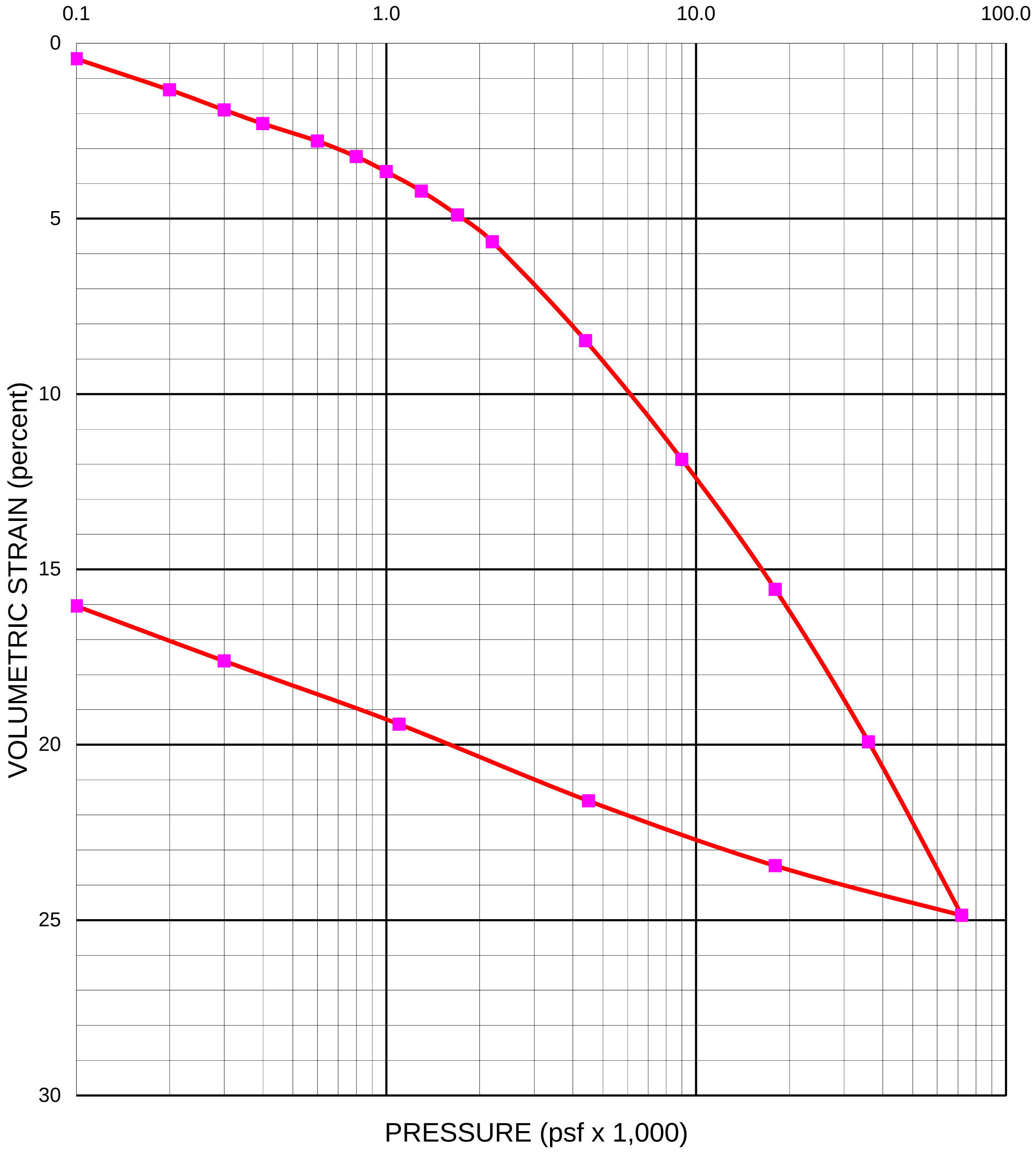
Source B-1 @ 24 feet

PEERLESS GREENS MIXED-USE DEVELOPMENT
Berkeley, California

CONSOLIDATION TEST REPORT

Treadwell&Rollo

Date 05/23/08 Project No. 4795.01 Figure B-5



Sampler Type: Sprague & Henwood				Condition		Before Test		After Test
Diameter (in)	2.34	Height (in)	1.00	Water Content	w_o	31.8 %	w_f	21.1 %
Overburden Pressure, p_o	3,800	psf		Void Ratio	e_o	0.84	e_f	0.55
Preconsol. Pressure, p_c	1,400	psf		Saturation	S_o	100 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.17			Dry Density	γ_d	92 pcf	γ_d	109 pcf
Recompression Ratio, $C_{\varepsilon r}$	0.03	LL	--	PL --		PI --	G_s	2.70 (assumed)
Classification CLAY (CL), brown				Source		B-2 @ 14 feet		

PEERLESS GREENS MIXED-USE DEVELOPMENT
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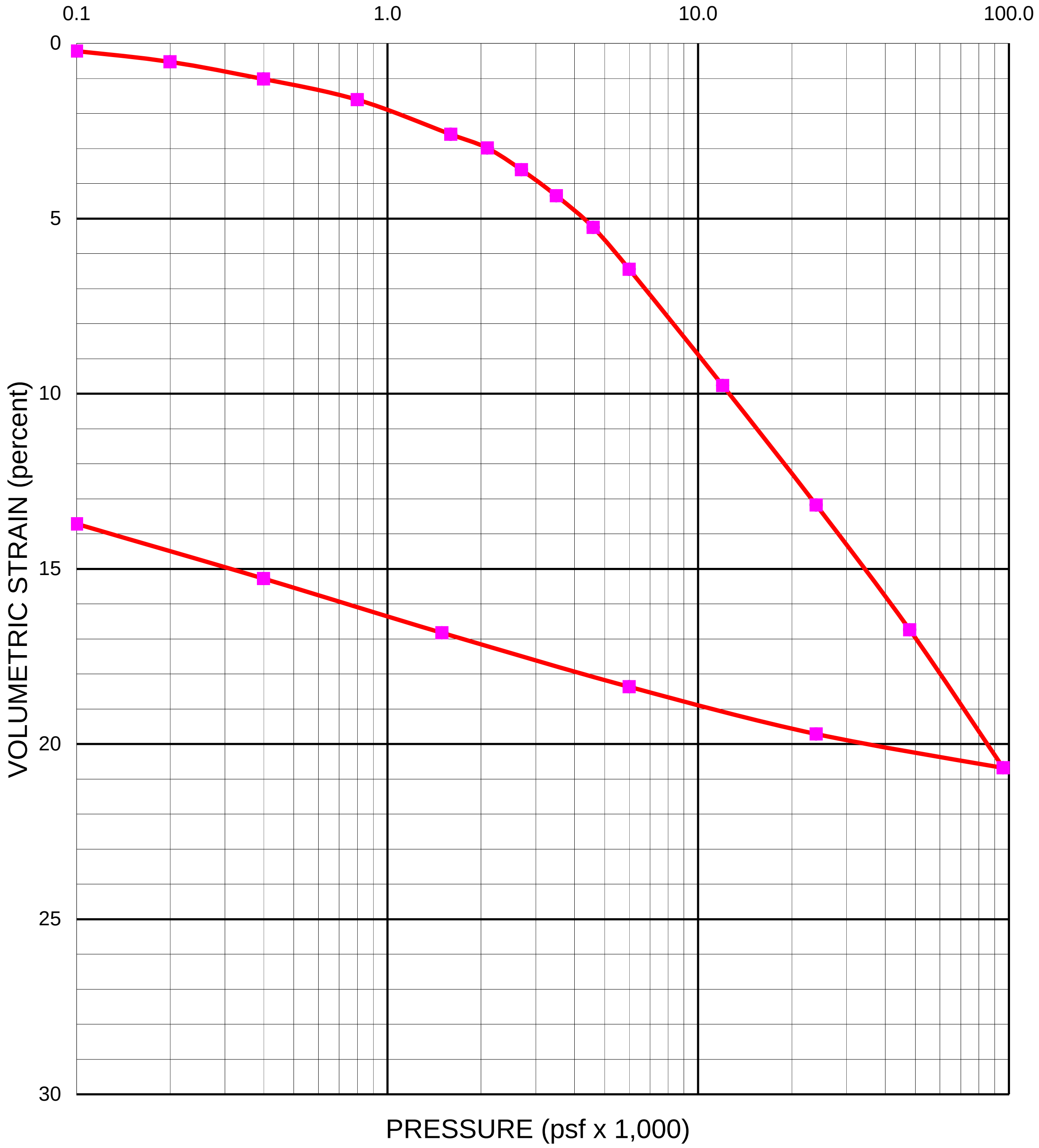
CONSOLIDATION TEST REPORT

Date 05/23/08 Project No. 4795.01 Figure B-6



Sampler Type:	Sprague & Henwood			Condition		Before Test			After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	25.8	%	w_f	18.1	%
Overburden Pressure, p_o	4,000	psf		Void Ratio	e_o	0.72		e_f	0.49	
Preconsol. Pressure, p_c	2,400	psf		Saturation	S_o	96	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.14			Dry Density	γ_d	98	pcf	γ_d	113	pcf
Recompression Ratio, $C_{\varepsilon r}$	0.02	LL	--	PL	--	PI	--	G_s	2.70	(assumed)

Classification CLAY (CL), brown

Source B-2 @ 29 feet

PEERLESS GREENS MIXED-USE DEVELOPMENT
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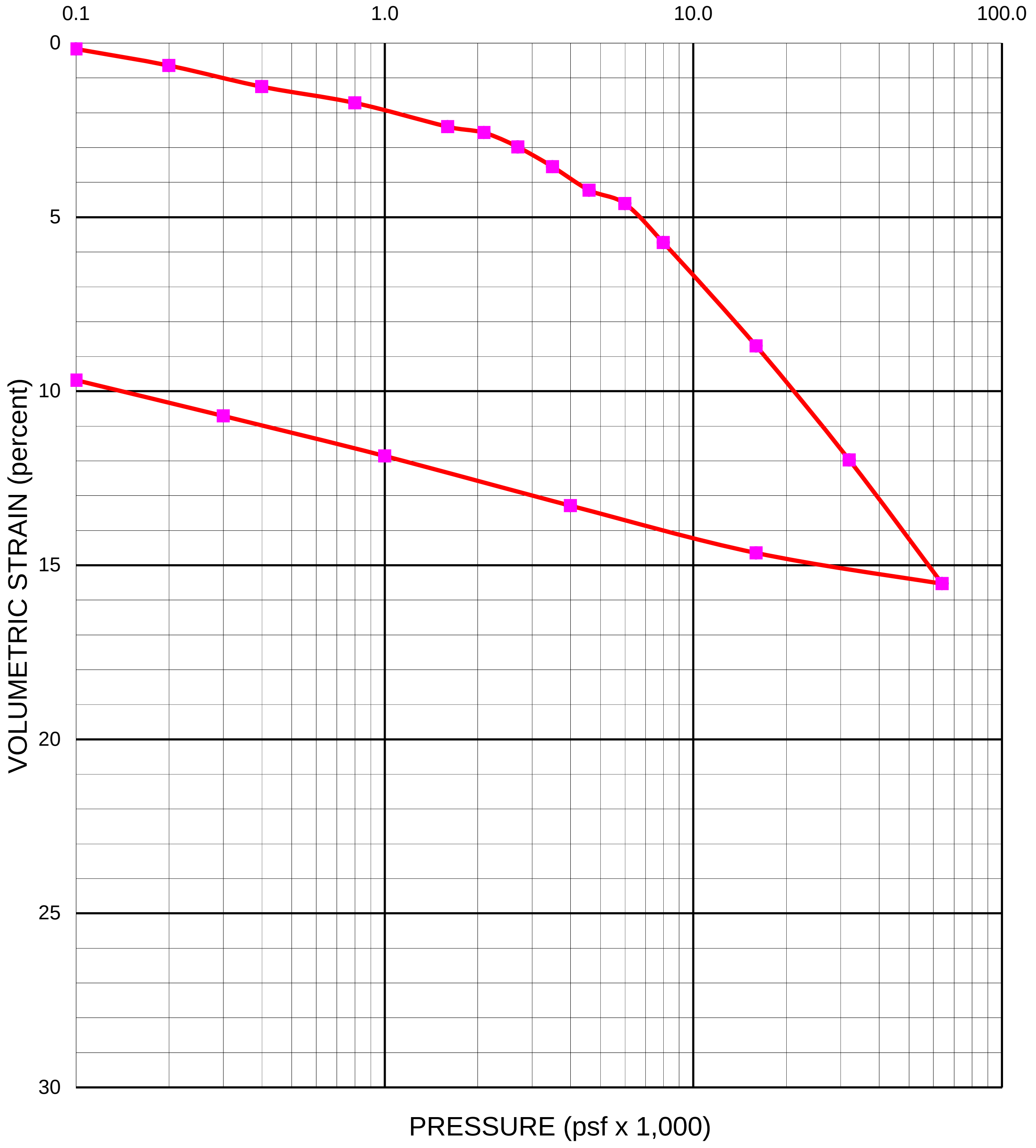
Date 05/23/08 Project No. 4795.01 Figure B-7



Sampler Type: Sprague & Henwood			Condition		Before Test		After Test
Diameter (in)	2.42	Height (in) 1.00	Water Content	w_o	22.4 %	w_f	17.9 %
Overburden Pressure, p_o	5,400	psf	Void Ratio	e_o	0.64	e_f	0.48
Preconsol. Pressure, p_c	1,800	psf	Saturation	S_o	94 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.12		Dry Density	γ_d	103 pcf	γ_d	114 pcf
Recompression Ratio, $C_{\varepsilon r}$	0.02	LL --	PL --	PI --		G_s 2.70	(assumed)
Classification CLAY (CL), brown			Source		B-4 @ 19.5 feet		

PEERLESS GREENS MIXED-USE DEVELOPMENT
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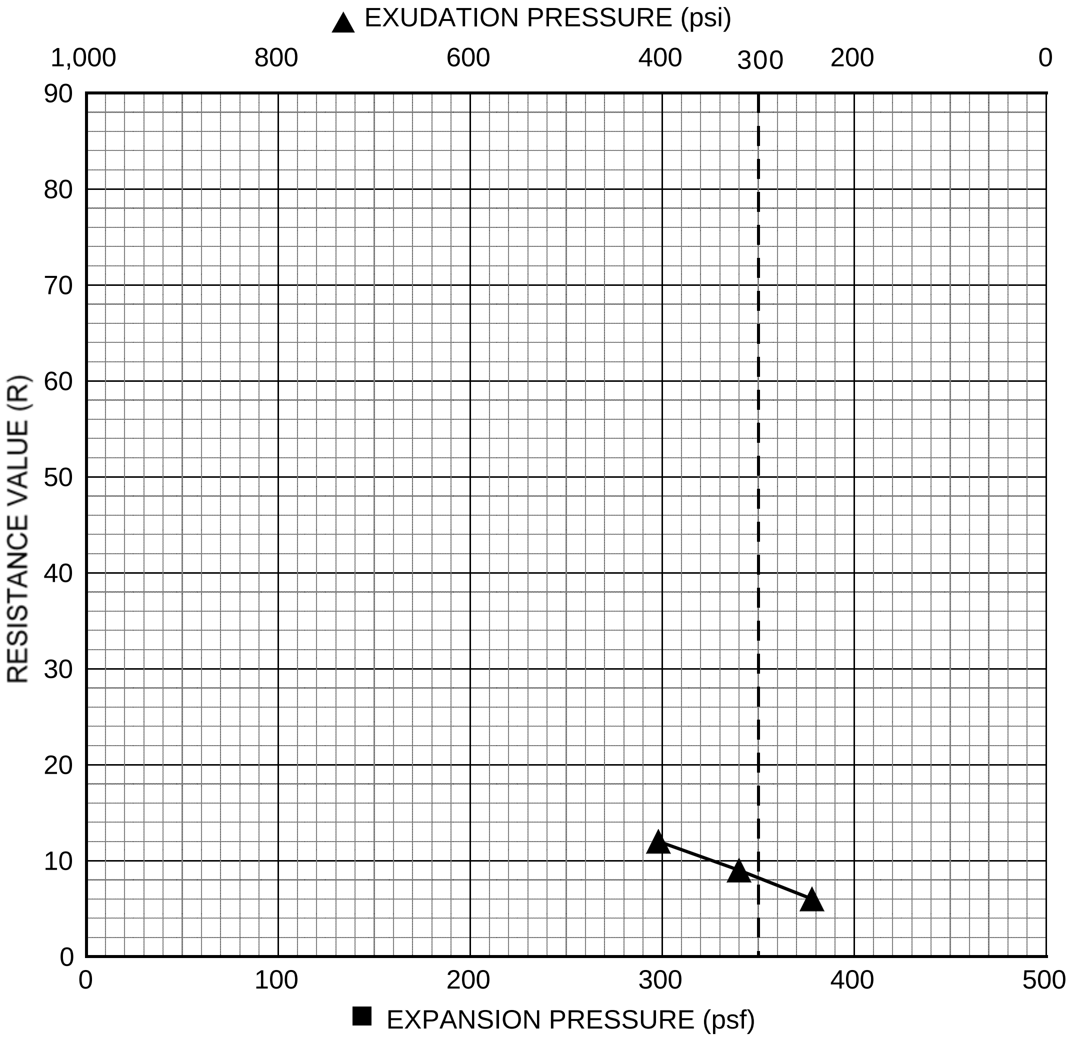
Date 05/23/08 Project No. 4795.01 Figure B-8



Specimen ID:	A	B	C	D
Water Content (%)	17.1	18.8	17.9	
Dry Density (pcf)	111	106.8	109.4	
Exudation Pressure (psi)	404	244	320	
Expansion Pressure (psf)	--	--	--	
Resistance Value (R)	12	6	9	

Sample Source	Sample Description	Sand Equivalent	Exudation Pressure	R value
B-4 at 0.5 to 5 feet	CLAY (CL), brown	---	300	8

PEERLESS GREENS
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RESISTANCE VALUE TEST DATA

Date 04/25/08 | Project No. 4795.01 | Figure B-9

ETS

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JOB SITE: Peerless Greens Mixed Use Development, Berkeley, California
JOB #: 4795.01

DATE RECEIVED 3/19/2008
DATE of COMPLETION 3/31/2008
ANALYST(S) D. Salinas, S. Santos
SUPERVISOR D. Jacobson
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LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
03037-1	PG1/B	B-2-2 @ 6.0'	**7.79**	**1,520**	**[660]**	**84**	**67**
03037-2	PG2/B	B-3-3 @ 9.0'	**8.41**	**1,250**	**[800]**	**120**	**70**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
03037-1	PG1/B	B-2-2 @ 6.0'		**0.108**		**+322.2**	
03037-2	PG2/B	B-3-3 @ 9.0'		**0.063**		**+181.1**	
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

*** COMMENTS ***

Resistivities are at 1,250-1,500 ohm/cm which is low, but soil reactions (i.e., pHs) are mildly to moderately alkaline which does help some; both sulfate and chloride are low; sulfides are at or near borderline; one soil is only very mildly reduced, but the other is moderate. The standard CalTrans times to perforation for these two soils are as follows: for PG1/B and 18 ga steel the time is over 29 yrs, and for 12 ga the time goes up to 65 yrs; and for PG2/B the respective times are ≈27 yrs, and 60 yrs. For steel the calculated average pitting rates are PG1 @ ≈0.090 mm/yr, and PG2 @ ≈0.095 mm/yr: thus, for PG1 pitting to 2 mm depth is at ≈22 yrs, and to a 4 mm depth is ≈42 yrs; and for PG2 times are at ≈21 yrs, and ≈42 yrs. Chlorides are low enough so as not to have any significant corrosion impact on concrete steel reinforcement; likewise, sulfates are low enough so as not to have any significant adverse impact on concrete, cements, mortars or grouts. Soluble sulfides are at or fairly near 0.1 ppm, and thus are a concern. Also, while one redox is very mild and not a concern, the other is low enough to be of some concern. These soils would not benefit at all from alkaline treatment since their pHs are already alkaline enough. To assure specific longevity in the soils could require treatment to reduce sulfides, especially in PG1, and increase oxidation state (of PG2). For both soils metals upgrading (e.g. increased gauge or resistant steels, etc.) would be prudent; and/or other actions could be taken (e.g. cathodic protection, increased engineering fill, wrapped or coated steel, plastic or fiberglas pipe, etc.). In the absence of treating soils for redox and sulfides it probably would be prudent to use more resistant concrete (e.g. at least ASTM Type II cement); with treatment standard concrete and cement mixes should be acceptable.

\\\\ NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

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