ROCKRIDGE GEOTECHNICAL

Prepared for **Westlake Development Partners, LLC**

FINAL REPORT
PRELIMINARY GEOTECHNICAL INVESTIGATION
SAN LEANDRO CROSSING DEVELOPMENT
SAN LEANDRO, CALIFORNIA

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November 7, 2008
Project No. 08-068

4319 Piedmont Ave., Suite 204
Oakland, CA 94611

510 420-5738 tel
510 652-3096 fax

ROCKRIDGE GEOTECHNICAL

November 7, 2008
Project No. 08-068

Mr. Sunny Tang
Westlake Development Partners, LLC
520 South El Camino Real, 9th Floor
San Mateo, California 94402-1722

Subject: Final Report
Preliminary Geotechnical Investigation
San Leandro Crossing – Blocks 1 and 2
San Leandro, California

Dear Mr. Tang,

We are pleased to present our preliminary geotechnical investigation report, dated November 7, 2008, for the proposed San Leandro Crossing project in San Leandro, California. Our services are being provided in accordance with our proposal dated April 28, 2008 and our Consultant Services Agreement dated April 29, 2008.

The conclusions and recommendations presented in our report are based on a preliminary field investigation and are not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on a supplemental field investigation.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
CRAIG S SHIELDS
GE2116
3/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

Craig S. Shields, P.E., G.E.
Principal Engineer

Enclosure

4319 Piedmont Ave., Suite 204
Oakland, CA 94611

510 420-5738 tel
510 652-3096 fax

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PRELIMINARY GEOTECHNICAL INVESTIGATION
SAN LEANDRO CROSSING
BLOCKS 1 AND 2
San Leandro, California

1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the proposed San Leandro Crossing Project located in San Leandro, California. The proposed development is located on two blocks near the San Leandro Bay Area Rapid Transit (BART) Station, as shown on the Site Location Map, Figure 1.

The project will be constructed on two "blocks" referred to as Blocks 1 and 2. Block 1 consists of the city block bordered by West Juana Avenue, San Leandro Boulevard, and Carpentier Street. It measures approximately 300 by 315 feet in plan dimensions and is currently an at-grade BART parking lot. Plans are to construct 200 market-rate residential units with 5,000 square feet of retail space and 290 parking spaces. The ground floor level will consist of retail space and residential units with a central parking structure. Four levels of residential units will be constructed above the ground floor level. Additionally, one level of below-grade parking will be constructed at the site; therefore, we anticipate an excavation on the order of 10 to 15 feet will be required. The basement and ground floor level will be of reinforced concrete construction and the upper four stories of residential units will be framed in wood or light gauge steel.

Block 2 is a vacant lot to the southwest of the San Leandro BART Station that measures about 150 by 600 feet in plan. Plans are to construct a three-story BART parking garage with 329 stalls and 100 affordable residential units with 102 parking stalls. The BART parking structure will be constructed in the southeastern third of Block 2 and will be constructed at grade. We understand the garage may be designed so that 2 or 3 stories can be added in the future. The affordable residential units will be constructed in the northwestern two-thirds of Block 2. This structure will be constructed at grade and consist of four stories of residential over one story of parking. Preliminary plans indicate that a portion of Martinez Street (between Parrott and Thornton Streets) will be removed and incorporated into the BART parking garage.

2.0 SCOPE OF SERVICES

The scope of our services was outlined in our proposal dated April 28, 2008. These services consisted of performing a preliminary field investigation consisting of three geotechnical borings and four cone penetration tests (CPTs). We used the data acquired during our field investigation to performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- the most appropriate foundation type(s) to support the proposed buildings
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement
- lateral earth pressures for design of basement walls
- subgrade preparation for slab-on-grade floors and concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- site seismicity and seismic hazards, including liquefaction potential and cyclic densification
- pavement sections
- 2007 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

The California State Geological Survey (CGS) has prepared a map titled *State of California Seismic Hazard Zones, San Leandro Quadrangle,* dated 14 February 2003. This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990 and is presented as Figure 3. The site is within a designated liquefaction hazard zone. The CGS has also recommended the content for site investigation reports within seismic hazard zones in the State of California Special Publication (SP) 117, titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated 13 March 1997. The liquefaction evaluation performed for this study was prepared in general accordance with the recommendations presented in SP 117.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Preliminary subsurface data was developed by drilling three borings and advancing four CPTs at the approximate locations shown on Figure 2. Prior to mobilizing to the site, we contacted Underground Service Alert (USA) to notify them of our work as required by law and also retained a private utility locator to check that boring and CPT locations were clear of existing utilities. Details of the field exploration are described in the remainder of this section.

3.1 Test Borings

The test borings, designated as Borings B-1 through B-3, were drilled on August 23, 2008 by Exploration Geoservices of San Jose, California, using a truck-mounted drill rig equipped with eight-inch-outside-diameter hollow-stem augers. Borings B-1, B-2 and B-3 were advanced to depths of 50, 55 and 50 feet below the existing ground surface (bgs), respectively.

During drilling, our field engineer logged the borings and retrieved representative samples of the soil encountered for further classification and laboratory testing. The boring logs are presented in Appendix A as Figures A-1 through A-3, respectively. The soil was classified in accordance with the classification system presented on Figure A-4.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.43-inch inside diameter, lined with brass tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter (no liners)
- 3.0-inch-outside-diameter, thin-walled Shelby tubes (ST).

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. The S&H and SPT samplers were driven with a 140-pound downhole wireline hammer falling about 30 inches per drop. The blow counts required to drive the S&H sampler the final 12 inches of an 18-inch drive were converted to SPT N-values using a conversion factor of 0.6. The converted SPT N-values are shown on the boring logs. Where the SPT sampler was used, the actual blow counts are shown on the boring logs. Hydraulic pressure was used to advance the 30-inch-long Shelby tubes into the soil; this pressure is shown on the boring logs as measured in pounds per square inch (psi).

After completion, the borings were backfilled with neat cement grout. The cuttings generated during drilling for Block 1 were placed in a 55-gallon drum and a representative sample of the cuttings was collected for analytical testing. The drum was subsequently disposed at a landfill by Advanced Environmental Services. The cuttings for Block 2 were left on the ground surface near the boreholes.

3.2 Cone Penetration Tests

John Sarmiento & Associates of Orinda, California advanced four CPTs, designated as CPT-1 through CPT-4, on 20 and 30 August 2008. The CPTs were performed by hydraulically pushing a 1.4-inch-diameter cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone-tipped probe measured tip resistance, and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated

data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered. The CPTs were advanced to depths ranging from 50 to 100 feet bgs. Upon completion, the CPTs were backfilled with cement grout. The CPT logs, showing tip resistance and friction ratio by depth, as well as interpreted SPT N-values, soil shear strength parameters, and soil classifications, are presented in Appendix A.

3.3 Laboratory Testing

We examined each soil sample in the office to confirm the field classification and select representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, fines content (percent passing the No. 200 sieve), Atterberg limits[1] (plasticity index), shear strength, and compressibility. The laboratory test results are presented on the boring logs and in Appendix B.

To evaluate the corrosion potential of the near-surface soil, two representative soil samples were tested. The results of the corrosivity analyses are presented in Appendix B and are discussed later in Section 6.8 of this report.

4.0 SITE AND SUBSURFACE CONDITIONS

The project consists of two parcels, Block 1 and Block 2, within the vicinity of the San Leandro BART station. Block 1 is located to the northeast of the BART station at the northern corner of the intersection of San Leandro Boulevard and West Juana Avenue. Block 1 is rectangular in shape and currently consists of an at-grade asphalt parking lot. Site elevations at Block 1 range between Elevation 44 and 47 feet[2]. Block 2 is located to the south of the BART station and to the southwest of the intersection of Martinez and Parrott Streets. Block 2 currently consists of two undeveloped, triangular-shaped parcels on either side of Martinez Street and the portion of

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1 Atterberg limits are an indirect measure of the expansion potential of the soil.

2 Elevations presented in this report are interpolated from the elevations shown on the ALTA/ACSM Land Title Survey by Lea & Braze Engineering, Inc. dated 7-30-07.

Martinez Street between Parrott and Thornton Streets. Site elevations at Block 2 range between approximately Elevation 45 and 48 feet.

The results of our field investigation indicate the site is underlain by alluvium consisting of interbedded clay, sand, and gravel. Generalized subsurface information for each block by order of depth encountered is described in the following paragraphs.

4.1 Block 1

Boring B-3 and CPT-3 and CPT-4 indicate Block 1 is underlain by underlain by clay that extends to depths ranging from about 25 to 50 feet below the existing ground surface (bgs). The upper approximately 20 to 23 feet of the clay deposit is generally stiff to very stiff. Between depths of about 20 and 40 feet bgs, there are some relatively thin zones of medium stiff clay. An Atterberg limits test performed on a sample of clay from a depth of 2 feet bgs indicates the clay is moderately to highly expansive at that depth.

Below the clay at a depth of 40 feet bgs at the CPT-3 location, we encountered dense to very dense sand that extends to the maximum depth explored of 50 feet bgs. Below a depth of 25 feet bgs in Boring B-3, we encountered medium dense to very dense sand and gravel layers with variable clay and silt content interbedded with medium stiff clay.

Groundwater was measured in Boring B-1 at a depth of 23.5 feet bgs immediately after drilling was completed. Groundwater was measured at depths of 24.9 and 13.6 feet bgs in CPT-3 and CPT-4, respectively, shortly after completion of the CPTs. Because the measurements were taken shortly after the borings and CPTs were completed, there may have been insufficient time for the groundwater to stabilize. However, based on our examination of the soil samples, we believe the stabilized groundwater level is on the order of 23 to 24 feet bgs at the time of drilling and the measurement of the higher groundwater level in CPT-4 is either perched groundwater or an erroneous measurement. The depth to groundwater is expected to vary several feet seasonally, depending on the amount of rainfall.

4.2 Block 2

The borings and CPTs performed on Block 2 indicate this area is blanketed by approximately 22 to 28 feet of clay with varying sand content. With the exception of the boring B-2 location, the clay is generally stiff to very stiff with some thin (less than two feet thick) medium stiff to stiff zones below a depth of 20 feet. At the boring B-2 location, the upper approximately 10 to 12 feet of clay is medium stiff. Below a depth of about 12 feet, the clay is stiff to very stiff. Atterberg limits tests indicate the clay is moderately expansive.

Beneath the upper clay layer, we encountered heterogeneous, interbedded alluvial soils that extend to the maximum depth explored of 100 feet bgs. The granular soils encountered above a depth of 50 feet include medium dense to dense clayey sand with varying gravel content and medium dense to dense gravel with varying clay and sand content. Above a depth of 50 feet, there are some medium stiff to stiff clay layers interbedded with the predominantly granular soils. Below a depth of 50 feet bgs, the soil predominantly consists of dense to very dense sand and gravel with occasional layers of very stiff clay.

Groundwater was measured at depths ranging from 22.5 to 26 feet bgs in the four borings and CPTs advanced on Block 2. Considering the borings and CPTs encountered predominantly granular soils below a depth of 28 feet, we anticipate the stabilized groundwater level is close to the measured depths. As mentioned above, we anticipate the depth to groundwater will vary several feet seasonally.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The major active faults in the area are the Hayward and San Andreas Faults. These and other faults of the region within 100 kilometers of the site are shown on Table 1. For each of the

active faults, the distance and direction from the site, the estimated maximum Moment magnitude[3] and the Mean Slip Rate are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Fault (km)	Direction from Site	Maximum Moment Magnitude	Mean Slip Rate (mm/yr)
Hayward - Total	2.9	Northeast	6.91	9
Southern Hayward	2.9	Northeast	6.67	9
Northern Hayward	12	North	6.49	9
Northern Calaveras	17	Northeast	6.78	6
Mount Diablo Thrust	20	Northeast	6.65	2
Concord	25	Northeast	6.25	4
San Andreas - 1906 Rupture	26	Southwest	7.90	24
San Andreas - Peninsula	26	Southwest	7.15	17
Monte Vista	33	South	6.80	0.4
Northern Greenville	34	Northeast	6.66	2
San Gregorio North	36	West	7.23	7
Southern Green Valley	36	North	6.24	5
Hayward - South East Extension	37	Southeast	6.40	3
San Andreas - North Coast South	37	West	7.45	24
Rodgers Creek	43	North	6.98	9
Southern Greenville	44	East	6.60	2
Central Calaveras	44	Southeast	6.23	15
Great Valley - 6	47	Northeast	6.70	1.5
West Napa	50	North	6.50	1
Great Valley - 5	54	Northeast	6.50	1.5
Great Valley - 7	56	East	6.70	1.5

[3] 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.

Northern Green Valley	60	North	6.02	5
San Andreas - Santa Cruz Mnts.	62	South	7.03	17
Point Reyes	64	Northwest	6.80	0.3
Sargent	68	South	6.80	3
Great Valley - 4	68	North	6.60	1.5
Zayante-Vergeles	72	South	6.80	0.1
Hunting Creek - Berryessa	81	North	6.90	6
Monterey Bay - Tularcitos	89	South	7.10	0.5
Ortigalita	93	Southeast	6.90	1
Great Valley - 8	94	East	6.60	1.5
San Gregorio South	97	South	6.96	3

In 2002, the WGCEP at the U.S. Geologic Survey (USGS) predicted a 62 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2032. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2

WGCEP (2002) Estimates of 30-Year Probability (2003 to 2032) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	27
San Andreas	21
Calaveras	11
San Gregorio	10
Concord-Green Valley	4
Greenville	3
Mount Diablo	3

5.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5], and cyclic densification[6]. We used the results of the borings and CPTs to evaluate the potential for these phenomena to occur at the project site. Our evaluation of site seismic hazards was performed in general accordance with the guidelines presented in SP 117.

5.2.1 Ground 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 fault offset at the site from a known active fault is very 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 from previously unknown faults is also very low.

5.2.2 Strong Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions.

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

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

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

The site is about 2.9 kilometers from the Hayward Fault as shown on Figure 4. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.3 Cyclic Densification

Seismically induced compaction or cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in differential settlement. Based on the subsurface data acquired to-date, we preliminarily conclude the soil above the groundwater table on both blocks contains sufficient clay that the risk of cyclic densification is very low.

5.2.4 Liquefaction and Ground Failure

Liquefaction is a phenomenon in which saturated soil temporarily loses strength from the build up of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. We evaluated liquefaction potential at the site in accordance with SP 117, as described below.

SP 117 states that liquefaction analyses should be performed using subsurface information from rotary-wash borings and/or CPTs. Our borings were drilled using hollow-stem augers; therefore, we primarily used the data from our CPTs for our liquefaction evaluation. Laboratory tests performed on samples collected at each boring location were used to check for fines content and soil classification.

Based on our engineering studies, we conclude intermittent potentially liquefiable clayey sand and silty sand layers are present beneath both Blocks 1 and 2. We performed engineering analyses to estimate the potential for liquefaction and liquefaction-induced ground-surface

settlement at the site. Based on our analyses, we conclude these thin layers will generate excess pore pressures during a moderate earthquake and will liquefy during a Magnitude 7-1/2 earthquake generating a ground surface acceleration on the order of 0.66g. This level of shaking is consistent with the design-level earthquake, as defined by the 2007 California Building Code (CBC).

Block 1

Potentially liquefiable soil was encountered at a depth of between about 42 and 50 feet at CPT-3 and between 49 and 50 feet at CPT-4. We estimate ground-surface settlement associated with liquefaction after a major seismic event on a nearby fault will be less than ½ inch at CPT-3 and less than ¼ inch at CPT-4.

Block 2

Potentially liquefiable soil was encountered to the maximum depths explored at CPT-1 and CPT-2 (76.5 and 100 feet bgs, respectively). Liquefaction below 50 feet should not have a significant impact on the performance of the proposed structures. Nonetheless, the design of the foundation systems for the proposed project should take into account the potential loss of strength and settlement associated with the deeper soils. Furthermore, a more thorough investigation involving additional CPTs, deep rotary-wash borings and extensive laboratory testing should be performed to further analyze deep liquefaction prior to construction. Preliminary studies indicate that liquefaction settlement after a major seismic event on a nearby fault will be between ½ and 2 inches in the upper 50 feet. Liquefaction-induced settlement of the soil below 50 feet will be between about 1 and 1-3/4 inches.

SP 117 states that "localized differential settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made". Based on this assumption, liquefaction-induced differential settlement due to liquefiable deposits within the upper 50 feet of less than ½ inch and approximately 1-1/4 in 50 feet should be expected at Block 1 and Block 2, respectively.

5.2.5 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as a shoreline slope, or in the direction of a regional slope or gradient. Based on the discontinuous nature and depth of the potentially liquefiable layers and the lack of controlling boundary conditions, we believe the potential for lateral spreading to occur at the project site is low.

6.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

Based on the results of our preliminary field investigation and engineering studies, we conclude that the proposed project can be developed as planned. The primary geotechnical concerns are:

- geologic hazards associated with strong shaking on a nearby fault
- the selection of an appropriate foundation system for each block
- the presence of compressible clay zones below the site
- shoring for the basement level at Block 1.

These geotechnical concerns and their impact on the proposed foundation design, and construction are discussed in the following sections.

6.1 Mitigation of Liquefaction Hazards – Block 1

As discussed above in Section 5.2.4 our preliminary engineering studies indicate that liquefaction-induced settlement within the upper 50 feet at Block 1 will be between 0 and ½ inch following a major seismic event on a nearby fault. We judge that the proposed development can be designed to resist liquefaction settlements of this magnitude; therefore no mitigation of liquefaction would be required at Block 1. These findings should be confirmed during our final investigation.

6.2 Mitigation of Liquefaction Hazards – Block 2

As discussed in Section 5.2.4 our engineering studies indicate that total settlement from liquefaction of the soil above a depth of 50 feet at Block 2 will be between 1/2 and 2 inches following a major seismic event on a nearby fault. We qualitatively evaluated the following measures which could be used to reduce the liquefaction-induced settlement at Block 2:

- excavation, removal, and recompaction of potentially liquefiable soil
- in-situ ground densification (e.g. compaction with vibratory probes, dynamic consolidation, compaction piles, compaction grouting, etc.)
- ground modification techniques, such as permeation grouting, columnar jet grouting, deep soil mixing, stone columns, gravel or other drains, etc.
- deep foundations.

The liquefiable deposits extend to the depths explored during our investigation; therefore, the excavation, removal and recompaction of potentially liquefiable soil is not practical for this project. In-situ ground densification can be used successfully to mitigate a well-defined liquefiable layer; however, the random nature of the liquefiable deposits at the site and the fines content of the granular soil would make it difficult to use in-situ ground densification at the site. Based on our understanding of the project and the results of our engineering studies, we conclude that soil-cement columns (ground modification) or deep foundations are the most appropriate methods for this project and are discussed in the following paragraphs.

6.2.1 Soil-Cement Columns

Soil-cement columns may be used to strengthen the potentially liquefiable soil and reduce potential settlement from both static and seismic loading. Soil-cement columns are installed by mechanically mixing the in-situ soil with cement grout to a specified depth. This results in a column of soil-cement that is resistant to liquefaction. If a sufficient number of soil-cement columns are installed, it is feasible to limit shear strains in the untreated soil between the soil-cement columns such that liquefaction does not occur.

A number of specialty contractors using proprietary mixing technology have performed soil-cement mixing within the Bay Area. Mixing equipment generally consists of a variation on hollow-stem drilling equipment fitted with a special drill/mixing tool.

We conclude a stiffened shallow foundation system is feasible if the upper soil is treated with soil-cement columns. The required spacing and depth of the columns will depend on the stiffness of the foundation system and its ability to resist differential settlement and span between soil cement columns, as well as the diameter of the soil-cement columns. Further, it appears the required depth of treatment would be less in the vicinity of CPT-2 than at the CPT-1 location. Once column spacing and loads are available for the proposed structures, we can develop preliminary recommendations for spacing and depth of soil-cement columns. Additional subsurface exploration would be required to develop final recommendations.

6.2.2 Deep Foundations

The effects of liquefaction on buildings are commonly mitigated by supporting the buildings on deep foundations extending below the liquefiable soil. Considering the CPT data from Block 2 indicate potentially liquefiable soil extends below a depth of 50 feet, deep foundations may not be practical for the proposed buildings on this block. The feasibility of deep foundations should be evaluated further during the final geotechnical investigation. Deep foundations are discussed below in Section 6.3.2.

6.2.3 Utilities

The design of underground utilities at the site should incorporate the effects of liquefaction-induced settlement that may occur on Block 2. Differential settlement may occur along utilities that extend between treated and untreated soil areas and at utility connections at the perimeter of buildings that are supported on deep foundations or on treated soil. Flexible utility connections and utility hangers should be used, where appropriate.

6.3 Foundations

We judge the presence of potentially liquefiable soil is the most significant factor influencing the selection of a suitable foundation system for Block 2, although the weak, upper clay layer at the location of boring B-2 is also a consideration. There is less potential for liquefaction at Block 1; consequently, the most significant factor influencing the selection of an economical foundation system for Block 1 is the presence of compressible clay zones beneath the building.

6.3.1 Block 1 – Mat Foundation

Based on our knowledge of the subsurface conditions and the plans for a basement at the site, we preliminarily conclude the proposed building on Block 1 can be supported on a mat foundation. Because the weight of the soil that will be removed to construct the basement is greater than the weight of the proposed building, little or no consolidation settlement should occur. Some settlement will occur, however, due to rebound and recompression of the soil underlying the building. Initially, as the proposed basement area is being excavated, we expect the removal of soil and resultant pressure relief will cause the base of the excavation to rebound (rise), especially near the center of the excavation. After the new foundation is constructed and new building loads are applied, the pressures will once again increase and the soil beneath the building will recompress. We compute the settlement associated with this recompression could range between ½ and 1 inch. We estimate post-construction differential settlement between columns may be on the order of ½ inch.

6.3.2 Block 2 – Deep Foundation Alternative

The selection of a deep foundation system should include cost, schedule and the impact of construction on adjacent improvements. A properly designed deep foundation system should limit total foundation settlements to less than 1 inch with no more than 1/2 inch differential settlement, between columns. The presence of deep liquefiable deposits should be taken into account during design of the deep foundation system. Additional studies, as described in Section 5.2.4 will be required prior to final foundation design.

Floor slabs and pavements may be supported on-grade, provided that settlement and the resulting distress to these elements would be acceptable after a large earthquake. Differential settlement between on-grade improvements could be partially mitigated by using a thicker and more heavily reinforced slab-on-grade than usual. The extent to which the differential settlement will be mitigated will depend on the thickness and reinforcing of the slab.

Driven Piles

On the basis of the results of our preliminary engineering studies, we conclude the proposed structures may be supported on driven piles. Pile foundations would derive most of their capacity from friction. From our experience with similar projects, we conclude precast, prestressed concrete piles are the most appropriate driven piles for this site.

The advantage of driven piles over the proprietary piles discussed in the following paragraphs is schedule and cost. The disadvantage of any driven pile installation is the noise, vibration and possible heave which may affect adjacent improvements. Proprietary piles are generally drilled into place, reducing the amount of noise and vibration during installation; however, they can be more time-consuming to install and typically require load tests to check their capacity.

Tubex Piles

Tubex piles consist of a steel pipe pile fitted with an oversized auger tip; the pile is advanced by hydraulically drilling the pile into the subsurface soil. The annular space between the steel pipe and the subgrade soil is filled with cement grout injected through ports in the auger tip. Upon completion, reinforcing steel is placed in the steel pipe and it is filled with concrete.

In the Bay Area, Tubex piles are installed under a design-build contract by Foundation Constructors; therefore, we cannot provide specific recommendations for their design. The allowable compression and uplift capacity given by the designer of the Tubex piles should be confirmed by a load test program prior to production pile installation.

Torque Down 1275 Piles

The Torque Down 1275 Pile (TDP) is a proprietary deep foundation system developed by Substructure Support, Inc. The TDP consists of a 12.75 inch in diameter pipe with a proprietary conical tip with pitched flights that allow the pipe pile to be "screwed-in" the soil or rock, resulting in displacement and densification of the surrounding soil. When the pipe pile is advanced to the design tip elevation, it is filled with structural concrete to provide additional bending resistance. As with Tubex piles, allowable compression and uplift capacities should be provided by their designer and a load test should be performed.

6.3.3 Block 2 – Shallow Foundations with Ground Improvement Alternative

If soil-cement columns are used to mitigate liquefaction of the soils beneath the Block 2, the soil-cement columns may also be used to support either a well-reinforced continuous footing system or a mat foundation. Soil-cement columns may be used to mitigate liquefaction-induced settlement beneath floor slabs and other at-grade improvements; however, if slab distress due to liquefaction-induced settlement is tolerable, ground improvement may not be necessary beneath the slabs. The performance of at-grade improvements will depend on the extent of the soil-cement columns as well as the design of these improvements to resist differential settlement. Liquefaction-induced settlements, as discussed in Section 5.2.4, should be expected to occur in areas where the potentially liquefiable soil is not improved.

6.3.4 Block 2 – Settlement of Floor Slabs and On-Grade Improvements

During a major seismic event on a nearby fault, floor slabs and on-grade improvements which are not structurally supported and where the underlying soils are not mitigated against liquefaction may experience total settlement of between ½ and 2 inches and differential settlement of between ½ and 1-1/4 inches. These estimates should be re-evaluated once the final foundation configuration has been determined. Recommendations for moisture barriers beneath floor slabs are presented in Section 7.6.

6.4 Areal Settlement Due to the Placement of New Fill

With the exception of the basement excavation at Block 1, the preliminary plans do not indicate any significant re-grading of either Block 1 or Block 2. If new fill is placed, some settlement will occur. The amount of settlement will depend on the lateral extent and the new fill thicknes, as well as whether or not soil-cement columns have been installed at the site. We should evaluate settlement further once plans have been finalized.

6.5 Block 1 – Shoring

Based on our understanding of the preliminary plans, there appears to be insufficient space to slope the sides of the excavation; consequently, shoring will be required. Several methods of shoring are available, and the system selected should take into account the requirements for dewatering, protecting adjacent properties as well as cost. We have qualitatively evaluated the following systems:

- conventional soldier pile and lagging
- soil nails
- sheet piles
- concrete of soil-cement walls

Although all of the above shoring systems are feasible for this site, a conventional soldier pile and lagging shoring system would likely be the most economical system. Considering the clay is relatively weak below a depth of 20 feet, it may be costly to design the soldier piles to cantilever. Shorter soldier piles and smaller steel sections could be used if a row of tiebacks is installed; however, it would be necessary to obtain permission to install tiebacks beneath adjacent properties. Dewatering of the static groundwater table should not be required, although it is possible that some perched groundwater may be encountered.

The selection, design, construction, and performance of the shoring system should be the responsibility of the shoring contractor. However, the shoring should be designed by a structural engineer knowledgeable and experienced in this type of construction.

6.6 Dewatering

Our initial studies indicate the static groundwater table is currently between about 22 and 26 feet below existing grades. Considering the low rainfall and the time of year our field investigation was performed, we estimate the groundwater level could rise five feet during years with heavy rainfall; however, we do not expect the groundwater to rise to the level of the proposed basement. Perched water from rainfall infiltration and/or irrigation may be encountered in the proposed basement excavation. We anticipate perched water can be addressed using trench drains and sump pumps. We do not anticipate the need for continuous dewatering at the site.

6.7 Excavation Monitoring

During excavation, any shoring system selected will yield and deform. The movement of the shoring system can cause surrounding improvements to settle and move laterally. The magnitude of shoring movements and resulting ground deformations are difficult to estimate because they depend on many factors, including the type of shoring system used and the contractor's skill in the shoring installation. We believe ground movements for a properly designed and constructed shoring system should be less than about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of adjacent improvements during excavation.

6.8 Soil Corrosivity

Two samples of near-surface soil were analyzed for corrosion potential. The results of the corrosivity analysis are included in Appendix C. On the basis of the test results, the soil is classified as "moderately corrosive to corrosive." Specific recommendations regarding corrosion protection are beyond the scope of this study. A corrosion consultant may be retained to provide specific recommendations regarding corrosion protection for buried utilities, foundation elements and concrete.

6.9 Construction Considerations

Soft, weak and easily disturbed soil may be encountered at the base of the basement excavation for Block 1. To facilitate construction, the contractor may elect to install either a three-inch-thick concrete working slab ("mud slab") or a six-inch thick layer of crushed rock to provide a uniform working surface prior to placing waterproofing and/or reinforcement, especially if the excavation will be open during the rainy season.

Residents of adjacent structures will hear and feel noise and vibration during pile driving. Structures within about 30 feet of the site may be affected by these vibrations. Proprietary piles, which are installed using drilling equipment, will result in minimal vibrations and significantly less noise during installation than driven piles. A pre-construction survey as well as vibration monitoring of the adjacent BART structures and tracks should conducted prior to and during indicator pile driving per the requirements of BART. Additional surveys and vibration monitoring may also be required during production pile installation.

7.0 PRELIMINARY RECOMMENDATIONS

Preliminary recommendations for site preparation and fill placement, soil-cement mixing, foundations, permanent basement walls, slab-on-grade floors, pavement design and seismic design are presented in this section of the report.

7.1 Site Preparation and Fill Placement

Grading operations should commence after demolition and removal of the existing pavements, foundations, concrete slabs, and underground utilities within the proposed development. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the building footprints and 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. Voids resulting from demolition activities should be properly backfilled with engineered fill following the recommendations provided later in this section.

Following demolition, all areas to receive improvements should be stripped of vegetation and organic topsoil. From a geotechnical standpoint, the stripped organic topsoil may be stockpiled for later use in landscaped areas; however, organic topsoil should not be used as compacted fill. Areas to receive fill should be scarified a minimum of eight inches, moisture-conditioned and compacted per the requirements in Table 3. If the on-site expansive clay is to be used as general site fill, it should be moisture-conditioned to at least two percent above optimum moisture content, placed in lifts not exceeding eight inches in uncompacted thickness, and compacted to at least 90 percent relative compaction.

Select fill should consist of imported soil that is free of organic matter, contain no rocks or lumps larger than four inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. Select fill should be placed in lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. Samples of proposed select fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site. The grading contractor 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 imported material.

In areas where wet and/or weak subgrade soils are encountered during subgrade preparation or other grading activities, the weak soil should be removed and replaced with select fill. The compaction requirements, including those for trenches and pavements, are summarized in Table 3 below.

TABLE 3
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
Building pad subgrade- native soil	90+	2+% above optimum
General fill - select fill	90+	Near optimum
General fill - native soil	90+	2+% above optimum
Utility trench backfill - native soil	90+	2+% above optimum
Utility trench backfill - select fill	90+	Near optimum
Utility trench - clean sand or gravel	95+	Near optimum
Pavement subgrade – native soil	95+	Near optimum
Pavement subgrade – imported fill	95+	Near optimum
Pavement - aggregate base	95+	Near optimum
Exterior slabs – native soil subgrade	90+	2+% above optimum
Exterior slabs – select fill	90+	Above optimum

7.1.1 Utility Trenches

Excavations for utility trenches can be readily made with a backhoe. Despite careful site preparation, unexpected obstructions may make some of the trenching operations difficult. All trenches should conform to the current CAL-OSHA requirements.

Backfill for utility trenches and other excavations is also considered fill, and it should be compacted according to the recommendations presented for site preparation and fill placement. 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.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved,

they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped.

7.2 Excavation and Shoring

We understand an excavation is planned at Block 1 for one level of below-grade parking. We do not anticipate any significant cuts at Block 2. The sides of the temporary cuts may be sloped where space permits; slope inclination should comply with OSHA requirements and be no steeper than 1:1 (horizontal:vertical). Where temporary cuts are deeper than ten feet, the slope inclinations should be no steeper than 1.5:1. Where space limitations do not allow for slopes, shoring should be installed.

Depending on the shoring system selected, tiebacks or internal bracing may be required. Lateral earth pressures will depend on the shoring system selected and we can provide lateral pressures for that system at that time. The shoring system extend below the excavation bottom a minimum of five feet and be sufficient to achieve lateral stability and resist the downward loading of the tiebacks.

If traffic occurs within 10 feet of the shoring depth, a uniform surcharge load of 100 psf should be added to the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable.

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

7.3 Foundations

We preliminarily recommend the proposed building on Block 1 be supported on a mat foundation system. The selection of the foundation system for the proposed structures on Block 2 should be determined by joint consultation between us, the structural engineer, and the owner. Our recommendations for the various schemes judged to be feasible are discussed in the following sections of this report.a

7.3.1 Block 1 – Mat Foundation

The bearing pressures and settlement estimates presented herein for the recommended mat foundation are based on a basement finished floor elevation of about 10 feet below existing grade. We recommend the mat foundation be designed using a modulus of vertical subgrade reaction of 10 pounds per cubic inch (pci); this value includes a width correction factor. The average bearing stress under the mat is expected to be low (less than 700 psf) due to the large plan area; however, concentrated stresses may occur at the edge of the mat and at columns. For this condition, the maximum local bearing stress should not exceed 2,000 psf for dead-plus-live loads; this may be increased by one-third for transient loads.

7.3.2 Block 2 – Deep Foundations

If driven concrete piles are chosen as the foundation system for the project, we recommend 14-inch-square, precast, prestressed, concrete piles be used. Proprietary piles should be designed by the design-build contractor. All piles, regardless of type, should gain support below a depth of 50 feet bgs. Furthermore, piles will experience downdrag due to liquefaction-induced settlement after a major seismic event on a nearby fault. For planning purposes, we estimate that pile lengths on the order of 90 to 100 feet will be required. Allowable dead-plus-live-load pile capacities should be between 200 and 250 kips. We should review the geotechnical aspects of their plans and specifications prior to final design.

Tubex and Torque Down Piles

Tubex and TDP should gain support in friction below a depth of 50 feet. The lengths and pile capacities should be determined by the installation contractor and reviewed by us.

The allowable compression capacity of Tubex and TDP piles should be confirmed by a load test program. We recommend testing at least two piles in compression. The piles should be loaded to at least 200 percent of the design compression load (dead-plus-live conditions) plus the estimated downdrag load that will be imposed by the upper 50 feet of soil following an earthquake. The piles should be tested in general conformance with the standard method given in ASTM D1143-81 and D3689-90 for static load tests in compression and tension, respectively. We should review the contractors test pile program prior to their mobilization on site.

Lateral Load Resistance

Any pile selected for the project will develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on:

- the pile stiffness
- the strength of the surrounding soil at the time the load is applied
- axial load on the pile
- the allowable deflection at the pile top
- the allowable moment capacity of the pile.

Once the pile type has been selected, we can develop deflection and moment profiles for single piles with depth as required by the pile designer. For planning purposes, lateral loads associated with 1/2 inch of pile-head movement for 14-inch driven concrete piles will be on the order of 10 to 15 kips and 25 to 30 kips for free- and fixed-head conditions, respectively. We should review the geotechnical aspects of any deflection and moment profiles prepared by a proprietary pile contractor. To account for group effects, the computed lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown on Table 4. We can analyze

other pile cap configurations not shown on Table 4 as required. The moment profile for a single pile with an unfactored load should be used to check the design of individual piles in a group.

TABLE 4
Pile Group Reduction Factors

Number of Piles in Pile Group	Single Pile Reduction Factor
3	0.84
4	0.80
5	0.76

Where piles are spaced at least eight pile widths in the direction of loading, no group reduction factor is necessary. The effects of liquefaction during a major seismic event should be included in the lateral pile analysis for the selected pile.

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. The passive resistance given in Table 5, Section 7.5, may be used to compute lateral load resistance for pile caps and grade beams. The upper foot of soil should be ignored in computing passive resistance, where not confined by a slab or pavement.

Temporary Downdrag Due to Liquefaction

The computed axial pile capacities should be reduced by temporary downdrag loads acting on the piles due to post-liquefaction reconsolidation. The temporary downdrag load resulting from post-liquefaction reconsolidation will depend on the pile type selected. For a 14-inch-square concrete pile, we estimate the maximum temporary downdrag will be 150 kips. The magnitude of the downdrag load is proportional to the depth of the potentially liquefiable layers beneath the pile cap. Because the downdrag load will occur after an earthquake (as the pore pressures in the liquefied soil dissipate), we conclude it would be appropriate to not add the downdrag load to the seismic load when checking the structural capacity of the pile. We conclude that the following design parameters should be used to evaluate post-liquefaction downdrag loads:

- A minimum factor of safety of 1.3 is maintained during post-liquefaction reconsolidation.
- The factor of safety is calculated by dividing the ultimate pile capacity by the sum of the static pile load and the downdrag load.

Indicator Pile Program

Before production piles are cast, we recommend at least 10 indicator piles be driven within each building footprint on Block 2. The indicator pile program will provide driving resistance data to correlate with information obtained during the field investigation and be used as the basis for establishing final production pile lengths. Indicator piles should be installed at production pile locations selected by us and approved by the structural engineer.

We recommend the indicator piles be at least 10 feet longer than production piles. The extra length will provide flexibility to drive piles deeper than planned, if necessary. The indicator piles should be driven using the same equipment that will be used to drive the production piles. Sufficient reinforcement should be provided in the top portions of the indicator piles to allow cutting off 10 feet without impairing their capacity.

Pile Installation

Selection of pile-driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length. Special consideration should be given to selecting a hammer which can deliver enough energy to the tip of the piles to drive them efficiently without damaging them. For 14-inch-square concrete piles, we recommend a hammer delivering as least 80,000 foot-pounds of energy per blow be specified.

7.3.3 Block 2 – Shallow Foundation with Soil Improvement

As an alternative to piles, the potentially liquefiable soil beneath Block 2 may be improved using soil-cement columns and the buildings supported on a mat slab or well-reinforced continuous footings. The determination of whether to use a mat slab or continuous footings should be based on discussions between us, the structural engineer and the owner. Furthermore, the final selection of the appropriate foundation system should be based on a detailed final report.

Well -Reinforced Continuous Footings

If a shallow foundation system with soil improvement is used, continuous footings should be at least 18 inches wide and should be bottomed a minimum of 18 inches below the lowest adjacent grade. The footings may be designed using allowable bearing pressures of 5,000 pounds per square foot (psf) for dead plus live loads and 6,500 psf for total design loads, which include wind or seismic forces. The continuous footing system should be designed for differential settlements due to liquefaction which are not mitigated by the soil-cement columns.

Lateral loads may be resisted by a combination of passive earth pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting clay. We recommend using the passive pressure presented in Section 7.5, Table 5. The upper foot of soil should be ignored unless it is confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30. These passive pressure and frictional resistance values include a factor of safety of at least 1.5.

The footing excavations should be free of standing water, debris, and disturbed material prior to placing concrete. The sides of the footing excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. The bottom of the footing excavation should bear directly on to the soil-cement columns. The soil-cement columns should be continuous below and extend to the perimeter of the entire footing. We should check the footing excavations prior to placing reinforcing steel.

Mat Design

If a mat foundation is used in conjunction with soil-cement columns, we recommend using a modulus of vertical subgrade reaction of 70 pounds per cubic inch (pci); this value does not include a width correction factor. The average bearing stress under the mat is expected to be relatively low due to the large plan area; however, concentrated stresses may occur at the edge of the mat and at columns. For this condition, the maximum local bearing stress should not exceed 5,000 psf for dead plus live loads and 6,500 psf for total loads. It is likely that the soil-cement

columns will not be continuous below the mat foundation; therefore, the mat foundation subgrade soil should be kept moist until concrete is poured.

7.4 Soil-Cement Columns

If soil-cement columns are used for ground improvement, we recommend a specialty contractor be contacted to design the improvement program. This type of construction is highly specialized and should be performed on a design-build basis. Settlements associated with soil-cement columns should be addressed by the specialty contractor.

The same issues concerning the floor slab, as previously discussed in this report, should be addressed in the soil improvement design. Prior to proceeding with the ground improvement program, we recommend the soil-cement columns be performed in a test section. A hole should be cored through at least two of the soil-cement columns to check the strength and uniformity of the columns.

7.5 Basement Walls

We recommend all retaining walls be designed to resist lateral pressures imposed by the adjacent soil, structures and vehicles. Accordingly, walls should be designed for the pressures presented in Table 5, where H is the height of the wall in feet.

A traffic surcharge of 100 pounds per square foot (psf) should be added to the top 10 feet of walls where traffic is expected within 10 feet of the walls.

The lateral earth pressures given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining is to place a prefabricated drainage panel against the back side of the wall. We should check the manufacturer's specifications regarding the proposed prefabrication drainage panel material to verify it is appropriate for its intended use. To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints.

TABLE 5
Lateral Earth Pressures

Soil Type	Unrestrained Walls	Restrained Walls	Seismic Conditions[7]	Passive Resistance[8]
Clay	40 pcf	60 pcf	40 pcf + 21H psf	2000 psf

7.6 Floor Slabs

As discussed in previous sections, floor slabs may be structurally supported or supported on-grade and designed for differential settlement due to liquefaction. At Block 1, we anticipate the mat foundation will be used as the basement floor slab. Considering the mat foundation will be founded on the order of 12 to 13 feet below existing grade (depending on the mat thickness) and at least five feet above the highest anticipated groundwater level, waterproofing of the mat foundation is not required. If moisture-sensitive material will be stored on the floor slab, we recommend placing either a water-proofing membrane or a vapor retarder below the mat. The vapor retarder, if used, should meet the requirements in the following section. Placement of a capillary break below the mat is not recommended.

If a conventional slab-on-grade floor or structurally supported floor slab is used for either of the proposed buildings on Block 2, we recommend at least 12 inches of select fill be placed below the slab to mitigate the potential adverse effects of the moderately expansive near-surface soil at the site. If a mat foundation is used to support the building, placement of select fill is not necessary.

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[7] Wall should be designed for the more critical loading condition of restrained or seismic conditions.

[8] Passive resistance includes a factor of safety of 1.5.

If it is essential to prevent moisture accumulation through floor slabs for the at-grade buildings, we recommend installing a capillary moisture break and a water vapor retarder beneath the floor. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 6.

The sand overlying the membrane should be 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 should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

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

TABLE 6
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

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

7.7 Pavement Design

The following sections provides geotechnical recommendations regarding concrete and flexible pavers pavements. Pavement soil subgrade should be prepared according to the recommendations provided in Section 7.1. Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

7.7.1 Concrete Pavements

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle of 32,000 pounds. The recommended rigid pavement section for these axle loads is six inches of Portland cement concrete over six inches of Class 2 aggregate base for slabs that receive sporadic truck traffic (i.e., a few trucks per week).

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For loading docks, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch-spacing in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described for asphalt pavement.

7.7.2 Interlocking Pavers

If interlocking pavers are used, we should provide recommendations once they have been selected. For planning purposes, interlocking pavers should be placed on a minimum of two inches of sand overlying at least six inches of Class 2 aggregate base.

7.7.3 Sidewalks/Patios/Courtyards

Exterior concrete slabs should be supported on compacted subgrade and at least four inches of Class 2 aggregate base. The subgrade and baserock should be compacted to at least 90 percent relative compaction and provide a smooth, non-yielding surface for support of the concrete slabs.

7.8 Seismic Design

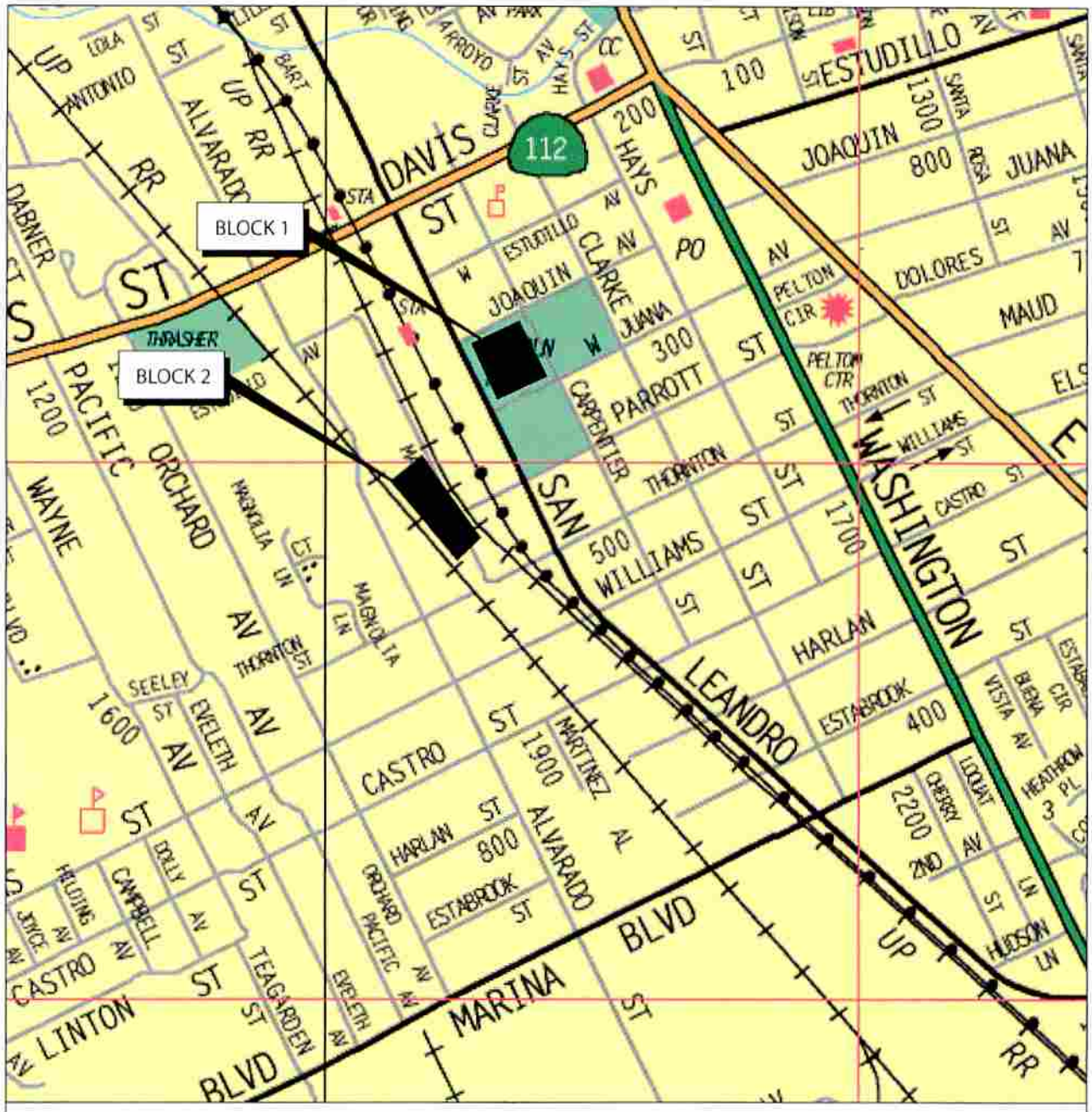
For seismic design in accordance with the provisions of California Building Code (CBC) 2007, we recommend using Site Class D with site modification factors Fa and Fv of 1.0 and 1.5, respectively.

The mapped site class B short (SS) and one second (S1) spectral values for Maximum Considered Earthquake (MCE) for the project site are 1.783g and 0.663g, respectively. Using the site class D modification factors, the corresponding SMS and SM1 for the project site are 1.783g and 0.995g.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

The conclusions and recommendations presented within are based on a preliminary field investigation and not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on a supplemental field investigation. Additional deep borings and CPTs will be required to further evaluate the liquefaction potential of the soil beneath the site. Once our final report has been completed, the design team has selected a foundation system, and prior to construction, we should review the project plans and specifications to check their conformance with the intent of our final recommendations. During construction, we should observe site preparation, soil improvement, foundation installation, shoring installation, and the placement and compaction of backfill. These observations will allow us to compare the actual with the anticipated soil conditions and to check if the contractor's work conforms with the geotechnical aspects of the plans and specifications.

FIGURES

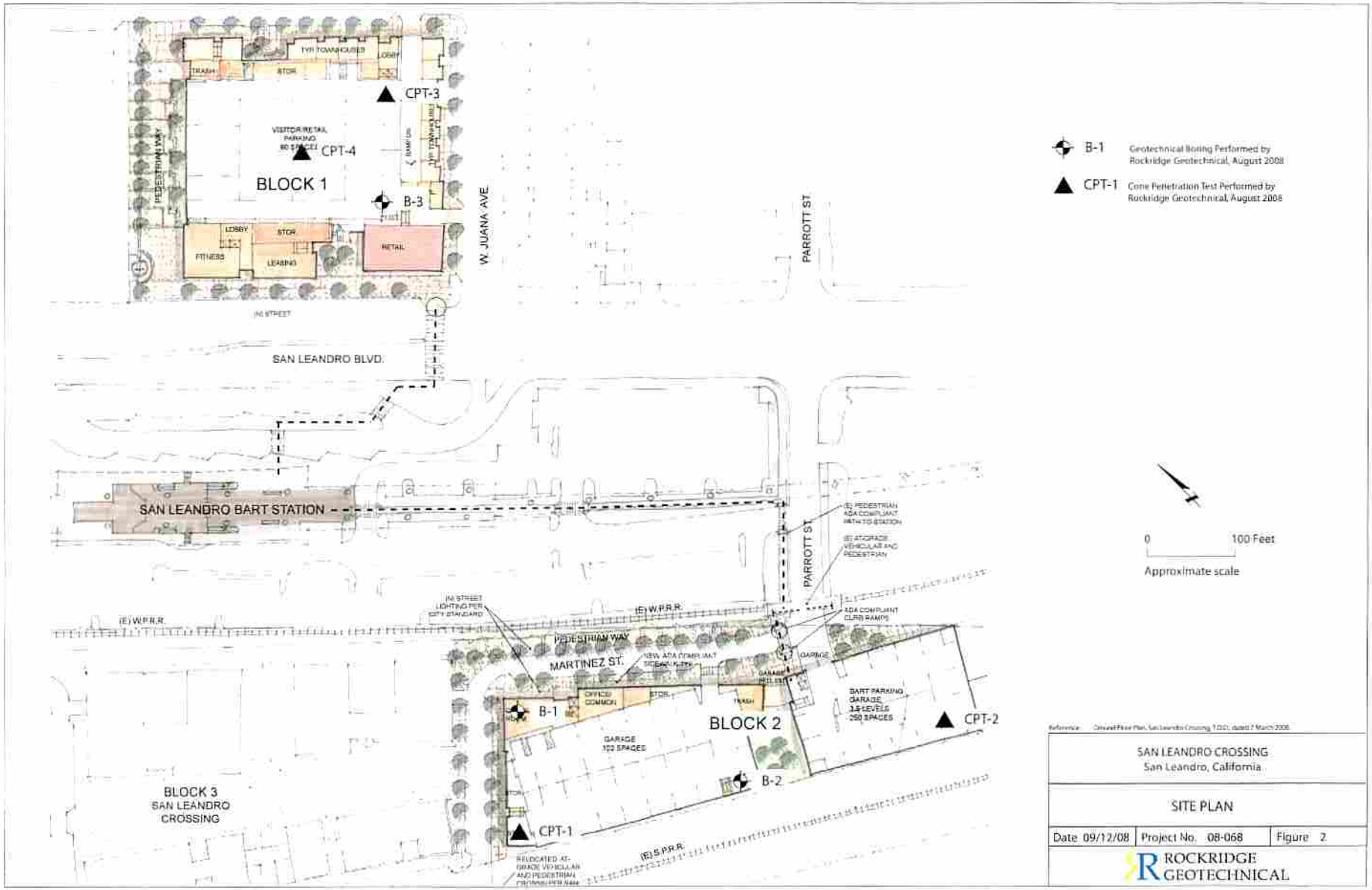


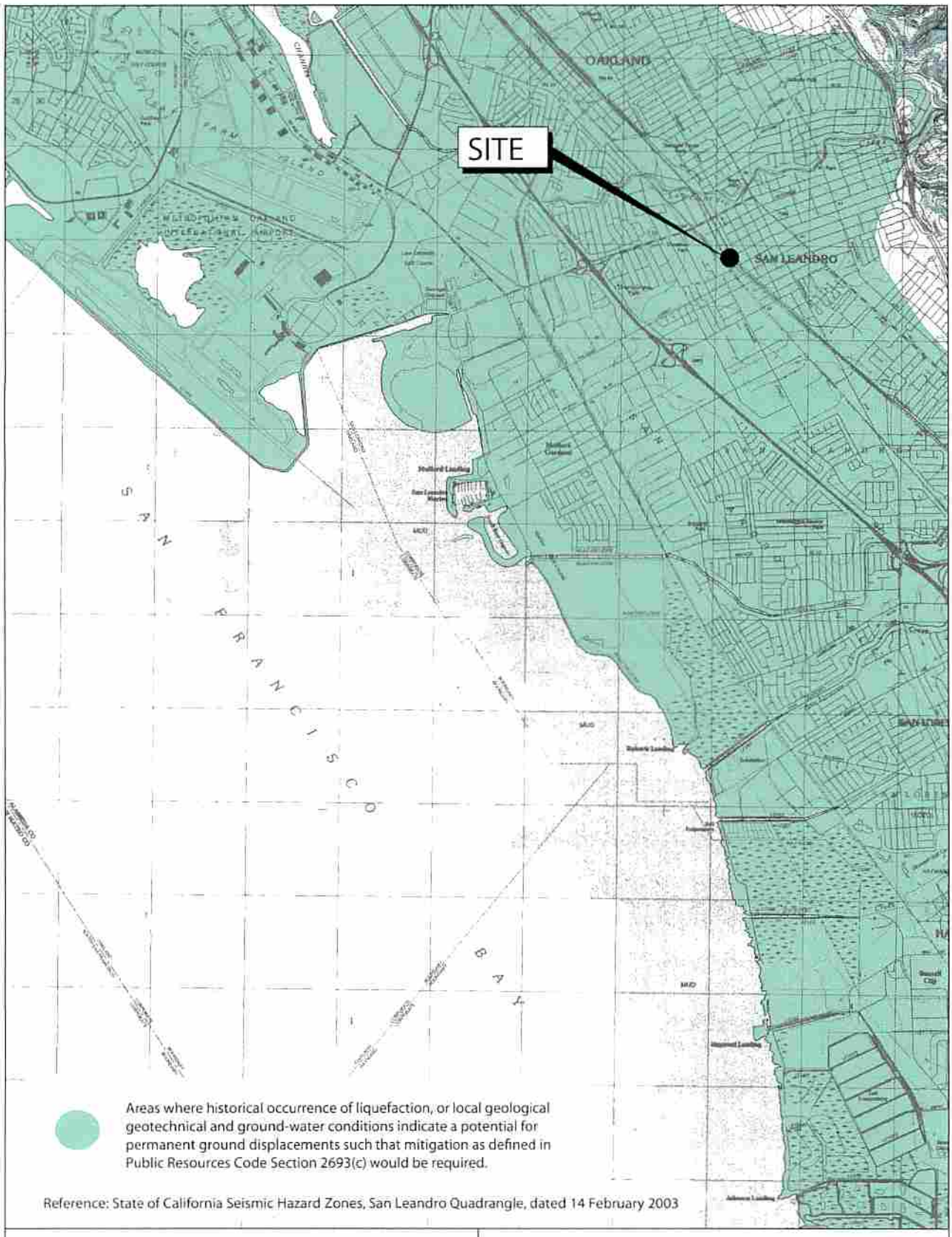
Base map: The Thomas Guide
San Francisco County
1999

0 1/4 1/2 Mile

Approximate scale

SAN LEANDRO CROSSING San Leandro, California	SITE LOCATION MAP
ROCKRIDGE GEOTECHNICAL	Date 09/12/08 | Project No. 08-068 | Figure 1

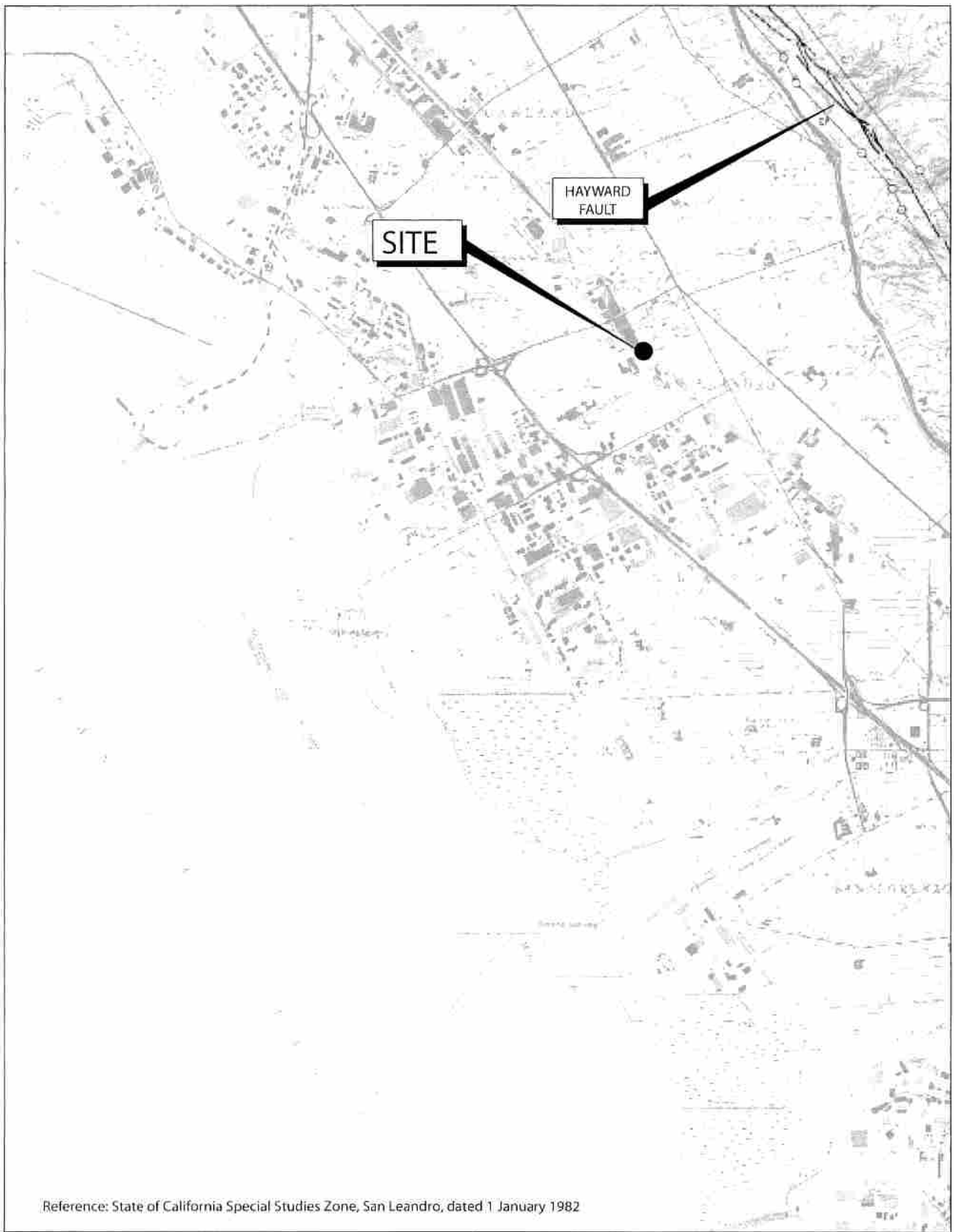

B-1 Geotechnical Boring Performed by Rockridge Geotechnical, August 2008
CPT-1 Cone Penetration Test Performed by Rockridge Geotechnical, August 2008
0 100 Feet
Approximate scale
BLOCK 1
BLOCK 2
BLOCK 3
SAN LEANDRO CROSSING
SAN LEANDRO BART STATION
SAN LEANDRO BLVD.
W. JUANA AVE.
PARROTT ST.
MARTINEZ ST.
PEDESTRIAN WAY
(E) W.P.R.R.
(E) S.P.R.R.
CPT-1
CPT-2
CPT-3
CPT-4
B-1
B-2
B-3
SAN LEANDRO CROSSING
San Leandro, California
SITE PLAN
Date 09/12/08
Project No. 08-068
Figure 2
ROCKRIDGE GEOTECHNICAL



Areas where historical occurrence of liquefaction, or local geological geotechnical and ground-water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Reference: State of California Seismic Hazard Zones, San Leandro Quadrangle, dated 14 February 2003

SAN LEANDRO CROSSING San Leandro, California	SEISMIC HAZARD ZONE MAP SAN LEANDRO QUADRANGLE
ROCKRIDGE GEOTECHNICAL	Date 09/12/08 | Project No. 08-068 | Figure 3



Reference: State of California Special Studies Zone, San Leandro, dated 1 January 1982

SAN LEANDRO CROSSING San Leandro, California	FAULT MAP SAN LEANDRO QUADRANGLE
ROCKRIDGE GEOTECHNICAL	Date 09/12/08 | Project No. 08-068 | Figure 4

APPENDIX A

Boring Logs and Cone Penetration Test Results

ROCKRIDGE GEOTECHNICAL

PROJECT: SAN LEANDRO CROSSING
San Leandro, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2

Date started: 23 August 2008 | Date finished: 23 August 2008

Logged by: C. Divis

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./ 30 inches | Hammer type: Downhole, Wireline

Sampler: SPT, S&H, ST

Ground Surface Elevation: 44 feet[2]

DEPTH (feet)	SAMPLES: Sampler/ Type	Sample	Blows/ foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines, percent	Natural Moisture Content, percent	Dry Density Lbs/Cu Ft
1				CL	CLAY with SAND (CL) mottled brown, dark brown, stiff to very stiff, moist						
2											
3	S&H		11		LL = 35, PI = 19; see Figure B-1	UC		2965		17.9	104
4	S&H		11								
5											
6											
7											
8											
9	S&H		14		dark brown	UC		3400		19.2	109
10											
11	ST		250 psi								
12					Consolidation Test; See Figure B-3					22.5	101
13											
14											
15											
16	ST		400 psi		very stiff					23.1	103
17											
18											
19	S&H		8							28.5	97
20					light brown, stiff						
21	ST		0 psi		Consolidation Test; See Figure B-4					28.8	92
22											
23				SC							
24	SPT		18		▼ 8/23/08; 12:54 LL = 31, PI = 16; see Figure B-1				21.3	17.1	
25					CLAYEY SAND (SC) brown/black, medium dense, wet						
26											
27											
28				SC	CLAYEY SAND with GRAVEL (SC) brown/black, dense, wet						
29	SPT		37								
30											

Project No. 08-068

Figure A-1a

ROCKRIDGE GEOTECHNICAL

PROJECT: SAN LEANDRO CROSSING
San Leandro, California

Log of Boring B-1

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines, percent	Natural Moisture Content, percent	Dry Density Lbs/Cu Ft
31				CL	SANDY CLAY with GRAVEL (CL) (continued)						
32											
33				SC	CLAYEY SAND with GRAVEL (SC) brown/black, medium dense, wet						
34	SPT		23		LL = 32, PI = 16; see Figure B-1				12.1	13.9	
35											
36											
37											
38											
39	SPT		37		dense						
40											
41											
42				GC	GRAVEL with SAND (GP) brown/black, dense, wet						
43											
44	SPT		39								
45											
46											
47											
48											
49	SPT		37								
50											
51					Boring terminated at a depth of 50 feet. Boring backfilled with Cement Grout Groundwater observed at 23.5 feet bgs						
52											
53											
54					1 S&H blow counts converted to SPT N-values using a factor of 0.6. 2 Elevation based on San Leandro City Datum						
55											
56											
57											
58											
59											
60											

Project No. 08-068

Figure A-1b

ROCKRIDGE GEOTECHNICAL

PROJECT: SAN LEANDRO CROSSING
San Leandro, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2

Date started: 23 August 2008 | Date finished: 23 August 2008 | Logged by: C. Divis

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./ 30 inches | Hammer type: Downhole, Wireline

Sampler: SPT, S&H, ST

Ground Surface Elevation: 45 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines, percent	Natural Moisture Content, percent	Dry Density Lbs/Cu Ft
1					SANDY CLAY (CL) brown, medium stiff, moist						
2											
3	S&H		6		Direct Shear Test; See Figure B-7				65.5	17.9	104
4	S&H		6		Corrosivity analysis, see Appendix C						
5											
6											
7											
8											
9	S&H		7						76.5	20.4	103
10				CL							
11	ST		50 psi		LL = 37, PI = 20; see Figure B-2						
12					Consolidation Test; See Figure B-5					20.7	103
13											
14	S&H		17		stiff to very stiff					21.6	99
15											
16											
17											
18											
19	S&H		18								
20					olive brown	UC		2160		23.6	100
21	ST		0 psi		LL = 38, PI = 20; see Figure B-2				62.2	17.5	
22											
23				SC	CLAYEY SAND (SC) gray, medium dense, moist				46.6	12.8	
24	S&H		8							15.5	121
25				CL	CLAY with SAND (CL) grey-brown, stiff, wet					19.4 14.4	113 117
26					▼ 8/23/08; 10:46						
27										17.1	
28				SC	CLAYEY SAND with GRAVEL (SC) gray, medium dense/very stiff, wet						
29	SPT		22		LL = 32, PI = 17, see Figure B-2 Grain-size analysis, see Figure B-8				18.2		
30											

Project No. 08-068 | Figure A-2a

ROCKRIDGE GEOTECHNICAL

PROJECT: SAN LEANDRO CROSSING
San Leandro, California

Log of Boring B-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines, percent	Natural Moisture Content, percent	Dry Density Lbs/Cu Ft
31				SC	CLAYEY SAND with GRAVEL(SC) (continued)						
32					GRAVEL (GP) brown/black/red, medium dense, wet						
33	SPT		24								
34											
35				CL	SANDY CLAY (CL) brown, very stiff, wet						
36											
37											
38				SP-SC	SAND with CLAY (SP-SC) brown, loose, wet						
39	S&H		11		CLAY (CL) brown, medium stiff to stiff, wet						
40											
41	ST		100 psi								
42											
43											
44	S&H		24	CL	very stiff						
45											
46											
47											
48	S&H		22								
49					SILTY SAND (SM) brown, medium dense, wet				19.7	23.4	101
50											
51				SM							
52											
53											
54	SPT		45	GP-GC	GRAVEL with CLAY (GP-GC) brown/black, dense, wet						
55											
56					Boring terminated at a depth of 55 feet. Boring backfilled with Cement Grout Groundwater observed at 26 feet bgs						
57											
58					1 S&H blow counts converted to SPT N-values using a factor of 0.6.						
59					2 Elevation based on San Leandro City Datum						
60											

Project No. 08-068

Figure A-2b

ROCKRIDGE GEOTECHNICAL

PROJECT: SAN LEANDRO CROSSING
San Leandro, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2

Date started: 23 August 2008 | Date finished: 23 August 2008 | Logged by: C. Divis

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./ 30 inches | Hammer type: Downhole, Wireline

Sampler: SPT, S&H, ST

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 45 feet[2])	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines, percent	Natural Moisture Content, percent	Dry Density Lbs/Cu Ft
					3.5 inches asphalt concrete over 10 inches aggregate base						
1				CL							
2					CLAY (CL) light to dark brown, stiff to very stiff, moist						
3	S&H		15		LL = 48, PI = 26; see Figure B-2					10.8	109
4	S&H		22							16.5	102
5											
6											
7											
8											
9	S&H		17		Corrosivity analysis, see Appendix C						
10											
11											
12											
13											
14	S&H		22		hard						
15						UC		4500		21.6	105
16											
17											
18											
19	S&H		9		sandy, stiff					16.4	103
20											
21	ST		0 psi		Consolidation Test; See Figure B-6 medium stiff, wet					19.2	102
22											
23											
24	S&H		7		▼ 8/23/08; 8:27					18.5	116
25				GP	with gravel						
26											
27											
28					GRAVEL with CLAY (GP) brown, dense, wet						
29	SPT		37								
30											

Project No. 08-068 | Figure A-3a

ROCKRIDGE GEOTECHNICAL

PROJECT: SAN LEANDRO CROSSING
San Leandro, California

Log of Boring B-3

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines, percent	Natural Moisture Content, percent	Dry Density Lbs/Cu Ft
31				GC	GRAVEL with CLAY (GC) (continued)						
32											
33											
34	SPT		31								
35											
36											
37				CL	CLAY (CL) olive, medium stiff, wet						
38											
39	S&H		7							28.1	94.3
40				CL/SC	SANDY CLAY (CL)/CLAYEY SAND (SC) brown/black, medium stiff/medium dense, wet						
41	ST		40 psi								
42											
43				SP	SAND with CLAY and GRAVEL (SP) brown, medium dense, wet						
44	S&H		24								
45											
46											
47				SC	CLAYEY SAND with GRAVEL (SC) brown/black, very dense, wet						
48											
49	SPT		53								
50											
51					Boring terminated at a depth of 50 feet. Boring backfilled with Cement Grout Groundwater observed at 23.5 feet bgs						
52											
53					1 S&H blow counts converted to SPT N-values using a factor of 0.6. 2 Elevation based on San Leandro City Datum						
54											
55											
56											
57											
58											
59											
60											

Project No. 08-068

Figure A-3b

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with split-barrel sampler other than Standard Penetration Test sampler. 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
- Groundwater level at the time and date indicated

SAMPLER TYPE

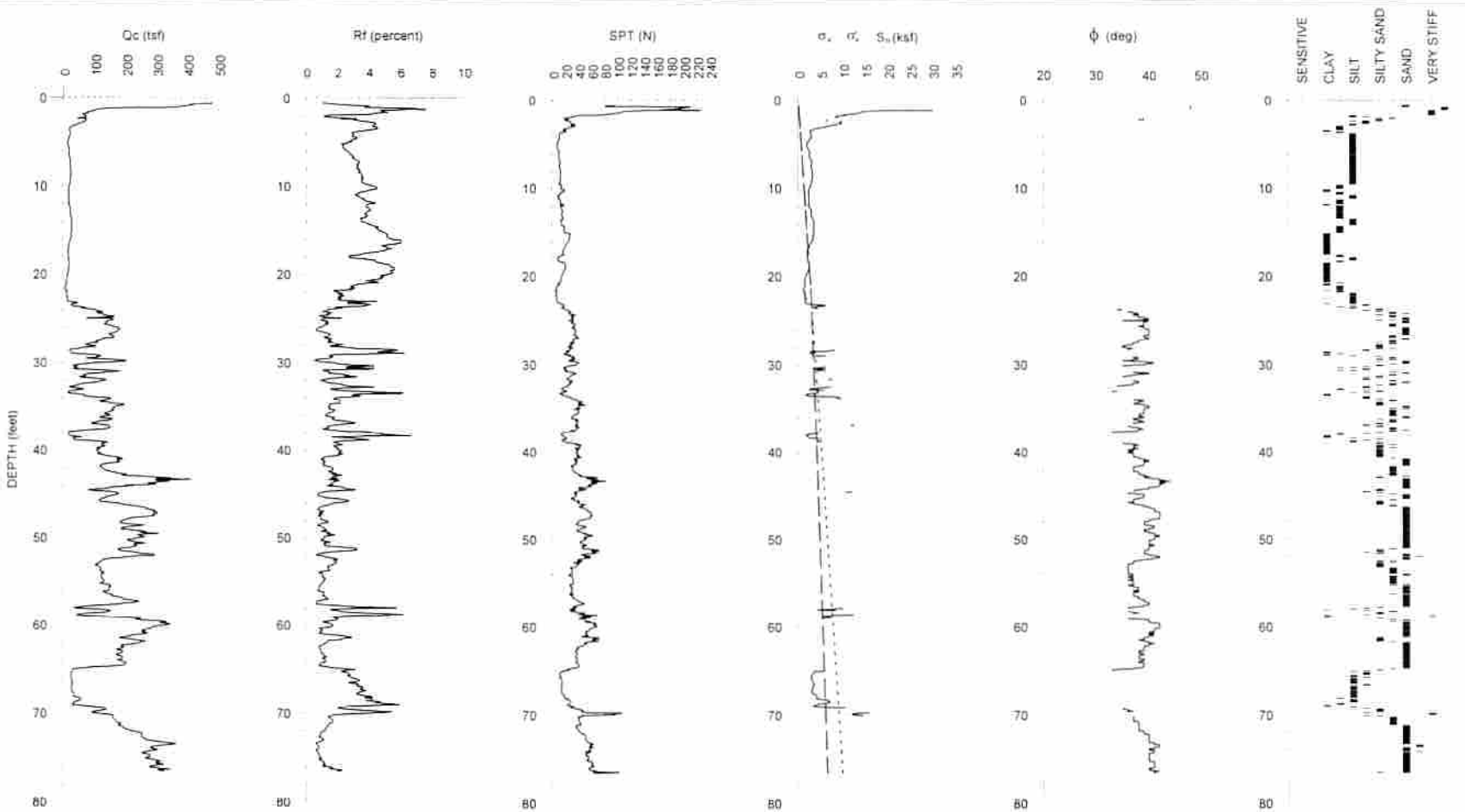
C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler 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

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 9/12/08 | Project No. 08-068 | Figure A-4



Effective vertical stress, σ'_v

Total vertical stress σ_v

Strength, S_u

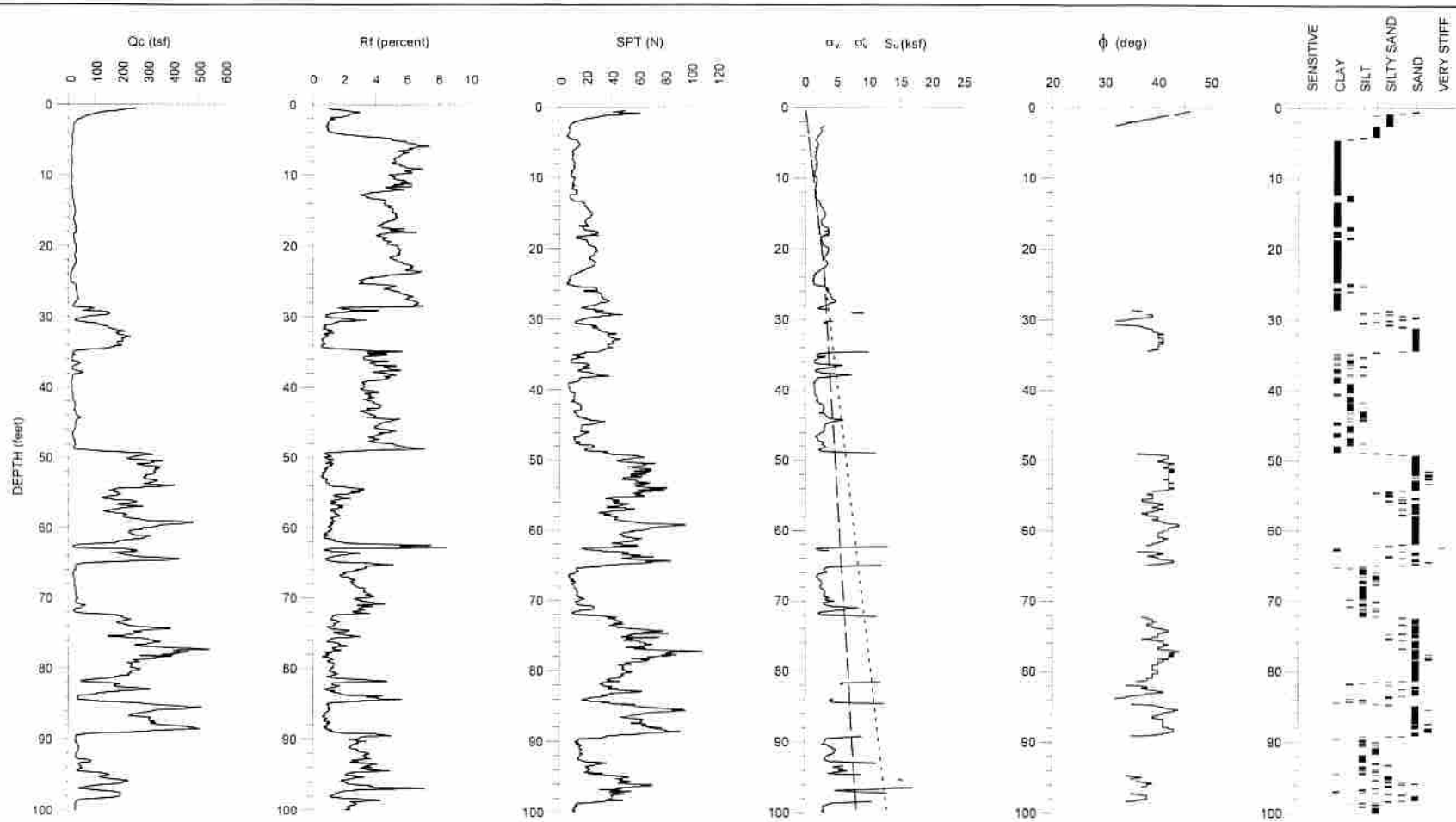
Terminated at 76.5 feet
Groundwater measured at 22.6 feet.
Date performed: 8/20/08

SAN LEANDRO CROSSING
San Leandro, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 10/14/08	Project No. 08-068	Figure A-5

ROCKRIDGE GEOTECHNICAL



Effective vertical stress, σ'_v

Total vertical stress, σ_v

Strength, S_u

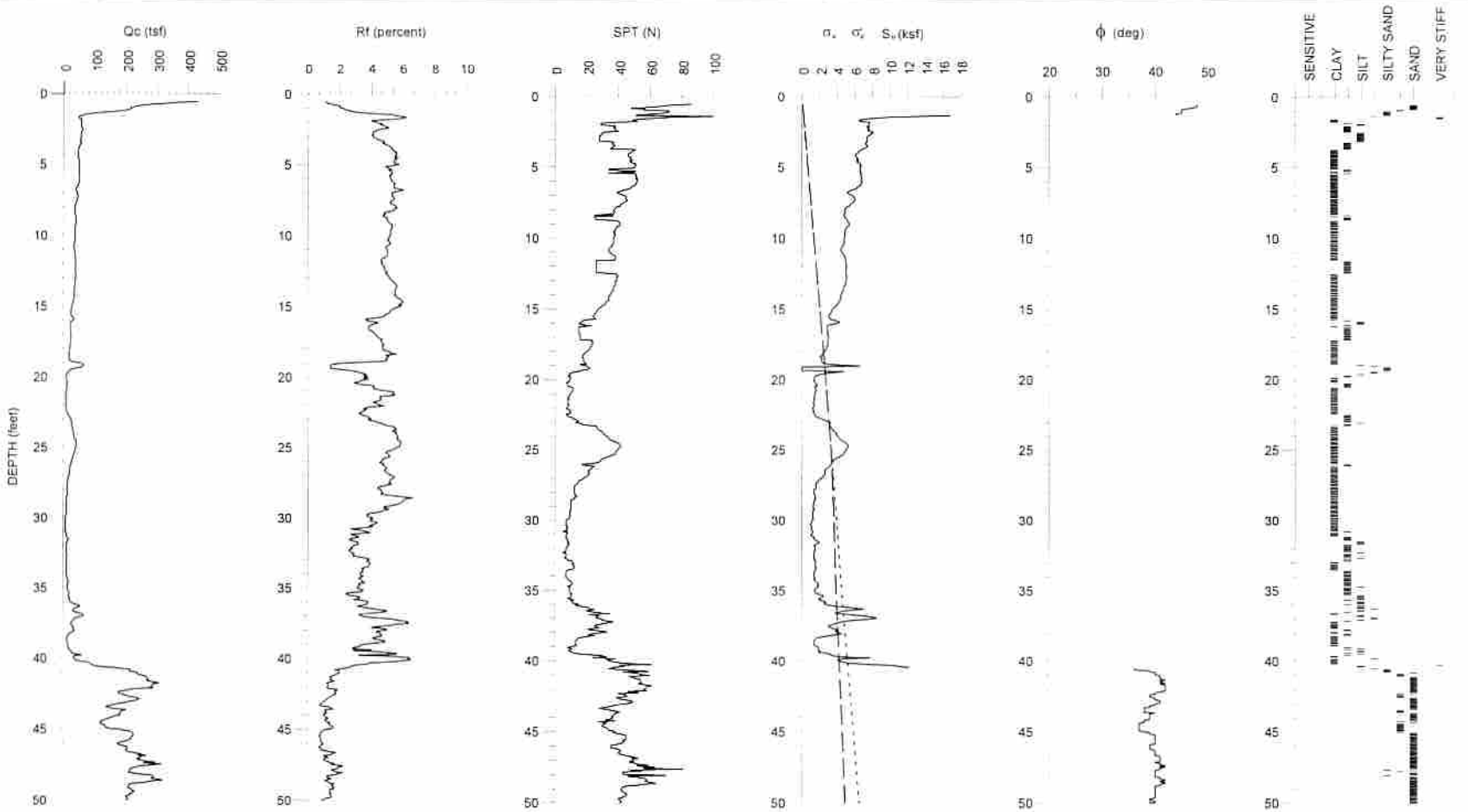
Terminated at 100 feet
Groundwater measured at 22.5 feet.
Date performed: 8/20/08

SAN LEANDRO CROSSING
San Leandro, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 10/14/08	Project No. 08-068	Figure A-6

ROCKRIDGE GEOTECHNICAL



Effective vertical stress, σ'_v

Total vertical stress, σ_v

Strength, S_u

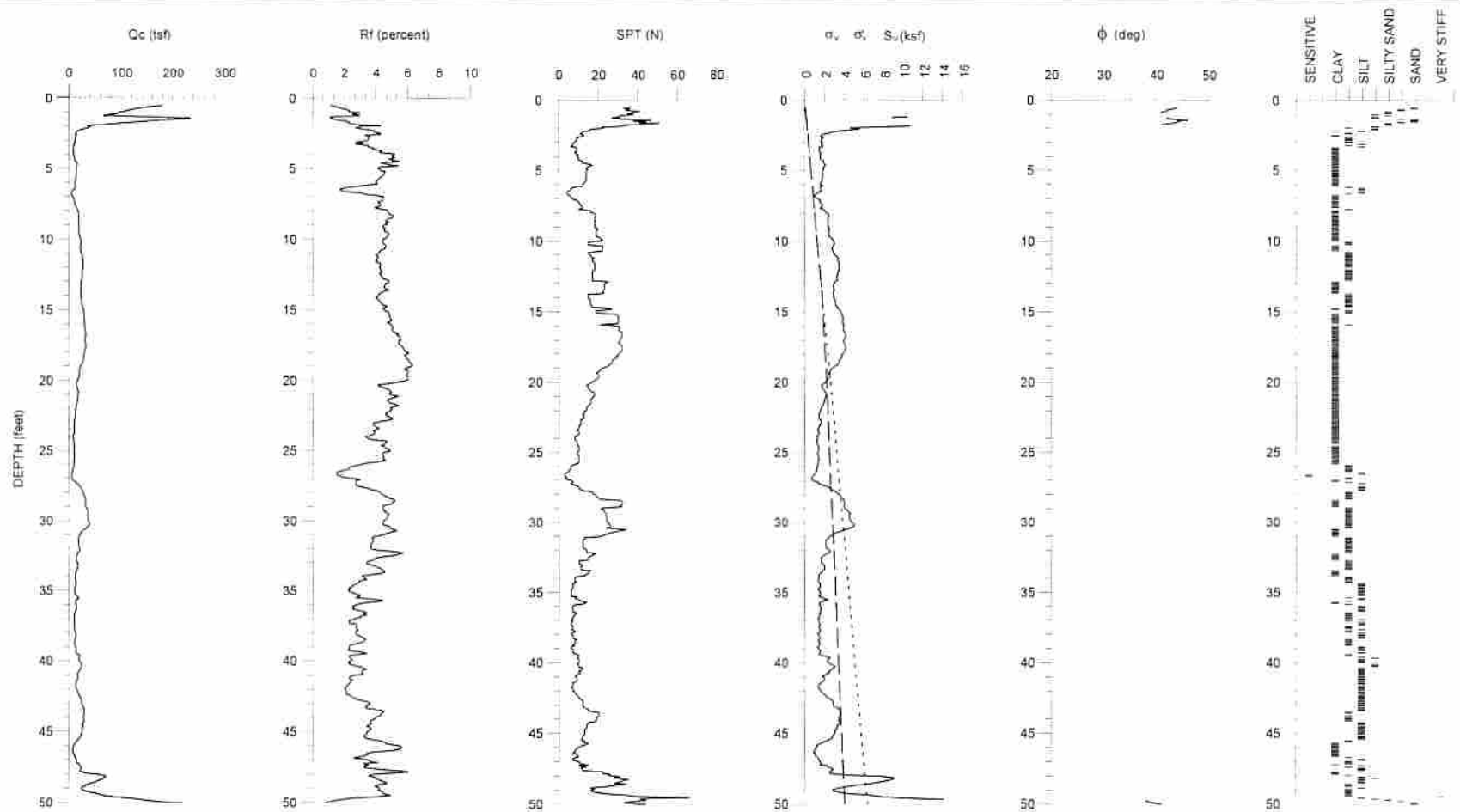
Terminated at 50 feet
Groundwater measured at 24.9 feet.
Date performed: 8/30/08

SAN LEANDRO CROSSING
San Leandro, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 10/14/08	Project No. 08-068	Figure A-7

ROCKRIDGE GEOTECHNICAL



Terminated at 50 feet
Groundwater measured at 13.6 feet.
Date performed: 8/30/08

Effective vertical stress, σ'_v

Total vertical stress, σ_v

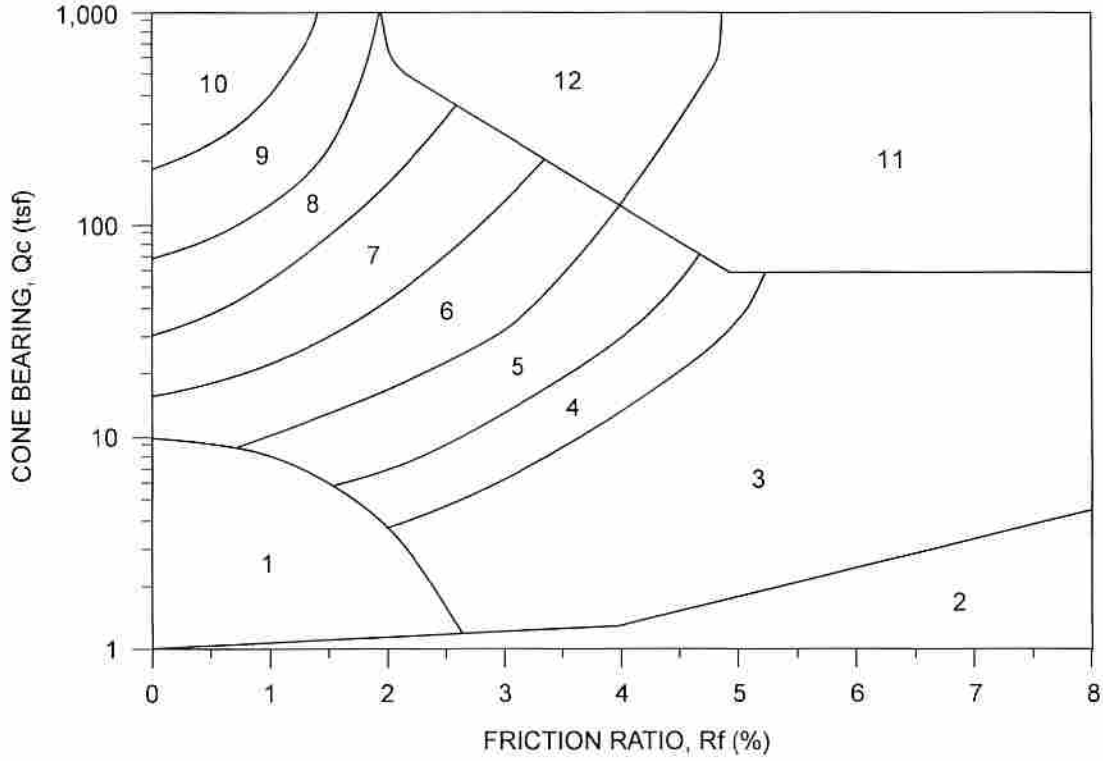
Strength, S_u

SAN LEANDRO CROSSING
San Leandro, California

CONE PENETRATION TEST RESULTS
CPT-4

Date 10/14/08	Project No. 08-068	Figure A-8

ROCKRIDGE GEOTECHNICAL



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).

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

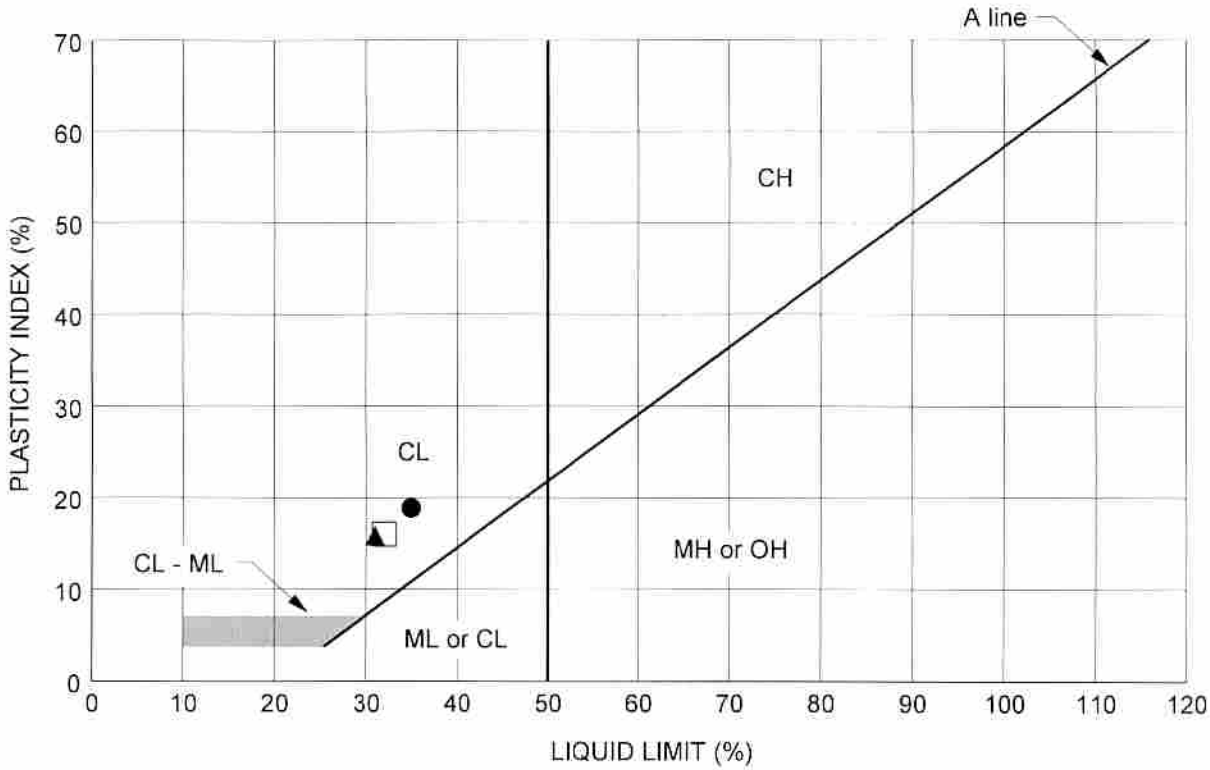
CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 9/12/08	Project No. 08-068	Figure A-9

ROCKRIDGE GEOTECHNICAL

APPENDIX B

Laboratory Test Results



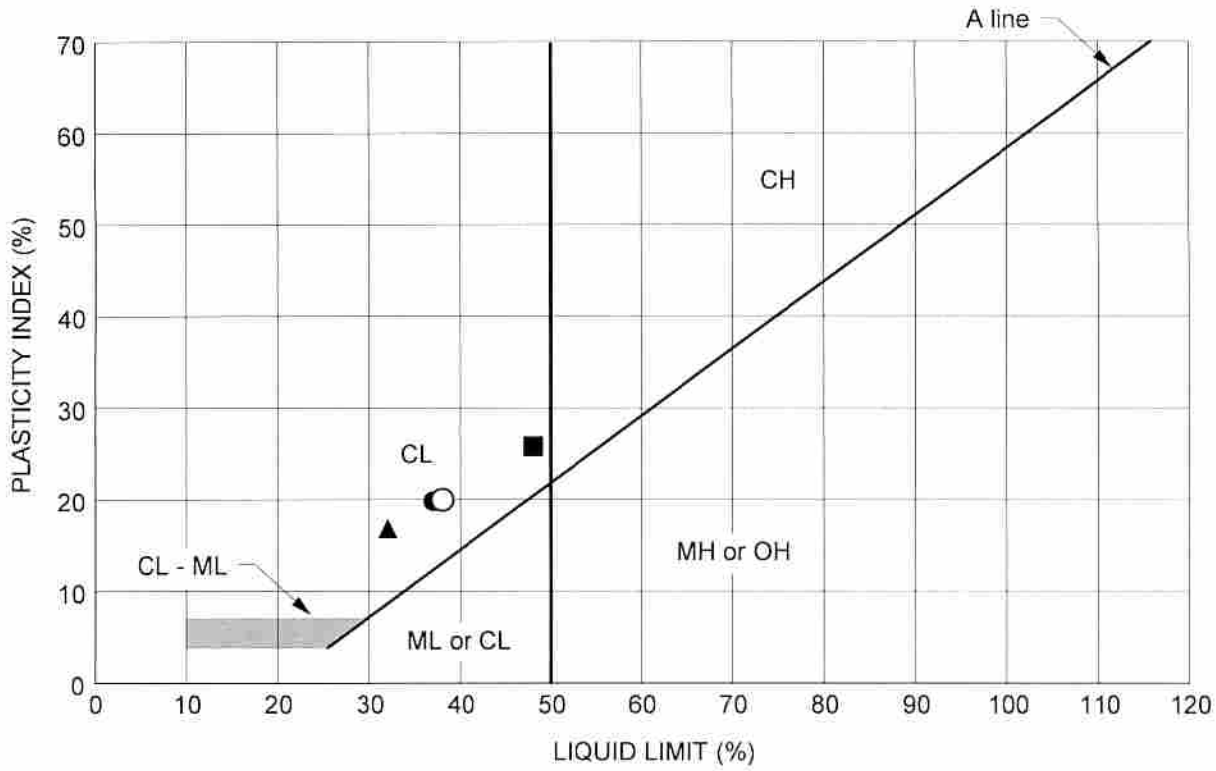
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 3 feet	CLAY with SAND (CL)	17.9	35	19	-
▲	B-1 at 23.5 feet	CLAYEY SAND (SC)	17.1	31	16	21.3
□	B-1 at 33.5 feet	CLAYEY SAND with GRAVEL (SC)	13.9	32	16	12.1

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 10/16/08 | Project No. 08-068 | Figure B-1



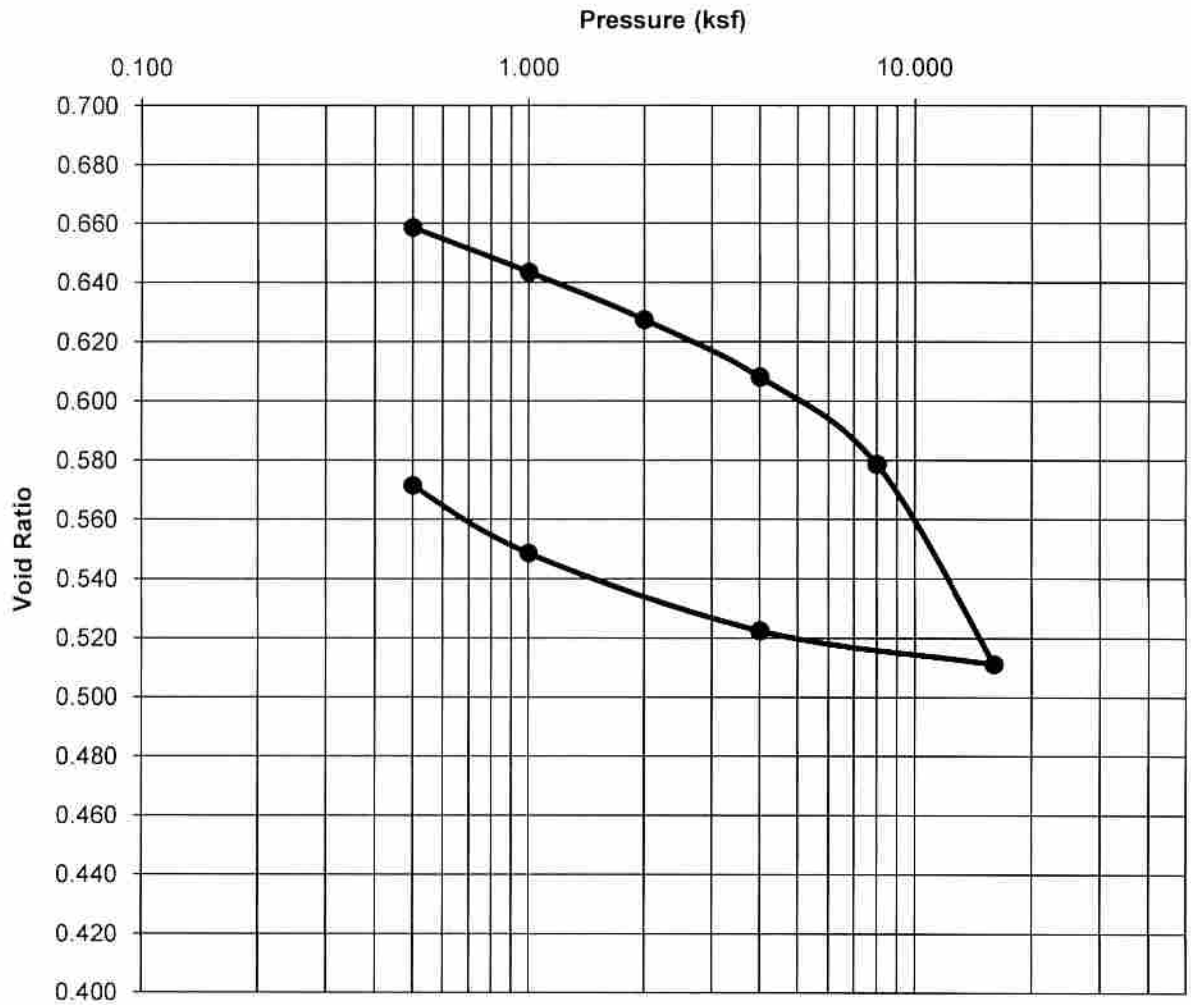
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 9.5 feet	SANDY CLAY (CL)	20.4	37	20	76.5
○	B-2 at 20 feet	SANDY CLAY (CL)	17.5	38	20	62.2
▲	B-2 at 28.5 feet	CLAYEY SAND with GRAVEL (SC)	-	32	17	18.2
■	B-3 at 3 feet	CLAY (CL)	10.8	48	26	-

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 10/16/08 | Project No. 08-068 | Figure B-2



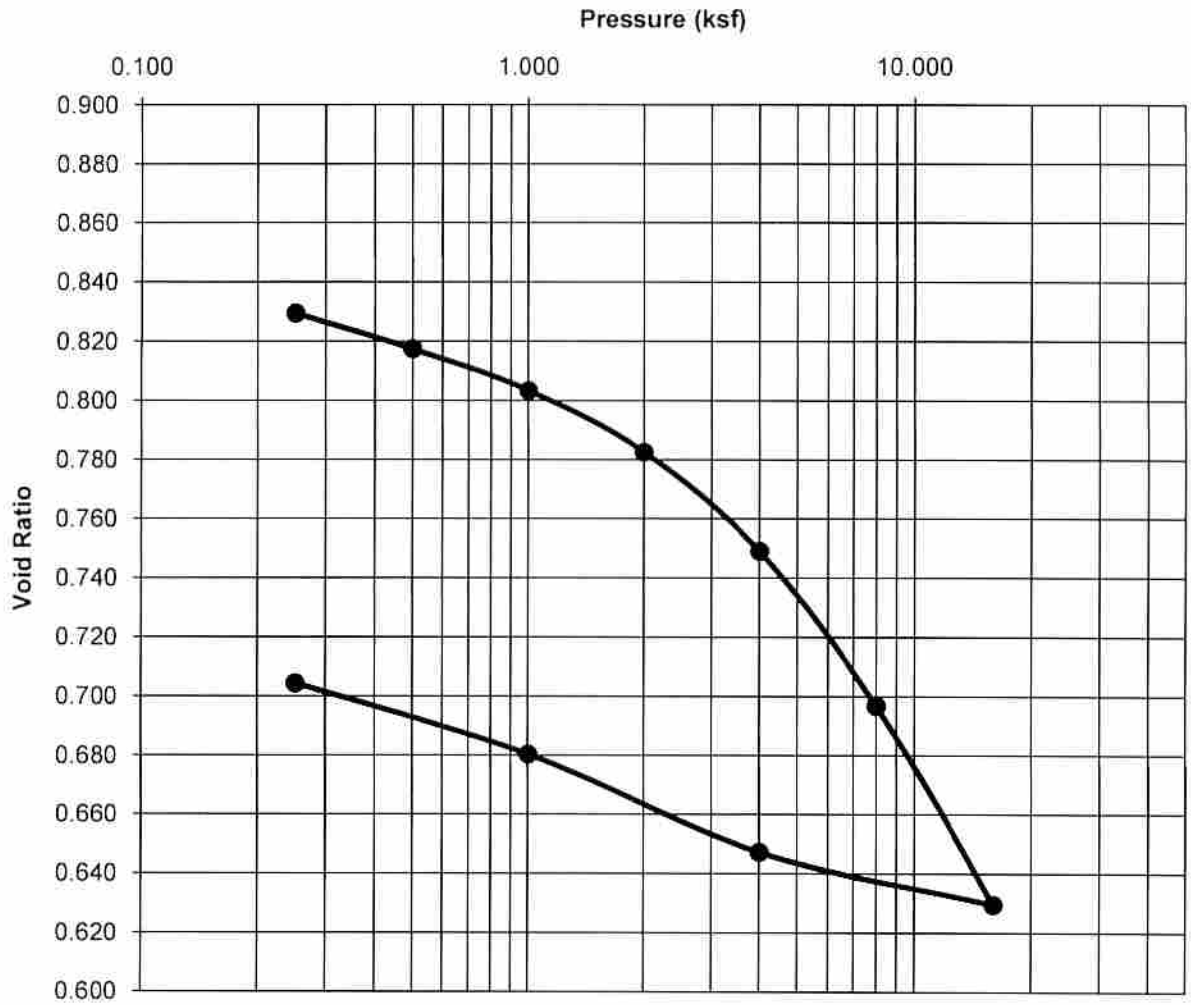
Sample Type		Sprague & Henwood			Condition	Before Test			After Test		
Diameter	2.625	Height	0.5		Water Content	w_o	22.5	%	w_f	20.7	%
Overburden		σ'_o	1,200	psf	Void Ration	e_o	0.664		e_f	0.572	
Preconsolidation		Pc	7,400	psf	Saturation	S_o	92	%	S_f	98	%
Compression Index		c_c	0.25		Dry Density	γ_{do}	101	pcf	γ_{df}	111	pcf
LL	-	PL	-	PI -		G_S	2.7		G_S	2.7	
Classification		CLAY with SAND (CL)			Source	B-1 at 10 feet					

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

CONSOLIDATION TEST REPORT

Date	10/16/08	Project No.	08-068	Figure	B-3



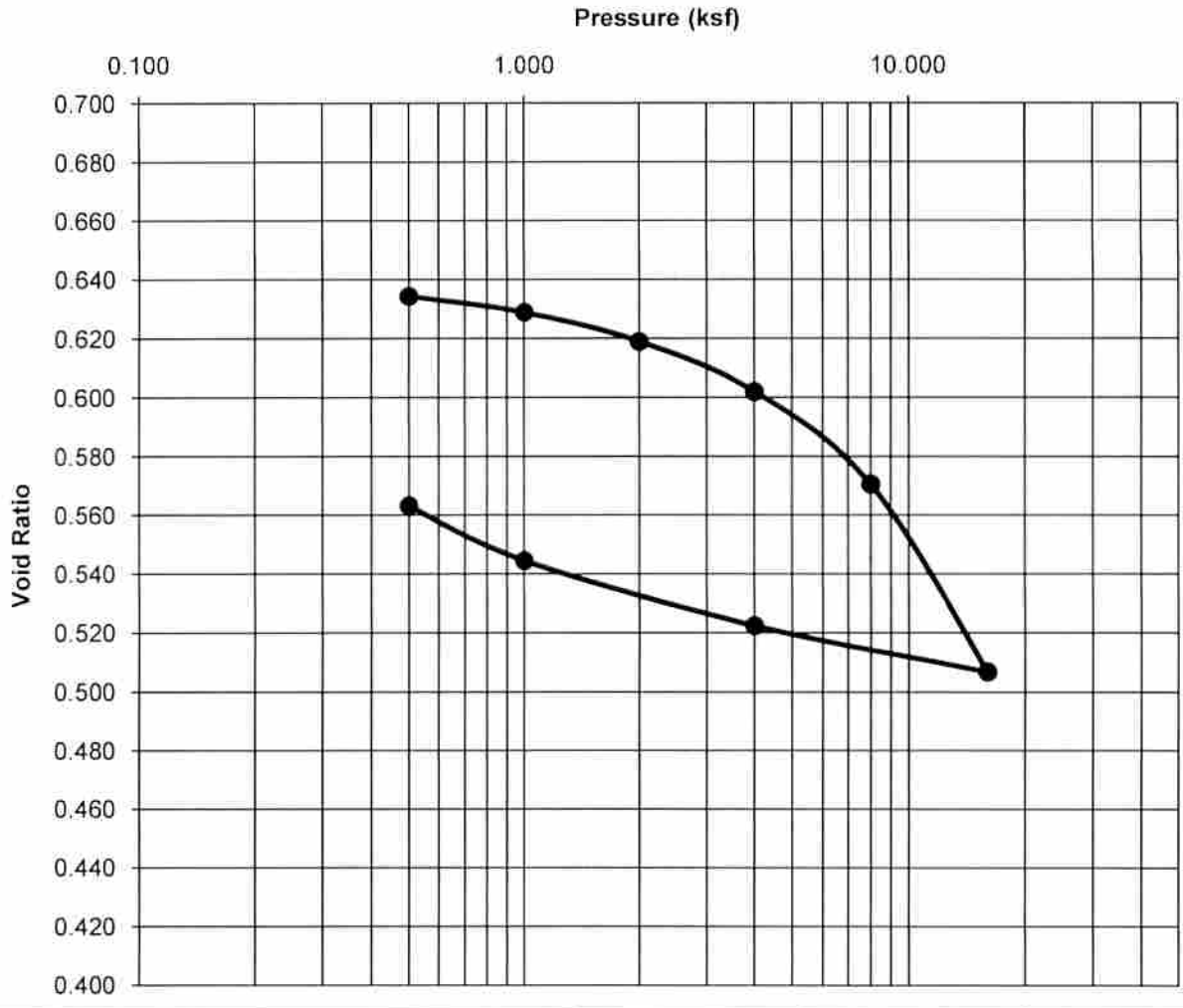
Sample Type		Sprague & Henwood			Condition	Before Test			After Test		
Diameter	2.625	Height	0.4		Water Content	w_o	28.8	%	w_f	26.6	%
Overburden		σ'_o	2,400	psf	Void Ration	e_o	0.832		e_f	0.705	
Preconsolidation		Pc	4,300	psf	Saturation	S_o	94	%	S_f	100	%
Compression Index		c_c	0.22		Dry Density	γ_{do}	92	pcf	γ_{df}	104	pcf
LL	-	PL	-	PI -		G_S	2.7		G_S	2.7	
Classification		CLAY with SAND (CL)			Source	B-1 at 20 feet					

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

CONSOLIDATION TEST REPORT

Date	10/16/08	Project No.	08-068	Figure	B-4



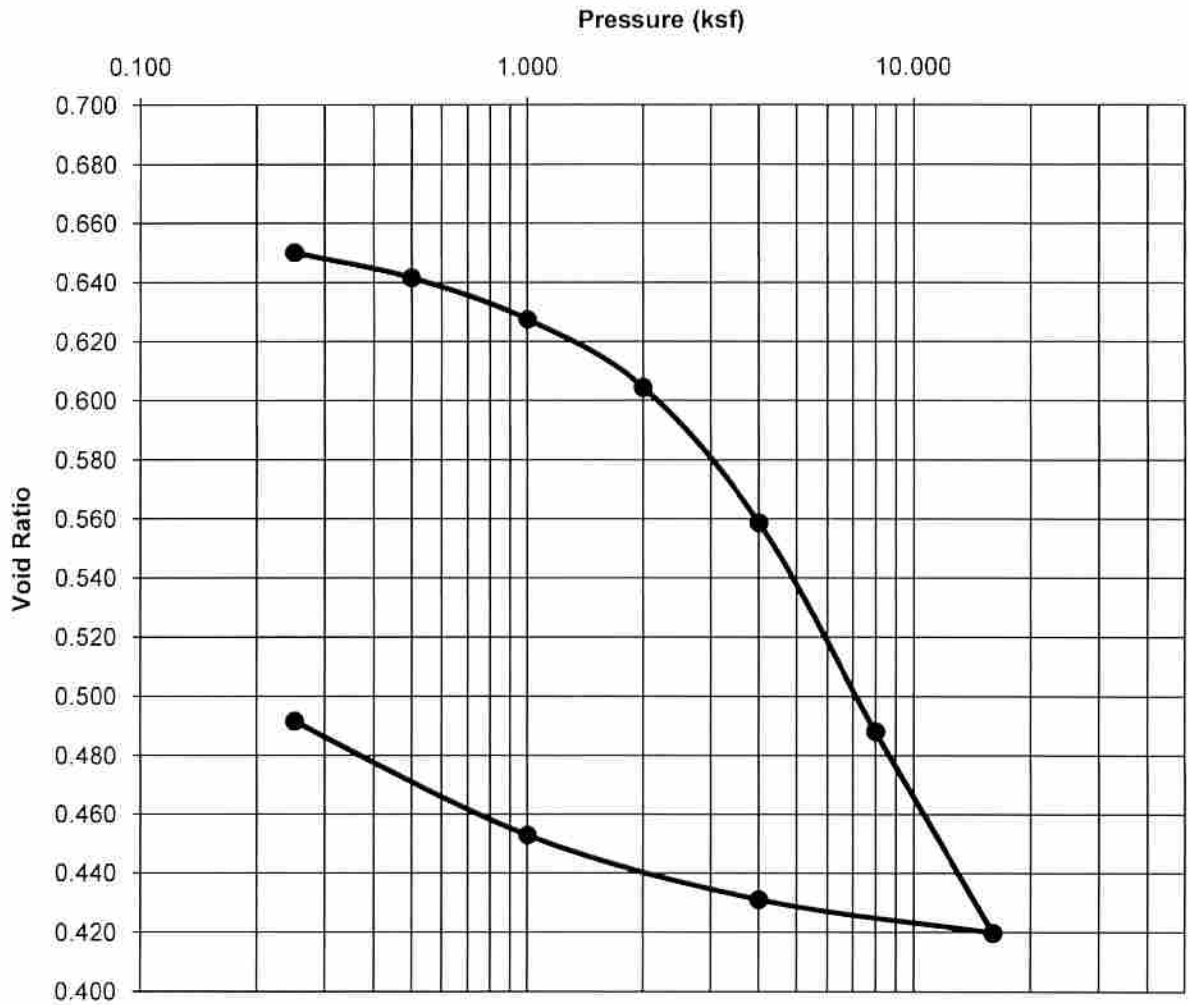
Sample Type		Sprague & Henwood			Condition	Before Test			After Test		
Diameter	2.625	Height	0.5		Water Content	w_o	20.7	%	w_f	17.9	%
Overburden		σ'_o	1,200	psf	Void Ration	e_o	0.636		e_f	0.563	
Preconsolidation		Pc	6,200	psf	Saturation	S_o	88	%	S_f	86	%
Compression Index		c_c	0.22		Dry Density	γ_{do}	103	pcf	γ_{df}	111	pcf
LL	-	PL	-	PI -		G_S	2.7		G_S	2.7	
Classification		SANDY CLAY (CL)			Source	B-2 at 10 feet					

SAN LEANDRO CROSSING
San Leandro, California

ROCKRIDGE GEOTECHNICAL

CONSOLIDATION TEST REPORT

Date	10/16/08	Project No.	08-068	Figure	B-5



Sample Type		Sprague & Henwood			Condition	Before Test			After Test		
Diameter	2.625	Height	0.5		Water Content	w_o	19.2	%	w_f	18.0	%
Overburden		σ'_o	2,500	psf	Void Ration	e_o	0.658		e_f	0.492	
Preconsolidation		Pc	2,500	psf	Saturation	S_o	79	%	S_f	99	%
Compression Index		c_c	0.23		Dry Density	γ_{do}	102	pcf	γ_{df}	119	pcf
LL	-	PL	-	PI -		G_S	2.7		G_S	2.7	
Classification		SANDY CLAY (CL)			Source	B-3 at 20 feet					

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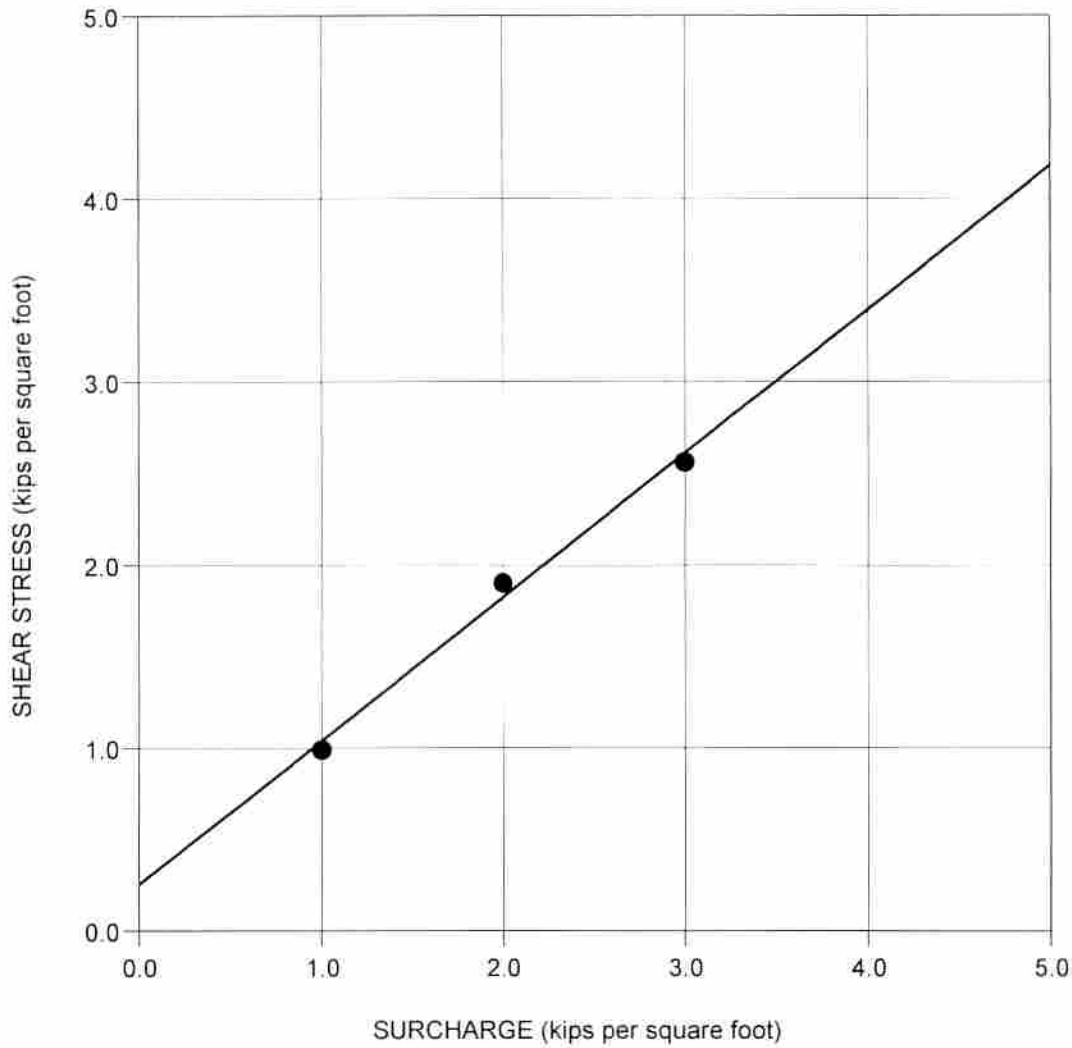
CONSOLIDATION TEST REPORT

Date	10/16/08	Project No.	08-068	Figure	B-6



DESCRIPTION OF SAMPLE SANDY CLAY (CL)			
BORING: B-2	DEPTH: 2 ft	ELEVATION:	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ) 39		APPARENT COHESION (C) 260 psf	
TEST NO:	1	2	3
VOID RATIO	0.648	0.650	0.654
PEAK SHEAR STRESS (ksf)	.999	1.913	2.570
DRY DENSITY (pcf)	103	103	103
WATER CONTENT (%)	21.4	21.5	21.0
SHEAR RATE (in/min)	.02	.02	.02

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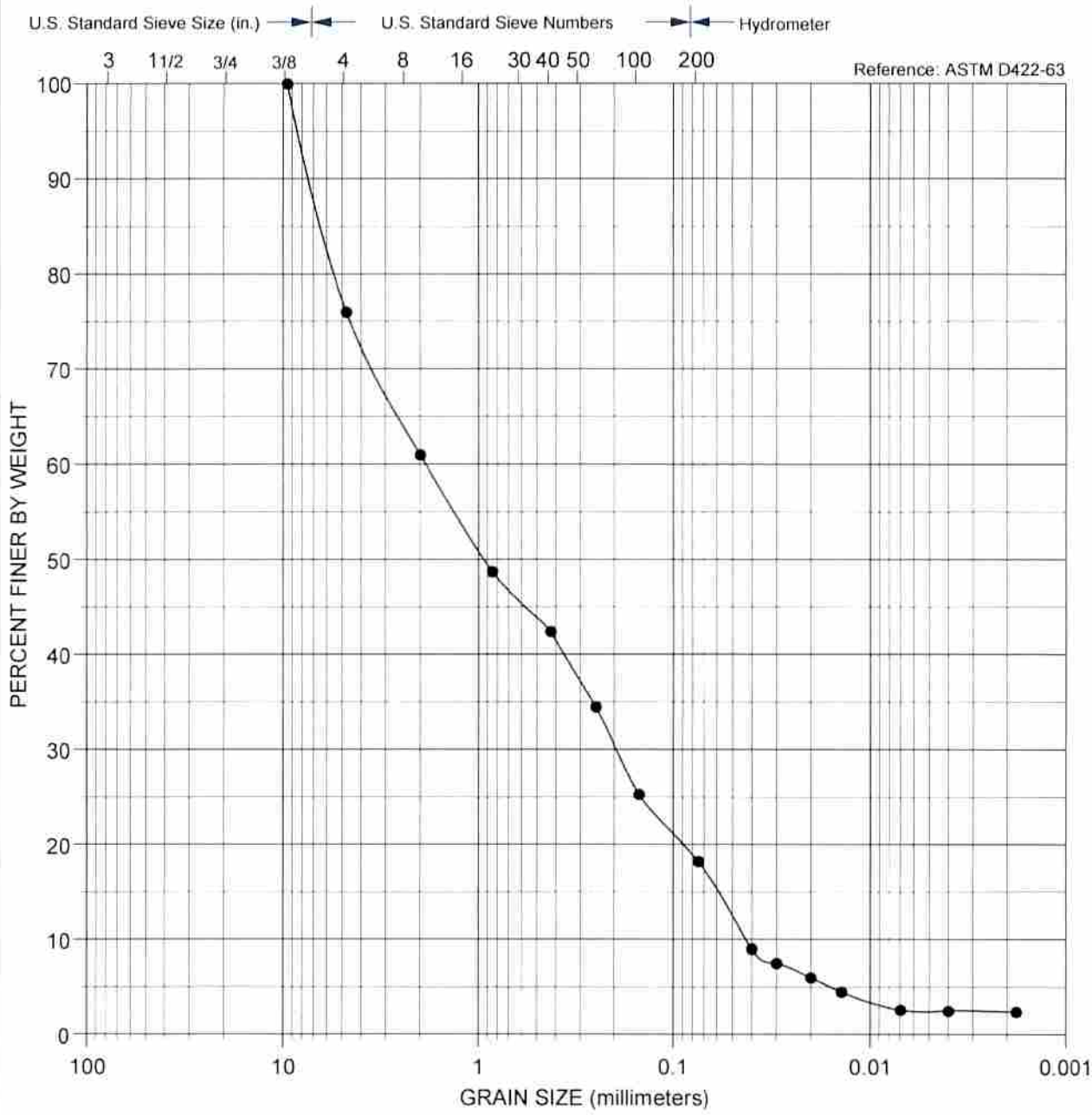
DIRECT SHEAR TEST

Date 10/16/08 | Project No. 08-068 | Figure B-7



SYMBOL	SOURCE	CLASSIFICATION
●	B-2 at 28.5 feet	CLAYEY SAND with GRAVEL (SC)

SAN LEANDRO CROSSING
San Leandro, California

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PARTICLE SIZE ANALYSIS

Date 10/16/08	Project No. 08-068	Figure B-8

California State Certified Laboratory No.2153

CERCO
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

10 September, 2008

Job No.0808206
Cust. No.12147

Mr. Craig Shields, G.E.
Rockridge Geotechnical
4319 Piedmont Avenue, Suite 204
Oakland, CA 94611

Subject: Project No.: 08-068
Project Name: San Leandro Crossing
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Shields:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on August 28, 2008. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

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

The sulfate ion concentration ranges from 16 to 39 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.6 to 8.3 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 470 to 480-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

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

Very truly yours,
CERCO ANALYTICAL, INC.

Sherri Moore
for, J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

C E R C O
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2773
www.cercoanalytical.com

Client: Rockridge Geotechnical, Inc.
Client's Project No.: 08-068
Client's Project Name: San Leandro Crossing
Date Sampled: 23-Aug-08
Date Received: 28-Aug-08
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 10-Sep-2008

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0808206-001	B-2 @ 4'	480	8.3	-	3,600	-	N.D.	16
0808206-002	B-3 @ 4.5'	470	7.6	-	1,900	-	N.D.	39

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	5-Sep-2008	9-Sep-2008	-	9-Sep-2008	-	9-Sep-2009	9-Sep-2008

Sherin Moore
for, Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

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