Treadwell&Rollo

6 July 2004
Project 3830.01

Mr. Robert Bond
Bond Companies
350 W. Hubbard Street, Suite 450
Chicago, IL 60610

Subject: Geotechnical Investigation
Cox Cadillac Site Development
Oakland, California

Dear Mr. Bond:

Enclosed are four copies of our geotechnical report, dated 6 July 2004, for the proposed development to be constructed at the Cox Cadillac site at the northeast corner of the intersection of Harrison Street and Bay Place in Oakland, California. Additional copies of this report have been transmitted to the project team members listed at the end of this report. This investigation was performed in accordance with our proposal dated 8 December 2003 and our subsequent requests for budget increase.

The site encompasses an area of approximately 2.25 acres and is bordered by Harrison Street to the northwest, Bay Place to the southwest, Vernon Street to the southeast, and residential properties to the northeast. The historic Cox Cadillac showroom building occupies the southwest corner of the site, and the remainder of the site is currently vacant. The majority of the site is relatively level and covered with asphalt concrete and concrete slabs associated with the garage the formerly occupied the site. The northern and eastern portions of the site are covered by steep vegetated slopes.

Plans are to develop the site with a new retail development that features about 56,000 square feet of ground-floor retail space on the western portion of the site and a parking garage on the eastern portion of the site. A ramp will be constructed adjacent to Vernon Street to provide access to an upper parking level that will cover the new building. The existing historic showroom building will also be renovated and seismically upgraded as part of the project.

We reviewed the results of geotechnical investigations performed at the site by Lowney and GeoForensics for previously proposed site developments. To supplement the existing information, we advanced four CPTs, drilled four borings, and observed and logged the conditions exposed in seven test pits. On the basis of our review of the existing information and our subsurface investigation, we conclude the site is generally blanketed by heterogeneous fill. Fill inside the historic building primarily consists of very loose to loose sandy clay and very soft to soft clay, and it extends to depths of at least 11 to 11-1/2 feet below the top of the floor slab. Fill beneath the former garage primarily consists of very soft to medium stiff clay extending to depths of 4 to 6 feet below the existing ground surface (bgs). The fill is underlain by weak clay on the west side of the site which appears to correspond to a historic drainage to Lake Merritt. The weak clay on the west side of the site and fill on the east side of the site are underlain by stiff to very stiff native clay with varying sand content, which generally become very stiff to hard within a few feet of the top of the layer. The site is underlain by clayey soils to the maximum depth explored (70 feet) with occasional thin clayey sand lenses. Groundwater has been measured between 0 (artesian condition) and 13 feet bgs during subsurface exploration at the site.

The primary geotechnical concern for the site are the presence of weak existing fill and native clay, shallow groundwater, and oversteepened slopes. We conclude the proposed building should be supported on a combination of conventional spread footings bearing on stiff to very stiff native clay and footings supported on compacted aggregate piers (CAPs). The proposed upgrade and seismic retrofit to the historic building should be supported on a combination of deepened footings and micropiles. Existing fill beneath the former garage should be overexcavated and recompacted to provide support for slab-on-grade floors, and the historic building should have a structural floor slab supported on micropiles.

The recommendations contained in our report are based on limited subsurface exploration and laboratory testing programs. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to perform on-site observation and testing during site grading, fill placement, and foundation installation, during which time we may make changes in our recommendations, if deemed necessary.

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

Sincerely yours,
TREADWELL & ROLLO, INC.

Hui Jun Ling, for
Andrew R. Blaisdell
Civil Engineer

Craig S. Shields
Geotechnical Engineer

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Enclosure

GEOTECHNICAL INVESTIGATION
COX CADILLAC SITE DEVELOPMENT
Oakland, California

Bond Companies
Chicago, Illinois

6 July 2004
Project No. 3830.01

Treadwell&Rollo
Environmental and Geotechnical Consultants

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GEOTECHNICAL INVESTIGATION
COX CADILLAC SITE DEVELOPMENT
Oakland, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Treadwell & Rollo, Inc. for the proposed Cox Cadillac Site development in Oakland. The site is on the northeast side of Bay Place, between Harrison Street and Vernon Street, as shown on the Site Location Map, Figure 1.

The site is currently occupied by the historic Cox Cadillac dealership building (referred to herein as the "historic building"), located in the southwest corner of the site, as shown on Figure 2. Another building that comprised the garage for the Cox dealership previously occupied approximately the western two-thirds of the site. The above-grade portion of the building was demolished during the past few months, and the floor slab and foundations are still in-place. The remainder of the site is currently covered by a combination of asphalt and concrete pavement. A steep slope with inclinations ranging from about 1:1 (horizontal to vertical) to 1-1/2:1 extends along the northern and northeastern portions of the site boundary, and the lower portion of the slope is retained by existing concrete and brick retaining walls which vary from about 7 to 20 feet in height.

2.0 PROPOSED DEVELOPMENT

Plans are to develop the site with a new retail development that features about 56,000 square feet of ground-floor retail space covering the western portion of the site, as shown on Figure 3. The historic building will be included as part of the retail space. The existing ground floor of the historic building will be removed and will be replaced with a structural slab supported on deep foundations, and the new building will have slab-on-grade floors. The floors will generally be near the elevation of the existing floor slab for the historic building, which is at approximately

Elevation 8.24 feet[1]. The floor slab in the northern portion of the building will be elevated, with elevations ranging from about 10.4 to 11.2 feet. The eastern portion of the site at ground floor level will be occupied by an asphalt-paved parking area and a ramp to access an upper parking level. Truck loading docks are to be located in the northwest corner of the building.

The upper parking level is proposed to cover the entire structure with the exception of the historic building. Portions of the parking level in the northern and northeastern corner of the building will be supported on engineered fill at the proposed second floor grade, as shown on Figure 3. The majority of the walls in the northern portion of the proposed building will be set back into and/or above the existing slope and will retain soil. Installation of temporary shoring will be required to install these walls. Additional retaining structures will also be constructed on the slope above the building walls. Other proposed improvements include concrete flatwork and/or pedestrian pavers, landscaping, and new underground utilities.

We understand Whole Foods will occupy the retail space, and they will be responsible for construction of the floor slabs to be constructed in the retail space. In addition, they require four feet of relatively unobstructed soil beneath the concrete floor slab for placement of all necessary utilities, which will also be performed by Whole Foods and/or their subcontractors.

Based on our conversations with Mr. Marc Press of KPFF Engineers, the project structural engineer, we understand dead-plus-live interior column loads for the proposed structure will range from about 150 to 390 kips. Seismic loads on shear walls for the proposed building were not available at the time this report was prepared. The historic building will be seismically retrofitted, and some new foundation loads will be transmitted through the improved building walls. New foundation elements will be installed to resist new dead-plus-live and seismic loads. The new loads at the historic building were not available at the time this report was prepared.

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[1] All elevations in this report are assumed referenced to the City of Oakland datum based on existing survey data by George Luk & Associates.

3.0 SCOPE OF SERVICES

Our scope of services was performed in accordance with our proposal dated 8 December 2003 and our subsequent requests for budget increase dated 27 January, 28 April, and 4 May 2004. We reviewed the results of two geotechnical investigations performed at the site by others, as described in the following section. Based on these results, we performed additional subsurface exploration, including advancing four cone penetration tests (CPTs), logging the conditions exposed in eight test pits, and drilling four borings. Based on the results of the investigations by others and our field investigation, laboratory testing, and engineering analyses, we developed conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- existing foundation conditions
- appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimated building settlement, including total and differential settlement
- the condition of existing fill
- methods to mitigate the potential detrimental effects of shallow groundwater
- allowable temporary and permanent slope inclinations
- temporary and permanent retaining structures
- seismic hazards, including potential for liquefaction and cyclic densification
- alternatives for mitigation of seismic hazards
- site grading and excavation, including subgrade preparation, criteria for fill quality and compaction, and chemical treatment of wet and/or expansive soil
- temporary dewatering
- asphalt and rigid concrete pavement design

- 2001 California Building Code soil profile type and near-source factors
- soil corrosivity
- construction considerations.

We understand environmental considerations predicate that existing soil should be removed from certain locations at the site. Levine-Fricke (LFR), the environmental consultant for the project, is providing guidance relating to environmental issues.

The California State Geological Survey (CGS) has prepared a map titled *State of California Seismic Hazard Zones, West Oakland Quadrangle,* dated 14 February 2003. This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990. The eastern corner of the site is within one of the designated liquefaction hazard zones. In addition, the map indicates the slopes along the northern site boundary are within a landslide hazard zone. The CGS has 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.

4.0 PREVIOUS GEOTECHNICAL INVESTIGATIONS

We have reviewed the results of the following two geotechnical investigation reports prepared by others for the site:

- *Preliminary Geotechnical Findings, Building Foundations, 230 Bay Place*, prepared by Lowney Associates (Lowney), dated 8 August 2000
- *Geotechnical Investigation for Proposed Development at Avalon Bay at Lake Merritt, 230 Bay Place, Oakland, California,* prepared by GeoForensics, Inc. (GeoForensics), dated May 2001.

The previous geotechnical investigations by Lowney and GeoForensics were performed for Avalon Bay Communities, who was considering development of the site with seven stories of residential construction. The subsurface exploration performed during the previous investigations is discussed in the following subsections.

4.1 Lowney Investigation

Lowney preliminarily investigated the subsurface conditions at the site by drilling two borings, designated as borings L-1 and L-2, using truck-mounted, rotary-wash drilling methods, and advancing four CPTs, designated as CPT-1 through CPT-4, at the approximate locations shown on Figure 2. The borings and CPTs were each advanced to a depth of about 40 feet below the existing ground surface (bgs). It should be noted that the site plan provided by Lowney does not show existing improvements and, therefore, the locations are very approximate. Logs of the Lowney borings and CPTs are attached to this report in Appendix C.

4.2 GeoForensics Investigation

GeoForensics performed additional subsurface exploration based on their review of the Lowney preliminary investigation. GeoForensics drilled nine borings, designated as GF-1 through GF-9, at the approximate locations shown on Figure 2. The borings were drilled with truck-mounted and portable Minuteman rigs equipped with solid-stem flight augers. The borings were advanced to depths of 6 to 30 feet bgs. They also observed the conditions exposed in two test pits that we believe were both excavated inside the historic building, although the locations are not shown on their site plan. Logs of the GeoForensics borings are presented in Appendix C.

5.0 SUPPLEMENTARY FIELD INVESTIGATION AND LABORATORY TESTING

To supplement the existing subsurface information and provide information regarding the foundations for the historic building, we advanced four additional CPTs, designated CPT-1A through CPT-4A, drilled four borings, designated as TR-1 through TR-4, and we observed the conditions exposed in seven test pits, designated as TP-1 through TP-7. The approximate locations of the CPTs, borings, and test pits are shown on Figure 2. Details of the geotechnical field exploration are described in the remainder of this section.

5.1 Cone Penetration Tests

The CPTs for this project were performed by John Sarmiento and Associates (JSA) of Orinda, California, on 3 February 2004. CPT-1A and CPT-2A were each advanced to a depth of about 70 feet bgs, and CPT-3A and CPT-4A were each advanced to a depth of about 50 feet bgs. Upon completion, the CPT holes were backfilled with cement-bentonite grout and patched with asphalt cold patch or concrete, as appropriate.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe into the ground. The cone tip measured tip resistance and a 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 and then processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance, friction ratio, equivalent SPT blow count, strength parameters, and soil classification type versus depth, are presented in Appendix A as Figures A-1 through A-4. The classification chart for the CPTs is shown on Figure A-5.

5.2 Borings

On 8 May 2004, borings TR-1 through TR-4 were drilled by RAM Geotechnical of Manteca, California, using a pickup truck-mounted Mobile B-24 drill rig equipped with 6.5-inch-outside-diameter, hollow-stem flight augers. The borings were advanced to explore the extent and geotechnical properties of existing fill inside the historic building. The borings were advanced to depths of 7-1/2 to 15-1/2 feet below the top of the floor slab inside the historic building. Borings TR-1 and TR-2 were terminated in stiff to very stiff native clay, and borings TR-3 and TR-4 each met practical drilling refusal at a depth of 7-1/2 feet bgs. During drilling, our field engineer logged the borings and retrieved representative samples of the soil encountered for further classification. Logs of borings TR-1 through TR-4 are presented in Appendix A on Figures A-6 through A-9, respectively. The soil was classified in accordance with the classification system presented on Figure A-10.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-spoon sampler with 3.0-inch and 2.43-inch outside and inside diameters, respectively (with brass liners)
- Standard Penetration Test (SPT) split-spoon sampler with 2.0-inch and 1.5-inch outside and inside diameters, respectively (without liners)
- Thin-walled Shelby tube (ST) with a 2.43-inch inside diameter.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the ST was used to obtain relatively undisturbed samples of cohesive soil. The ST was advanced by applying steady downward pressure from the drill rig. The S&H was used to obtain samples of all other cohesive soil, and the SPT was used to obtain samples in sandy soil. The S&H and SPT samplers were driven with a 140-pound, downhole, hydraulic wire-line safety 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 blows are shown on the boring logs. Upon completion of drilling, the borings were backfilled with neat cement grout. The soil cuttings produced were left on-site adjacent to the borings.

5.3 Test Pits

Test pits TP-1 through TP-7 were excavated on 27 and 28 April 2004 by a subcontractor retained by LF using a backhoe with a two-foot-wide bucket. Prior to excavating the test pits, the asphalt or concrete at each pit location was broken with a hoe ram. The test pits were excavated to depths ranging from about 5-1/2 to 8 feet bgs under the direction of our field engineer, who logged the soil conditions encountered in the test pits. Representative samples were collected from the test pits for laboratory testing. Upon completion, the test pits were backfilled with the excavated material; this material was tamped with the backhoe bucket and should not be considered well compacted. Test pit TP-6 was left open (not backfilled) for additional observation, as discussed in the following section. Test pits TP-2 and TP-4 were backfilled to within about two feet of previous grades to leave a portion of the historic building foundation exposed for additional observation. Our observations from the test pit excavations are summarized in Table 1. These observations include the total depth of excavation, the thickness of fill encountered (measured below the existing ground surface), and the types and sizes of debris encountered in the fill.

TABLE 1
Summary of Test Pit Excavations

Test Pit	Total Depth (feet)	Fill Thickness (feet)	Groundwater Depth (feet)	Notes
TP-1	5-1/2	3	5	Free product (classified by LF) observed in groundwater
TP-2	8	5	7	16-inch-thick subsurface brick wall extending perpendicular from building foundation
TP-3	2.5	> 2.5	N/A	Concrete slab encountered extending to brick foundation
TP-4	7	4-1/2	5-3/4	16-inch-thick subsurface brick wall extending perpendicular from building foundation
TP-5	6-1/2	2	5-3/4	
TP-6	6-1/2	1 to 1-1/2	5-1/2	Groundwater seeping in at about 3 feet bgs
TP-7	6-1/2	2	N/A	

Notes: 1) Fill thickness includes existing asphalt and/or concrete.
2) N/A indicates groundwater not encountered in test pit.

5.4 Laboratory Testing

We reexamined each soil sample obtained from our borings and test pits in the office to confirm the field classification and select representative samples for laboratory testing. Soil samples were tested to measure moisture content, Atterberg limits[2] (plasticity index), resistance value (R-value), and corrosivity. The laboratory test results are presented on the boring logs and in Appendix B on Figures B-1 and B-2. The corrosivity test results were not available at the time this draft report was prepared, and they will be included in our final report.

6.0 SITE AND SUBSURFACE CONDITIONS

6.1 Site Conditions

The approximately 2.25-acre site is bordered by Harrison Street to the northwest, Bay Place to the southwest, Vernon Street to the southeast, and six existing residential structures to the northeast, as shown on Figure 2. The historic Cox Cadillac showroom building occupies the southwestern corner of the site. The tall one-story, rectangular-shaped building has plan dimensions of about 65 by 185 feet and was constructed of brick and mortar in 1890. The reported elevation of the existing slab-on-grade floor is 8.24 feet; however, based on our field observations, the floor slab elevation appears to vary throughout the building, generally lowest in the western portion of the building and higher in the eastern portion. The results of our test pits indicate the walls of the historic building are supported on continuous brick footings. The brick footings do not appear to have pedestal-like lower portions; they appear to be uniform in thickness with depth. We observed brick footings extending perpendicularly away from the perimeter footing both inside and outside the building in locations that do not currently have above-grade walls; however, evidence of former walls was observed in some locations where this condition exists.

A garage, which we understand was constructed around 1924, formerly existed northeast of the historic building. The above-grade portion of this irregularly shaped building has been demolished by Pankow Builders, the project general contractor, leaving the concrete slab-on-

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

grade floor and foundation elements in place. The concrete floor slab slopes down towards Bay Place, with elevations ranging from about 9 to 10 feet.

Asphalt pavement covers the relatively level area between Vernon Street and the existing and former building. The ground surface slopes down gently towards the south, with ground surface elevations ranging from about 8 to 11 feet. The relatively level area northeast of the former building is covered by Portland cement concrete pavement with ground surface elevations between 10 and 11 feet.

Steep, vegetated slopes, the lower portions of which are generally retained, cover the remainder of the site. The slopes dip down towards the east and southeast. The slopes are currently covered by dense vegetation consisting of trees, shrubs, and tall grass. A shallow slope failure was reportedly observed by GeoForensics during their investigation. The slope inclinations range between about 1:1 and 1-1/2:1 between the site and the adjacent residential properties to the northeast. The ground surface elevations at the top of the slope, which is generally near or just outside the site boundary, range from about 40 to 54 feet, resulting in an elevation difference of about 30 to 45 feet between the project site and the adjacent properties.

Some portions of the existing retaining walls are constructed of bricks and mortar while others are reinforced concrete. The existing retaining walls at the toe of the slopes vary in height from about 7 to 20 feet. Some of the former building perimeter walls at the northeastern end of the former garage currently act as retaining walls, with additional lateral support being provided by interior walls and slabs. The portion of the wall that retains soil and some of the adjacent interior walls were left in-place after demolition of the remaining portion of the building to retain the slope.

The existing buildings to the northeast of the project site consist of 2- and 4-story, wood-framed residences. The buildings apparently have slab-on-grade floors near existing site grades, which range between approximately Elevation 48 and 55 feet. The buildings are generally set back about 2 to 6 feet behind the property line, and some of the concrete slabs adjacent to the buildings extend over the property line by up to a few feet.

6.2 Subsurface Conditions

Except where the historic building is present, the ground surface at the site is generally covered by asphalt pavement and/or concrete slabs. The asphalt pavement is about 3-1/2 inches thick where measured in test pit TP-5, and appears to be underlain by 4 to 6 inches of granular soil. The concrete slabs generally vary between 3 and 6 inches thick. Wire-mesh reinforcement was observed in the slab in test pit TP-6, but not in the slab exposed in TP-4 and TP-7.

Below the asphalt and concrete, the site is generally blanketed by a heterogeneous mixture of sandy and clayey fill. Considering the different conditions encountered in different borings drilled at the site, we anticipate the fill conditions may vary significantly over relatively short distances. Based on the conditions encountered in our borings and borings by others, we conclude the fill beneath the historic building consists primarily of very loose to loose sand and very soft to soft clay. The results of borings drilled by us and others in the historic building indicate the sand fill extends to depths of at least 5 to 8 feet bgs. Very soft to soft clay fill extends to depths of at least 11 to 11-1/2 feet in the western portion of the building. This area is where excavations were likely the deepest for installation of footings, indicating this may be approximately the maximum fill depth inside the building. The condition of the fill indicates it was likely loosely dumped below the water table.

A buried 1.5- to 3-foot-thick concrete slab was encountered at most locations in the eastern 1/2 to 2/3 of the historic building. The deeper slab is typically about 18 inches below the top of the existing floor slab, and it appears to consist of multiple layers in some locations. The concrete appears to be structural, with square rebar observed in some concrete cores. At one location near the northern wall of the building, an approximately 2-1/2 foot void was observed beneath the deeper slab, and water was observed in the void area. An approximately eight-inch void was observed beneath the slab at one location near the southern wall of the building. The space between the existing floor slab and the deeper slab(s) at this location is backfilled with brick, rock and concrete rubble, and sand and clay. The soil below the slabs generally consists of a mixture of uncompacted sand and clay with varying amounts of gravel and rubble. We did not penetrate the fill layer in the eastern portion of the building (TR-3 and TR-4), as refusal was encountered on an obstruction. Based on the observations of our field engineer, we believe the obstruction consisted of either concrete or large rocks or debris. The results of other borings performed in the eastern portion of the historic building do not provide conclusive evidence of the fill thickness in this area.

The fill beneath the former garage appears to consist primarily of very soft to medium stiff clay with varying amounts of sand extending to depths of 4 to 6 feet bgs, although some sand fill was reportedly observed, as described below. We performed laboratory testing on a sample of clayey fill retrieved from test pit TP-4 at a depth of 2.5 feet bgs, which is near the proposed soil subgrade elevation for the proposed building. The testing indicates the clayey fill is moderately expansive, with a plasticity index (PI) of 19. The moisture content of the soil is 29.4 percent, indicating it is likely about 14 to 18 percent over optimum moisture content at this location. We understand environmental borings advanced by LFR in the southeastern portion of the former garage encountered loose sand extending to depths of 6 to 7 feet bgs. Therefore, heterogeneous fill conditions should be anticipated. Based on the consistency of the fill encountered, we believe very little (if any) compaction was performed during placement of the fill. Buried concrete slabs of varying thickness were also encountered in the fill beneath the former garage in many locations.

The fill is thinner towards the northern and southeastern portions of the site (i.e., outside of the existing and former building footprints). The fill apparently consists of a mixture of clay and sand in these areas, and it generally appears to extend to depths of about 2 to 3 feet bgs.

The soil conditions beneath the fill vary considerably across the site. The western portion of the site is underlain by relatively weak native clay with varying sand content. The weak clay layer appears to correspond to a historic drainage into Lake Merritt and appears to be a marsh deposit.

The approximate depth to the bottom of the weak clay deposit is shown in parentheses, where encountered, on Figure 2. Based on our observations, we believe the weak clay appears to include two distinct layers; an upper gray-green layer and a lower dark gray to black layer. The results of the borings and CPTs indicate the gray-green upper clay layer is soft to medium stiff, and the dark gray to black layer is medium stiff to stiff. The weak clay is generally lightly to moderately overconsolidated under the existing overburden load. The green-gray clay appears to have low expansion potential. Laboratory testing performed on a sample of dark gray, relatively weak clay from test pit TP-6 indicates the soil is moderately expansive, with a PI of 23, and it is about 15 percent over optimum moisture content.

The weak clay layer is generally underlain by stiffer clay with varying sand content. The clay is generally stiff to very stiff immediately below the weak clay layer, and it is very stiff to hard within about 3 to 10 feet of the bottom of the weak clay layer. A sample of the stiff to hard clay retrieved during Lowney's preliminary investigation indicates some of the deeper soil at the site is highly expansive. The very stiff to hard clay layer underlies the fill beneath the eastern portion of the site, where it was encountered at a depth of two feet bgs in test pits TP-5 and TP-7. We performed laboratory testing on a representative sample of the clay retrieved near the proposed soil subgrade elevation of the proposed building, and the testing indicates the clay is moderately to highly expansive, with a PI of 26. However, considering the relatively high moisture content of this soil, we conclude the expansion potential of the clay in its current condition is low. An approximately six-inch-thick layer of silty sand was observed in test pit TP-5 at a depth of 4-1/2 feet bgs. This silty sand layer was not observed in TP-7, but other similar layers were reportedly encountered in the borings by others. The site is underlain by clayey soils to the maximum depth explored in our CPTs (70 feet), with occasional thin (less than one-foot-thick) clayey sand lenses.

6.3 Groundwater

Groundwater has been measured between depths of 0 feet (i.e., artesian condition) and 13 feet bgs in borings, CPTs, and test pits performed at the site. The locations in which artesian conditions have been encountered are generally located near the bottom of the existing slope, as shown on Figure 2. The average groundwater depths measured at the site are four feet bgs inside the historic buildings and between 5 and 6 feet bgs elsewhere at the site. The groundwater depths measured in our test pits were also generally between 5 and 6 feet bgs. These depths correspond to elevations between about 4 to 5 feet. However, in test pit TP-6, groundwater was seeping into the pit from a layer of sandy clay at a depth of about three feet bgs. Evidence of an artesian condition was not observed in the test pit at the time it was excavated, indicating the artesian conditions occurs below a depth of six feet bgs in this location. A second measurement of the water level in TP-6 about a week after the test pit was excavated showed the groundwater level had stabilized at 2.75 feet bgs, which corresponds approximately to Elevation 7.5 feet.

Groundwater was not encountered in test pit TP-7, but it was encountered seeping into TP-5 from the thin layer of silty sand encountered at that location. This indicates local groundwater elevations are likely controlled by zones of higher permeability soil (sand) in a generally low permeability (clay) matrix in areas with shallow native soil. We anticipate the groundwater depth may be more consistent in areas with deeper fill and weak soil. However, based on the available groundwater information, we believe the average groundwater elevation beneath the site is between 4 and 5 feet with the exception of the area nearest the base of the slope. The groundwater flow direction appears to be towards the southwest based on preliminary information from LFR. We anticipate the groundwater level beneath the site fluctuates about 1 to 2 feet yearly depending on seasonal conditions.

7.0 SEISMIC CONSIDERATIONS

7.1 Regional Seismicity

The major active faults in the area are the Hayward, Calaveras, San Andreas, and Concord Faults. These and other faults of the region are shown on Figure 4. For each of the active faults within 50 km of the project site, the distance from the site and estimated maximum Moment magnitude[3] (California Division of Mines and Geology 1996) event are summarized in Table 2.

TABLE 2
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
Hayward – Total	4.4	Northeast	7.1
Northern Hayward	4.4	Northeast	6.6
Southern Hayward	7.6	East	6.9
Mount Diablo Thrust	20	East	6.7
Northern Calaveras	22	East	7.0
San Andreas – 1906 Rupture	26	Southwest	7.9
San Andreas – Peninsula	26	Southwest	7.2
Concord	26	Northeast	6.5
San Andreas – North Coast South	29	West	7.5
San Gregorio North	31	West	7.3
Southern Green Valley	31	Northeast	6.5
Rodgers Creek	31	North	7.1
Northern Greenville	33	Northeast	6.6
West Napa	39	North	6.5
Great Valley – 6	39	Northeast	6.7
Central Greenville	40	East	6.7
Monte Vista	42	South	6.8
Great Valley – 5	44	Northeast	6.5

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.

Figure 4 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 5) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 430 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an M_w of 6.9. The epicenter of the earthquake was in the Santa Cruz Mountains, approximately 92 km from the site.

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

In 1999, the Working Group on California Earthquake Probabilities (WGCEP 1999) at the U.S. Geologic Survey (USGS) predicted a 70 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2030. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 3.

TABLE 3
WGCEP (1999) Estimates of 30-Year Probability (2000 to 2030) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	32
San Andreas	21
Calaveras	18
San Gregorio	10
Concord-Green Valley	6
Greenville	6
Mount Diablo	4

7.2 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.

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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.

We classified the site according to the procedures and soil profile types defined in Chapter 16 of the 2001 CBC. As mentioned previously, the site is about 4.4 kilometers from the Hayward Fault, which is a Type A fault according to the CGS publication titled *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada* (1998). We assumed a soil profile type of S_D. Therefore, we estimated the PGA would be 0.54 times gravity (g). We assumed an earthquake magnitude of 7.1, which is the maximum characteristic earthquake magnitude for the Hayward Fault.

The results of our CPTs indicate the native soil beneath the site is sufficiently cohesive (i.e., clayey) such that the liquefaction potential of the native soil is low. However, based on the results of our borings and borings by others, we conclude potentially liquefiable sand fill exists inside the historic building. At the location of borings TR-1 through TR-4, the thickness of very loose to loose sand below the water table varies between 1 and 3 feet. However, the results of environmental borings drilled by LFR and others indicate the thickness of liquefiable sand may be in excess of six feet. We estimate between 1 and 4 inches of liquefaction-induced settlement may occur following a major earthquake on one of the nearby faults. Considering the shallow groundwater table, we conclude ground damage, including ground rupture and sand boils, could also occur inside the historic building following a large earthquake. However, we believe this condition exists only in the historic building.

7.2.3 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. Considering the potentially liquefiable fill inside the historic building is contained by the perimeter wall foundations, we conclude the potential for lateral spreading occurring at the project site is low.

7.2.4 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. The surficial sandy fill layers encountered in several of the borings by others at the site are loose to medium dense. We conclude up to five feet of loose sand may exist above the water table in some locations. We estimate cyclic densification of this material in its existing condition could result in 1/2 to 1 inch of settlement during a large earthquake generating a peak ground acceleration (PGA) of 0.54 g. However, existing loose sandy fill, where it exists, will be overexcavated and/or recompacted in place during construction. Therefore, we conclude the potential for cyclic densification after site grading is complete will be low.

8.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns to be addressed during site development are:

- the presence of weak, poorly compacted fill, including potentially liquefiable fill inside the historic building
- the presence of weak native clay beneath the fill in the western portion of the site
- shallow groundwater
- steep existing slopes in the northeastern and eastern portions of the site.

These and other geotechnical issues are discussed in the remainder of this section.

8.1 Foundations and Settlement

8.1.1 Historic Cox Showroom Building

Based on the conditions exposed in test pits excavated adjacent to the historic building, we believe the building is supported on continuous brick footings bearing on firm native soil below the existing fill and weak native clay. The results of our borings and CPTs near the historic building indicate the underlying clay is stiff to very stiff. The performance of the building walls with respect to the floor slab indicate the footings have performed satisfactorily under the existing building loads. New loads will be transferred through the historic building as a result of the proposed seismic retrofit. Based on our discussion with the structural engineer and the project constraints, we conclude the most appropriate foundation to support new loads from the historic building varies based on the environmental requirements. At the eastern end of the building, soil will be excavated to a depth of about 8 feet bgs to remove contaminated materials. Since the excavation will likely extend into firm native soil, we believe the most appropriate foundation type for new loads consists of spread footings or grade beams gaining support in stiff native soil below the existing fill and marsh deposit. The deepened portion of the footings may consist of lean concrete and/or controlled-density fill (CDF) adjacent to the existing brick footing and the upper portion can consist of structural concrete. We conclude footings bearing on native soil designed using the allowable bearing capacity presented later in this report will settle 1/2 inch or less, with 1/4 inch or less of differential settlement over a horizontal distance of 25 feet.

It will be necessary to excavate the confining soil on both sides of the existing footings to construct the adjacent new footing. Accordingly, the bearing capacity of the existing footings will be reduced. Recommendations regarding reduced bearing capacities for existing footings during construction will be presented in Section 9.3.

The proposed excavation to remove contaminated soil will not to extend around the central and western portions of the historic building. The bottom of the marsh deposit is generally anticipated to be 10 to 15 feet below existing grades in this area. Because of the shallow groundwater table, continuous dewatering would likely be required to excavate through the marsh deposit to expose firm native clay that would provide suitable bearing support for spread footings, which would be a difficult and costly operation. Therefore, we conclude micropiles are the most appropriate foundation type to support new structural and seismic loads beyond the extent of the environmental excavation. The micropiles should gain frictional support in stiff to very stiff clay below the marsh deposit. Micropiles, also known as mini piles, typically consist of 6- to 8-inch-diameter drilled shafts that are filled with high-strength cement grouted under high pressure. The shafts are typically reinforced with threaded steel bars (Dywidag bars) that are installed before the grout is placed. Because the shafts are grouted under pressure, relatively high skin friction can usually be obtained between the micropiles and the surrounding soil. We estimate total settlement of structural elements supported by micropiles under static and seismic loads will be 1/2 inch or less under design loads.

8.1.2 Proposed Structure

We considered several foundation alternatives for the proposed structure, including deep foundations, a well-reinforced mat foundation, and spread footings, with and without soil reinforcement. As discussed in Section 2.0, one of the main considerations for construction of the building is that Whole Foods will construct the slab-on-grade floor and they require four feet of relatively unobstructed soil beneath the floor slab for placement of underslab utilities. This requirement affects the foundation type selected for the new building. While deep foundations such as driven piles or drilled piers would provide adequate support for the building, we believe there are other more economical foundation alternatives because strong native soil is present at relatively shallow depths over much of the site. Further, grade beams connecting piles or piers, which are required by the current building code, would complicate utility installation by Whole Foods.

We conclude conventional spread footings bearing above the weak clay layer in the western portion of the site would experience unacceptable total and differential settlements relative to the stronger soil in the eastern portion of the site and, therefore, are not appropriate. A mat foundation could likely be designed to resist differential settlement between the stronger soil underlying the eastern portion of the site and the weaker soil underlying the western portion. However, given the requirements of Whole Foods to have four feet of soil beneath a slab-on-grade for placement of underslab utilities and that a mat foundation would need to be relatively deep, construction would be costly.

We estimate the most economical method of foundation support for the building consists of spread footings bearing on native stiff to very stiff clay in the eastern portion of the site and spread footings bearing on improved soil in the western portion of the site, with a transition zone between. The depth to very stiff clay varies along the margin of the weak clay deposit. We believe significant deepening of footings would be required in some locations to reach very stiff clay throughout. Therefore, we conclude spread footings should be designed using a lower bearing capacity in a zone immediately east of the area where soil improvement below footings is required. We conclude spread footings bearing on firm native soil designed using the allowable bearing capacities presented later in this report will settle about one inch, with 3/4 inch or less of differential settlement between adjacent columns. Footings bearing in native soil should be bottomed at a sufficient depth to mitigate the potential adverse effects of expansive soil, which may be subject to volume changes (i.e., shrink and swell) during fluctuations in moisture content.

As discussed above, we conclude the soil beneath footings in the western portion of the site should be improved to reduce potential settlement to an acceptable level. One type of soil improvement that could be used is compacted aggregate piers (CAPs), which are typically constructed by drilling 10- to 20-foot-deep, 30- to 36-inch-diameter shafts and backfilling the shafts with compacted aggregate. The aggregate is compacted in 12-inch lifts using a modified hydraulic tamper attached to an excavator. During CAP installation, compaction of the aggregate fill produces high lateral stresses around the pier, increasing skin friction and densifying the surrounding soil. Additionally, a prestress zone consisting of an aggregate-filled bulb is formed beneath the bottom of the drilled shaft, which typically extends about 1 to 2 feet below the bottom of the drilled shaft. CAPs resist vertical loads through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix. The purpose of the CAPs is to reduce settlement potential and increase allowable bearing capacities (thus reducing the footing size) by strengthening the soil matrix. Compacted aggregate piers can also be designed to resist uplift loads by installing a steel plate at the base of the CAPs. Vertical steel reinforcing bars connect the plate to the footings installed above the CAP. CAPs are typically constructed through a design-build contract with a licensed foundation installer. Locally, the most common type of CAPs is a Geopier, installed by the Geopier Foundation Company of Northern California (GFCNCA).

Based on the requirements of Whole Foods, we conclude CAPs for this project should extend no higher than four feet below the top of the floor slab. Lean concrete can then be placed between the top of the CAPs and the proposed bottom-of-footing elevation, if required. We preliminarily estimate settlement of CAP-supported footings designed using the preliminary allowable bearing capacity presented later in this report will be 3/4 inch or less, with less than 1/2 inch of differential settlement between adjacent columns.

One disadvantage of the CAP foundation alternative is the potential for vibration (especially adjacent buildings) caused by the hydraulic tamping system. Considering the close proximity of proposed footings to the existing historic building, we anticipate CAPs will be installed within about five feet of the existing brick foundation. Vibration monitoring of the existing building should be performed when CAPs are being installed near the historic building to provide a record of the actual vibration levels. The structural engineer should be consulted to determine what vibration level is acceptable. In addition, CAPs should not be installed within 20 feet of any portion of the building where an excavation is open adjacent to the existing brick footing.

8.2 Slab-on-Grade Floors

Site preparation for slab-on-grade floors should address the potential detrimental effects of existing uncontrolled fill beneath the western portion of the site and expansive soil. Our conclusions regarding each of these issues are presented in the following subsections.

8.2.1 Improvement of Existing Fill

We conclude the existing fill overlying the weak clay deposit, including the fill beneath the historic showroom and the former garage, is too poorly compacted to provide uniform support for concrete floor slabs. Therefore, mitigation measures should be implemented to provide adequate support for floor slabs.

On the basis of our investigation and the investigations by others, we believe the fill beneath the former garage is generally clayey in nature and varies from about 2 to 6 feet in thickness. Additionally, the results of moisture content tests indicate the clay is currently too wet to achieve adequate compaction without significant drying. Based on our understanding of existing and proposed grades, we anticipate an average of about two feet of existing fill will be excavated to reach the soil subgrade elevation for the proposed finish floor elevation. Therefore, up to four feet of existing fill will remain over the weak clay deposit. Because the existing floor slab elevation in the vicinity of the loading docks is approximately the same as the proposed finish floor elevation, we estimate about 1 to 2 feet of existing fill will remain under the floor slab. We believe the existing fill will generally be completely removed in the eastern portion of the site (approximately in the area where soil improvement will not be required beneath foundations). In this area, the floor slab will be underlain by moderately to highly expansive native clay, which will be addressed in the following subsection.

Where up to four feet of existing fill remains below the floor slab subgrade elevation, we conclude at least 2-1/2 feet of existing fill should be overexcavated. Additional shallow test pits should be excavated after the existing floor slab is demolished to better define the existing fill thickness and the amount of overexcavation required. The lower 12 to 18 inches of fill can be compacted in place provided it is chemically treated. The base of the overexcavation should be treated with lime, cement, or a lime-cement mixture, depending on the soil encountered, and recompacted. It will be necessary to lower the moisture content of the excavated fill prior to backfilling the overexcavation to achieve adequate compaction. Typical methods used to lower the moisture content include aeration, which may require up to two weeks (depending on weather), or chemical treatment. Recommendations for both alternatives are presented later in this report.

Isolated areas of deeper poorly compacted fill likely exist in some locations. Depending on the condition of the fill encountered, it may be necessary to excavate fill below the water table. Where excavations extend below the water table, it may be necessary to place geogrid or a geotextile at the base of the excavation and place open-graded crushed rock until the fill extends above the water table.

Numerous buried concrete slabs have been encountered in the existing fill outside the historic building. We conclude the fill should be sufficiently explored during site grading to ensure all concrete slabs within four feet of the proposed finish floor elevation have been removed.

Where explored, the fill properties inside the historic building are very poor. The fill consists of very loose to loose sand and very soft to soft clay in most locations. Therefore, we believe the fill inside the building was generally loosely dumped into place. The large settlement of the existing floor slab appears consistent with loosely placed fill. Based on our observations in test pits and our borings, we believe the existing soil was excavated to a depth of at least 11-1/2 feet below the top of the existing floor slab (bts) to construct the existing brick footings. It is not clear whether the bottom of the excavation was sloped up towards the middle of the historic building, which would result in a large differential fill thickness below the slab, or the entire building was excavated to a relatively uniform depth.

Based on the conditions encountered, we conclude all of the existing fill would require removal and replacement with engineered fill if the new floor slab was to be a slab-on-grade floor. This is anticipated to be a very difficult and complicated operation considering all of the constraints, and based on our discussion with the project team, we understand the potential for unknown conditions to affect construction of this option makes it undesirable.

We conclude the floor slab for the historic building should be structurally supported on micropiles that gain support in stiff to very stiff clay below the marsh deposit and fill. Based on our discussion with KPFF, we understand this could consist of either a slab supported on micropile-supported grade beams or a thicker slab supported directly on more closely-spaced micropiles. It will be necessary to suspend utilities from the structurally supported floor and use flexible connections where they extend through grade beams because of the potential for additional settlement of the uncompacted fill beneath the floor to occur. We conclude the upper five feet of soil and/or slabs and debris beneath the existing floor should be re-worked to provide suitable engineered fill for installation of utilities as required by Whole Foods.

8.2.2 Expansive Subgrade Soil

The concrete slab-on-grade will be underlain by moderately to highly expansive soil where native clay is exposed at subgrade elevation. Volume changes in expansive soil can cause cracking of the floor slab. Potential adverse impacts of the expansive soil can be mitigated by moisture conditioning the expansive soil beneath the slab and providing a layer of select, non-expansive fill beneath the floor slab. Select fill may consist of imported fill or on-site sandy soil meeting the specifications presented later in this report or on-site native clay treated with five percent lime by dry weight.

8.3 Subsurface Drainage

As discussed previously, we conclude the average groundwater elevation at the site is between 4 and 5 feet except in the northern portion of the site, where it appears to be higher. The groundwater gradient appears to be towards the southwest. We conclude the most appropriate method to reduce the potential for near-surface seepage to adversely impact the proposed improvements is to install subdrains extending around the northern and portions of the eastern and western sides of the proposed building. Because the historic building is completely enclosed by deep brick footings, we conclude the potential for groundwater to be present shallower than four feet bgs is low, and special measures are not required. Specific recommendations regarding location and depth of the subdrains are presented later in this report.

We believe a subdrain system designed and installed according to the recommendations presented later in this report should intercept most of the shallow groundwater that may impact the proposed structure. However, the potential exists the subdrain may not lower the groundwater level sufficiently in some portions of the building pad. This should be further evaluated by excavating shallow holes extending about three feet below soil subgrade for the new slab prior to slab construction. If these excavations indicate groundwater is within two feet of the soil subgrade elevation in some areas, we will provide recommendations for a supplemental subdrain system in such areas.

8.4 Temporary Retaining Structures

Construction of the proposed building will require excavation back into the existing slopes along the northern portion of the site. Excavation to depths up to 30 feet below existing grades will be required along the perimeter of the building to construct the proposed walls. In addition, it will be necessary to place engineered fill upslope of the proposed building in some areas to flatten the existing slopes and reduce the potential for future slope instability.

Based on our review of cross-sections prepared by Pankow (general contractor) and reviewed with KPFF, we believe the most appropriate methods of shoring for the project consist of cantilever soldier beam and lagging walls and soil-nail walls. Soil nails will generally be used for deep excavations, and cantilever soldier beams walls will primarily be utilized for shallow excavations in close proximity to property lines. Where tiebacks or soil nails are to be used, permission must be obtained to install the soil nails or tiebacks beneath adjacent properties, including adjacent residences, City of Oakland streets and/or adjacent utility easements, as appropriate. The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. Some seepage through the sides of the excavation may be encountered. If this is the case, soil nails may not be appropriate. Before final design of the shoring system is performed, we should drill at least one boring to the bottom depth of the proposed excavation to investigate whether seepage and/or clean sand will be a concern. Special consideration should be given to prevent erosion of the excavation and/or piping of soil through the face of the shoring. Recommendations for design of the shoring systems, including seepage considerations, are provided in Section 9.6.

8.5 Construction Dewatering

The current development plans do not call for improvements to extend below the groundwater table. However, temporary dewatering will be required during construction when excavations extend below Elevation 4 feet. Considering the soil conditions beneath the site, we believe a passive system, in which water is collected from a low point/points in an excavation using trench drains, will be more appropriate than a system utilizing dewatering wells. The water collected should be discharged into a controlled drainage facility. The need for and extent of a passive dewatering system should be determined by the contractor during construction.

8.6 Construction Considerations

As previously discussed, the existing fill that is to be overexcavated and recompacted will need to be aerated or chemically treated so it can be re-used as engineered fill. Chemical treatment will be required to stabilize the base of the overexcavation, but it may be feasible to significantly reduce the amount of chemical treatment of excavated soil by initiating construction during the dry season. This assumes sufficient staging area is available to spread the existing fill to dry. Grading should not be performed between December and April for this project if the soil is to be aerated. If the project proceeds during the rainy season, it is likely the soil will need to be treated to dry it sufficiently to be used as engineered fill.

Based on the condition of the existing fill and the potential for subsurface obstructions, we anticipate significant delays could be encountered if CAP installation is performed before the existing fill is overexcavated and recompacted as recommended in this report. Therefore, we believe construction should be scheduled to perform all overexcavation and recompaction prior to installation of CAPs in the western portion of the site.

Excavation for CAPs will produce a significant amount of spoils, and we anticipate it may be desirable to use the spoils as engineered fill in other areas of the site. We anticipate most of the spoils will not be able to be reused as fill without chemical treatment to dry and improve the compaction characteristics of the soil. If it is necessary to use the deeper spoils as fill, it should be planned to treat this material. Otherwise, the wet spoils should be offhauled from the site.

As discussed previously, Whole Foods requires four feet of relatively unobstructed fill below the floor slab for installation of utilities. Beneath most of the proposed retail area, the upper approximately 2 to 3 feet of soil will consist of recompacted engineered fill. However, beneath the northern corner of the site, we anticipate only the upper 18 inches of soil below subgrade elevation will consist of chemically treated soil, which will be underlain by moderately to highly expansive weak clay. We do not believe it will be feasible to recompact this material as utility trench backfill without chemically treating the soil, which will likely not be possible at the time utilities are placed. Therefore, some offhaul of existing soil and replacement with imported fill will be required when Whole Foods installs underslab utilities. The amount of offhaul cannot be estimated until underslab utility plans are developed, but we anticipate the most significant offhaul will occur in the northernmost corner of the site.

We anticipate groundwater may be present in footing excavations in locations where it is necessary to deepen the footings to bear on firm soil as well as where footings are deepened to bear on CAPs. Groundwater should not be allowed to sit in footing excavations for extended periods of time, as it will cause the soil to soften prior to placement of concrete. We conclude footing excavations where groundwater is encountered should be backfilled with lean concrete or sand-cement slurry as soon as possible after excavation to reduce the potential for softening of the footing excavation bottom. If the footing subgrade soil softens, it will be necessary to overexcavate and remove the softened material.

9.0 RECOMMENDATIONS

In accordance with the scope of work, the following subsections present our recommendations regarding demolition, site preparation, foundation support, subsurface drainage, slab-on-grade floors, temporary and permanent slopes, dewatering, temporary and permanent retaining structures, asphalt and concrete pavements, seismic design, and concrete flatwork.

9.1 Site Demolition, Grading, and Fill Placement

9.1.1 Demolition

Site demolition should include the removal of existing building elements and foundations, pavements, utility lines, and other below-grade improvements, if any, that will interfere with the proposed construction. The foundations and other below-grade remnants associated with the former garage should be removed to a depth of at least four feet below final subgrade elevation in accordance with the requirements of Whole Foods. Where below-grade elements are located beneath proposed foundations, it may be feasible to alter the location of CAPs slightly to avoid removal of existing improvement that are deeper than four feet below final subgrade. This should be reviewed on a case-by-case basis during demolition. Excavations resulting from demolition activities should be backfilled according to the recommendations provided later in this section.

We recommend all existing concrete slabs, brick footings, and other deleterious materials be removed to a depth of at least five feet below final subgrade elevation in the historic showroom to provide relatively unobstructed soil as required by Whole Foods beneath the structurally supported floor.

Where utilities to be removed extend off site, they should be capped or plugged with concrete at the property line. It may be feasible to abandon utilities in-place, provided they will not impact future utilities or building foundations. If pipelines are abandoned in-place, they should be completely filled with flowable cement grout over their entire length. All existing utility lines encountered should be addressed on a case-by-case basis.

9.1.2 Site Grading

Following demolition, existing fill should be overexcavated and recompacted to provide a uniform surface for support of slab-on-grade floors. Beneath the former garage, we believe this will consist of overexcavation of up to three feet of existing fill below the proposed floor slab subgrade elevation. Overexcavation may be limited to leave up to 18 inches of existing fill in place provided the base of the overexcavation is treated with 4 to 5 percent lime, cement, or a lime-cement mixture by dry weight of soil to a depth of 18 inches. If the existing fill thickness beneath the proposed finish floor elevation is 18 inches or less, overexcavation is not required, and treatment of fill can consist of chemically treating the fill that remains. The appropriate type and exact percentage of the admixture used for stabilization should be determined during construction based on the soil type and moisture content of the soil to be treated. Detailed recommendations regarding chemical treatment are provided in the following subsection.

In the historic building, overexcavation should extend to a depth of at least five feet below the floor slab subgrade elevation. We recommend a layer of geogrid (Tensar BX1200 or equivalent) be placed five feet below the proposed slab, 12 inches of crushed rock be placed over the geogrid and a second layer of geogrid be placed over the crushed rock. The geogrid layers will reduce the potential for damage to the slab if sand boils form following an earthquake, and they will also create a relatively stable surface over the weak soil below for placement of four feet of engineered fill. We recommend differential soil excavation on either side of the existing brick footing be limited to a depth of three feet to reduce the soil retaining requirements of the footing.

Once a firm base has been created at the base of the overexcavation, the excavation should be backfilled. Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned and compacted to at least 90 percent relative compaction. Considering the high moisture content of the existing fill beneath the garage, we believe the moisture content of the soil will need to be lowered before proper compaction can be achieved. One method to lower the moisture content is aeration to naturally dry the soil. Aeration typically requires at least a week of warm, dry weather to effectively dry a thin lift of the material. Material to be dried by aeration should be turned at least twice a day promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be placed and compacted in accordance with our previous recommendations. Aeration typically is the least costly method to reduce the moisture content of the soil; however, it generally requires the most time to complete and requires a large staging area to spread the material for mixing and turning. In addition, aeration is typically only feasible during warm weather. Alternatively, the material to be placed as fill can be treated with 4 to 5 percent lime, cement, or a lime-cement mixture to reduce the moisture content. Chemically treated soil can typically be placed within 24 hours of treatment, which will likely allow for a faster grading operation.

In some locations, excavations will likely extend below the groundwater table. This includes the excavation required for removal of contaminated soil near the southeastern corner of the historic building and other excavations required to remove existing deep slabs and structural elements. It may not be cost effective to dewater such excavations to allow for placement and compaction of engineered fill, and the bottoms of excavations below the water table are likely to be relatively soft. Excavations below the water table should be backfilled with open-graded rock placed on firm, undisturbed native material. Crushed rock should be used to fill the excavation to a level that allows placement of engineered fill, which should be about 6 to 12 inches above the groundwater elevation. If more than 12 inches of crushed rock is placed, the rock should be mechanically tamped in 12-inch-thick lifts during placement. Recycled concrete is acceptable for this use, provided no more than six percent of the recycled concrete passes the 3/8-inch sieve. A layer of geotextile filter fabric (Mirafi 140N or equivalent) should be placed over the crushed rock, and then engineered fill should be placed in lifts not exceeding eight inches in loose thickness and compacted to at least 90 percent relative compaction.

Soil excavated during site grading should be suitable for reuse as fill or backfill provided it contains no debris, organic material, or rocks greater than four inches in greatest dimension. Where imported fill is required, it should also be free of rocks or lumps larger than four inches or other deleterious or hazardous material, and should have a liquid limit less than 40 and a plasticity index (PI) less than 12. The Geotechnical Engineer should approve all sources of engineered fill at least three days before use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not provided, up to two weeks may be required to perform any required analytical testing on proposed import soil.

The upper 12 inches of soil beneath the building pad should consist of non-expansive (select) fill; the select fill does not need to extend beyond the building footprint or beneath the garage asphalt pavement. Select fill should consist of either on-site or imported fill meeting the requirements provided in the preceding paragraph or existing on-site soil treated in accordance with the recommendations presented in the following subsection.

All fill and backfill should be placed in lifts not exceeding eight inches in loose thickness. The existing clay fill and native clay are typically moderately to highly expansive. These soil types should be moisture-conditioned to at least three percent above optimum moisture content and compacted to at least 90 percent relative compaction[7]. Sandy soils, including silty and clayey sand and lean sandy clays, should be moisture-conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction (95 percent relative compaction if sand with less than 10 percent fines[8] is used). Fill five feet thick or greater should be compacted to at least 95 percent relative compaction, except highly expansive clay, which should be compacted in accordance with our previous recommendations.

Positive surface drainage should be provided around the building to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

9.1.3 Chemical Treatment

Chemical treatment is typically performed by specialty contractors using specialized blending and mixing equipment. The soil can be treated to a maximum depth of 18 inches and treated and compacted in one lift, assuming specialized compaction equipment is used. If conventional compaction equipment is used, lifts should not exceed eight inches in loose thickness. The soil

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7 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-00 laboratory compaction procedure.

8 Silt and/or clay particles passing the No. 200 sieve.

should be scarified and thoroughly broken up to full depth and width. The soil to be treated should not contain rocks or soil clods larger than two inches in greatest dimension or other debris, as these will prevent proper mixing and may damage the blending equipment. All organic material, including any peat or other organics encountered in the weak clay layer, should be removed, as they may retard the soil-lime reaction. The soil should be mechanically shaped and sized for the addition of the selected chemical. Treated soil should be compacted to at least 90 percent relative compaction except in the upper six inches of the pavement subgrade, where at least 95 percent relative compaction should be achieved.

If lime is used, it should conform to the requirements in ASTM Designation C977; the percentage of free lime as calcium hydroxide in the applied lime mixture should be determined by California Test 414. The lime quality, spreading, mixing, compacting, and curing should comply with Caltrans Standard Specification Section 24.

Where low plasticity sandy clay or clayey sand fill are to be treated, the soil may be treated with 4 to 5 percent of a lime-cement mixture or Portland cement by dry weight of soil. If chemical treatment of mainly granular soil (sands or gravels) is required, the most appropriate treatment will most likely consist of 4 to 5 percent Portland cement by dry weight of soil. Where cohesive, moderately to highly expansive clay is to be treated, we recommend the soil be treated with five percent quicklime by dry weight of the soil to be treated. An appropriate assumed unit dry weight of the soil to be treated should be determined by our field engineer based on the soil type.

Where cement treatment is used, it is essential to achieve proper compaction immediately after blending because the cement will begin to hydrate and harden immediately after mixing. During blending, the soil should be moisture-conditioned to remain above optimum moisture content. The cement-treated soil should be compacted immediately after blending with proper compaction equipment, and testing should follow immediately to ensure adequate compaction has been achieved.

Where lime treatment is used, the lime should be spread over the soil and uniformly mixed into the soil. During blending, the soil should be moisture-conditioned to remain at least two percent above optimum moisture content. The blended soil should be allowed to mellow overnight, and it should be re-blended the following day, while maintaining the moisture content at least two percent above optimum moisture content. Following the second blending, the lime-treated soil should be compacted with specialized compaction equipment to at least 90 percent relative compaction and maintained at least two percent above optimum moisture content. For soil treated at the base of an excavation, we recommend construction equipment not be allowed on the pad for at least two days following compaction to allow for proper set-up and curing.

Existing fill that is excavated and chemically treated to reduce its moisture content should be treated using a "table mixing" method. "Table mixing" consists of excavating the soil, spreading it in a layer of uniform thickness of 18 inches or less, treating it in the open area, and then placing it in the excavation.

During the curing period for lime-treated soil, the surface of the treated material should be kept moist. Use of chemically treated soil in landscape areas should be avoided because the high pH and the high sulfate content of the soil may restrict or prevent proper plant growth. Chemically treated spoils can be reused as backfill in utility trenches, beneath slabs, and behind retaining walls.

9.1.4 Utility Trenches

Bedding for utility trenches should extend at least D/4 (with D equal to the outside pipe diameter) below the bottom of the pipe as a minimum. However, the bedding should be at least four inches thick. The soil excavated from the trenches can be reused to backfill the trenches, provided the material can be compacted to the required compaction specification. Untreated clay with a PI greater than 15 should not be used in the upper 18 inches of the trench. Trench backfill should be compacted in accordance with the recommendations presented above for general site fill. Jetting and flooding of trench backfill should not be allowed. If crushed rock, rod mill, or pea gravel is used, it should be mechanically tamped in 12-inch-thick lifts. In accordance with City standards, the upper three feet of utility trenches in City of Oakland streets should be backfilled to at least 95 percent relative compaction. Special care should be taken when backfilling trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Utilities inside the historic building should be hung (supported) from the structurally supported floor slab. To avoid overstressing these utilities and/or hangars during an earthquake, uncompacted granular soil, such as sand or pea gravel, should be placed in the utility trenches. Clay should not be used as backfill over hung pipes.

9.2 Foundation Support

We conclude the proposed building and the seismic retrofit of the historic building can be supported on a combination of conventional spread footings, deepened footings, compacted aggregate piers, and micropiles. Recommendations for each foundation system are presented in the following subsections.

9.2.1 Spread Footings

We recommend perimeter footings be bottomed at least 30 inches below the lowest adjacent soil subgrade, and interior footings be bottomed at least 24 inches below the lowest adjacent soil subgrade. Isolated and continuous footings should be at least 24 and 18 inches wide, respectively. Continuous and spread footings bearing on very stiff native clay or medium dense clayey sand should be designed using allowable bearing pressures of 5,500 and 6,000 pounds per square foot (psf), respectively, for dead-plus-live loads. Continuous and spread footings that bear on stiff native clay within the transition zone between the weak clay deposit and the stronger native clay should be designed using allowable bearing pressures of 4,000 and 4,500 psf, respectively, for dead-plus-live loads. The allowable bearing capacities presented above may be increased by one-third for total loads, including wind and seismic. The zone in which reduced bearing capacities should be used for design is shown on Figure 3. These values include factors of safety against bearing capacity failure of at least 2.0 and 1.5 for dead-plus-live loads and total loads, respectively. All footings should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of adjacent utility trenches. Where footings are adjacent to the perimeter subdrain, they should be deepened to bear at or below the bottom of the subdrain, as necessary.

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of footings and friction along the base of the footings. Passive resistance should be calculated using an equivalent fluid weight (triangular distribution) of 300 pounds per cubic foot (pcf); 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 0.35. These values include a factor of safety of 1.5.

Momentary uplift of footings can be resisted by the weight of the footings and soil above them. The weight of the overburden should be computed using a density of 125 pounds per cubic foot (pcf) multiplied by the volume of soil above the footing.

If additional uplift resistance is required for footings, soil anchors (tiedowns) or micropiles may be used. We can provide recommendations for tiedowns, if required. Alternatively, CAPs can be installed beneath columns where uplift resistance is required. Recommendations for CAP-supported footings are presented in the following subsection.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. We should check foundation excavations after cleaning but prior to placement of reinforcing steel to confirm the excavations are bottomed in suitable bearing material and have been cleaned properly. The bottoms and sides of footings should be maintained in a moist condition until concrete is placed.

9.2.2 Deepened Footings Below Soft Soils

We recommend deepened footings constructed adjacent to the existing brick footing for the seismic retrofit of the historic building bottom at the same elevation as the existing footings or on stiff native clay, whichever is shallower. Deepened footings bearing on stiff native clay should be designed using an allowable bearing pressure of 3,000 psf for dead-plus-live loads. The allowable bearing capacity presented above may be increased by one-third for total loads, including wind and seismic. These values include factors of safety against bearing capacity failure of at least 2.0 and 1.5 for dead-plus-live loads and total loads, respectively. The lower portion of deepened footing excavations can be backfilled with lean concrete or CDF with a 28-day compressive strength of at least 100 psi. The structural engineer should be consulted to determine whether lean concrete as specified above has sufficient strength for the intended use.

Recommendations regarding passive and frictional resistance and footing cleanout are the same as those presented in Section 9.2.1. Dewatering should be maintained to keep deepened footing excavations free of water until lean concrete is poured.

9.2.3 CAP-Supported Footings

We recommend footings in the area on Figure 3 where soil improvement is specified be supported on CAPs. CAPs should also be used to support footings located within areas of deep fill, including environmental excavations, if any. Because the CAPs are designed and installed under a design-build contract, we cannot provide specific recommendations regarding spacing, costs, vertical capacity, or estimated settlement. As a minimum, the CAPs should extend through the recompacted fill and weak clay deposit and bottom in very stiff to hard clay. CAPs should be installed such that the top of the pier is at least four feet below the finish floor elevation to allow Whole Foods clear space for installation of utilities. We estimate the CAPs will be about 6 to 18 feet long. We anticipate footings supported on the CAPs can be designed for allowable dead-plus-live load bearing capacities on the order of 4,500 to 5,000 psf with a one-third increase for total loads, including wind and seismic. Frictional resistance for CAP-supported footings can typically be calculated using a friction coefficient of 0.4, and passive resistance should be calculated according to the recommendations presented for footings. Actual values for bearing pressures and friction should be given by the designer.

The design capacity of the CAPs should be verified by at least one load test in compression and one test in tension, if uplift elements are used. Geopiers are the most common type of CAP used in the Bay Area; however, other types of CAPs may be appropriate for use at the site as well. We should review the design prior to construction.

In general, the clayey material encountered over most of the site should stand vertically in a drilled shaft. However, since the CAPs will primarily be installed below the groundwater table, they may be prone to caving. Therefore, the contractor should be prepared to case the holes if caving soils are encountered.

It will be necessary to install CAPs within a few feet of the existing brick footing that supports the historic building. Vibration monitoring should be implemented during installation of CAPs within 10 feet of the historic building.

9.2.4 Micropiles

We conclude micropiles should be used to support the new structural elements and the structurally supported floor for the historic building. The micropiles should be designed to gain support through skin friction between the shafts and the stiff to very stiff clay that underlies the weak fill and marsh deposit. For planning purposes, we recommend it be assumed that micropiles installed in the western and eastern halves of the building will not gain frictional support in the upper 10 and 15 feet of soil, respectively, to account for the weak fill and marsh deposits. The micropiles should be spaced at least four shaft diameters or four feet apart, center-to-center, whichever is greater. Industry publications list a wide range for transfer loads for different soil types. These transfer loads are dependent on site-specific soil characteristics, groundwater conditions, and the contractor's method(s) and skill of installation. Therefore, it is not possible to give specific recommendations for micropile capacities. For planning purposes, we recommend ultimate transfer loads of 5.5 and 7.5 kips per foot be used for estimating purposes for 6- and 8-inch-diameter, pressure-grouted micropiles, respectively. Because the micropiles will be used to support dead-plus-live loads, this ultimate transfer load should be reduced by a factor of safety of at least 2.0 when determining the allowable transfer load. The specialty contractor should determine the actual transfer load (which may be less), and may use higher values provided they are verified by a load-test program.

The micropiles will be installed through weak fill below the water table; therefore, the contractor should be prepared to case the holes if caving soil is encountered. Poor construction techniques (i.e., allowing the drilled holes to fill with water for several days) may result in softening of the sides of shaft and low micropile capacities. If water is present in the shaft, concrete should be placed using a tremie system. Because the micropiles will be permanent, we recommend they be double corrosion protected.

The required micropile bond length should be confirmed by a performance- and proof-test program conducted under the observation of an engineer from Treadwell & Rollo. The total number of anchors required is not known at this time. We recommend 10 percent of the micropiles be performance-tested and 20 percent of the remaining piles be proof-tested to 200 percent of the design dead-plus-live load or 150 percent of the seismic load, whichever is greater. Because we anticipate the micropiles will be used to support compressive and uplift loads, we recommend one-third be tested in compression and two-thirds in tension.

During testing, the deflection of each micropile should be monitored with a free-standing, tripod-mounted dial gauge accurate to at least 0.001 inches. We recommend deflection of the micropiles be measured at the following load increments, expressed as a percentage of the design load (DL), during the performance test, in sequence: 5, 25, 50, 5, 25, 50, 75, 5, 25, 50, 75, 100, 5, 25, 50, 75, 100, 125, 5, 25, 50, 75, 100, 150, 175, 190, and 200 percent of the design load, where five percent of the design load is the recommended alignment load (AL). Dial gauges shall be zeroed at the first setting of AL. The load should be held at each increment just long enough to obtain movement reading. Except for the reading of the residual movement at AL, no movement reading needs to be taken during unloading of the pile. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is less than 0.04 inch during the loading, the test shall be discontinued. If the difference is more than 0.04 inch, the holding period shall be extended to 60 minutes, and the movements shall be recorded at 15, 20, 25, 30, 45, and 60 minutes.

For acceptance of a tested pile, the pile shall sustain the compression and tension design loads (100 percent DL) with no more than the computed elastic deflection of the reinforcing bar plus 1/4 inch total vertical movement at the top of the pile as measured relative to the top of the pile prior to the start of testing; however, the total movement of the micropile should be less than 3/4 inch at both the compression and tension design loads. If an AL is used, the allowable movement will be reduced by multiplying by a factor of (DL-AL)/DL. The creep rate at the end of the 200 percent DL increment should be 0.04 inches per log cycle of time or less from 1 to 10 minutes or 0.08 inches per log cycle time from 6 to 60 minutes and has a linear or decreasing creep rate.

For the 200-percent proof test, the deflection should be measured at the following load increments, expressed as a percentage of the DL, in sequence: 5, 25, 50, 75, 100, 125, 150, 175, and 200. Each load should be held for a minimum of one minute and the final load for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test can be discontinued. If the difference is more than 0.04 inch, the load should be held for an additional 50 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes. The proof test results should be compared to the performance test results. Any significant variation from the performance test results will require performance testing on the anchor. The acceptance criteria for proof testing are the same as that described for performance testing.

Replacement anchors should be provided, as directed by the structural engineer, for anchors that fail the test. After testing, the anchors should be loaded to 10 percent of the design load (higher if specified by the structural engineer) and locked off.

9.3 Temporary Bearing Capacity of Existing Brick Footings

We recommend an ultimate bearing capacity of 6,500 psf be assigned to the existing brick footings bearing on native stiff clay after the existing overburden soil has been removed. We believe the most efficient construction method of footing improvements may consist of exposing the lower portion of the footings over a large area at one time to allow the footing improvements to be performed in the minimal number of operations. If this is the case, we recommend a minimum construction factor of safety against bearing capacity failure of 1.5 be used and, accordingly, the existing brick footings should be assigned an allowable bearing capacity of 4,330 psf. If the allowable bearing capacity presented above is too low for the existing loads, the lower portion of the footings should be exposed using a staged excavation approach, with no more than 20 lineal feet of the existing footing exposed in an excavation.

9.4 Subsurface Drainage

We recommend a subdrain system be installed around the perimeter of the proposed building to intercept shallow groundwater flow towards the building. The recommended location of the subdrain system is shown on Figure 3. The subdrain should be constructed in a 12-inch-wide trench excavated immediately inside of the proposed building footprint. Geotextile filter fabric (Mirafi 140N or equivalent) should be placed on the bottom and sides of the trench, and approximately four inches of 3/4-inch free-draining crushed rock or Class 2 permeable material should be placed in the bottom of the trench. A four-inch-diameter, Schedule 40 or SDR 23.5 perforated PVC pipe should be placed at the center of the trench over the bottom four inches of permeable material with the perforations facing down. Open-graded crushed rock or Class 2 permeable material should then be placed in 12-inch lifts and mechanically tamped. The top of the subdrain should be at least six inches below the soil subgrade elevation, and the filter fabric should be wrapped around the top of the permeable material to ensure it is fully protected from migration of fine-grained soil. The top of the trench should be capped with a concrete slab or lean concrete to prevent surface water from entering the drain. A detail for the subdrain system is shown on Figure 6.

We recommend the bottom of the subdrain system be at least 3.5 feet below the proposed finish floor elevation. However, the subdrain elevation should extend no deeper than necessary to reduce the potential for the bottom to be located below the static groundwater elevation. We have developed recommended subdrain elevations (City of Oakland datum) assuming a minimum slope of 0.5 percent to provide a suitable gradient to maintain drainage of the subdrain system, which are shown on Figure 3. The pipes should be connected to solid PVC collector pipes which should be properly sized to collect the required volume of water. Where two subdrain elevations are shown on Figure 3 (Gridline D.5/4.3), we recommend the deeper pipe be connected to a collector pipe and a shallower pipe be started above to minimize the required depth of the subdrain system. The collector pipes should be discharged to a controlled drainage facility.

Cleanouts should be provided to ensure the subdrain system can be cleared if it becomes clogged. We recommend one cleanout be installed at each 90-degree bend in the drain, which corresponds to the locations where elevations are shown on Figure 3.

9.5 Concrete Floor Slabs

The slab-on-grade floor should be underlain by at least 12 inches of nonexpansive, select fill or lime-treated existing fill or native clay compacted to at least 90 percent relative compaction. Because it is not planned to excavate all of the existing fill beneath the historic building, the new floor slab should be structurally supported. If the subgrade surface is disturbed during construction, it should be rerolled to provide a smooth, uniform subgrade surface prior to placement of the capillary moisture break described below.

To reduce water vapor transmission through the floor slab (in occupied areas), 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 capillary moisture break should not be considered part of the select fill layer. Because of the relatively shallow water table, we recommend the vapor retarder meet the requirements for Class B 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 4.

TABLE 4
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90-100
¾ inch	30-100
½ inch	5–25
3/8 inch	0-6
Sand	
No. 4	100
No. 200	0-5

The sand overlying the membrane should be moist at the time concrete is placed; however, it should contain no free water. 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.

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.

9.6 Temporary Slopes

Temporary sloping or shoring will be required to maintain cuts higher than five feet, including utility trench excavations. The safety of workers and equipment in or near excavations is the responsibility of the contractor. The contractor should be familiar with the most recent OSHA Trench and Excavation Safety standards. We should review plans for temporary sloping prior to construction. During construction, we should observe cut slopes to verify the inclinations are appropriate for the soil conditions encountered. Where temporary slopes are excavated in competent native cohesive soil, we recommend the inclination of temporary slopes over four feet high not exceed 3/4:1. Where temporary slopes are excavated in granular soil such as sand or gravel or in weak, uncontrolled fill, we recommend the inclination of temporary slopes over four feet in height not exceed 1.5:1.

9.7 Permanent Slopes

As discussed previously, the existing native slopes on the north side of the site are generally stable at slope inclinations of about 1.5:1 with the exception of minor surface sloughing.

Therefore, we recommend native slopes be maintained at a maximum slope angle of 1.5:1. We anticipate slopes steeper than 2:1 will be particularly susceptible to erosion. Therefore, landscaping in areas with slope inclinations between 2:1 and 1.5:1 should consist of aggressive soil-fixing ground cover that is highly resistant to erosion. Even with such ground cover, erosion should be anticipated during heavy rainfall. Permanent fill slopes should be graded to a maximum slope inclination of 2:1.

Permanent native slopes covered by the proposed building should also be excavated to a maximum slope angle of 1.5:1. These slopes should be covered with a three-inch layer of shotcrete.

Prior to placement of fill on slopes, all vegetation and topsoil containing greater than four percent organics should be removed. In addition, any existing loose soil, shallow slumps, or slope failures should be excavated. The surface to receive fill should be scarified to a depth of six inches, moisture conditioned to above optimum moisture content, and compacted to at least 90 percent compaction. Since slope fill will be placed against retaining walls, benching is not required. Fill placement should proceed in accordance with the recommendations presented in Section 9.1.

To protect against slope erosion, we recommend concrete-lined drainage ditches be placed at the top of all slopes higher than 10 feet. The drainage ditches should flow into closed pipes leading to suitable discharge facilities. The drainage ditches will require periodic cleaning of any debris or soil. Completed slopes should be planted with soil-fixing ground cover to further limit erosion. If slopes are completed in November or later, it may be necessary to cover the slopes with erosion control material until the vegetation is established.

9.8 Temporary Earth Retaining Systems

We conclude it will be necessary to support the excavation north of the proposed building using a combination of soil-nail shoring systems and soldier pile and lagging shoring systems. Both shoring systems are discussed in the following subsections. If clean, saturated sand or significant seepage is encountered, it may not be feasible to install soil nails because the soil may not stand vertically. We should perform additional investigation prior to final design to determine whether the conditions are more appropriate for soil nails or soldier piles and lagging.

The anticipated deflection of the shoring system to be used should be estimated to check if it is acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement of the temporary shoring and/or movement of adjacent improvements. The shoring system should be designed by and experienced shoring designer. The shoring designer should be responsible for determining the type and size of shoring members required to resist the pressures presented herein. However, we should review the shoring design prior to bidding of the documents for construction.

All shoring systems should be installed by an experienced shoring specialty contractor. The contractor should be familiar with applicable local, state, and federal regulations for temporary shoring, including the current OSHA Excavation and Trench Safety Standards. We recommend a representative from our office observe the installation of the temporary shoring system as part of our "Special Inspection" services.

9.8.1 Cantilevered Soldier Piles and Lagging

Based on our review of proposed cross sections, we anticipate cantilevered soldier pile walls will generally retain either relatively level or oversteepened native slopes. The cantilevered soldier pile and lagging system should be designed using an active equivalent fluid weight of 35 pcf where the ground surface slopes up behind the shoring system at an inclination of 3:1 or less. Where the slope behind the wall is 2:1 or steeper, an equivalent fluid weight of 60 pcf should be used for shoring design. For intermediate slope inclinations, the design equivalent fluid weight can be determined by interpolating between these values. These pressures should be assumed to act over the entire width of the lagging installed above the base of the excavation; the pressures need only be assumed to act over one pile width below the bottom of the excavation.

The lateral earth pressures presented above were developed assuming the groundwater level will be at least three feet below the bottom of the excavation during construction, or will be allowed to flow through the lagging to be collected by a passive dewatering system. Where zones of seepage are encountered, filter fabric should be placed behind the lagging and all voids filled with sand-cement slurry. The lateral pressure to be resisted by the lagging will depend on the size of the soldier piles and the spacing between them. If traffic is expected within 10 feet horizontally from the face of shoring, a uniform surcharge load of 100 psf acting on the upper 10 feet should be used in the design. If buildings are situated within a horizontal distance equal to the wall height from the top of the wall, a uniform surcharge load of 100 psf acting on the entire wall should be used in the design.

Passive resistance for soldier piles should be calculated using an equivalent fluid weight of 300 pcf. For soldier piles spaced greater than three times the soldier pile diameter, the passive pressure should be assumed to act over three pile diameters provided structural concrete is used to backfill the portion of the soldier pile holes below the excavation. The calculated embedment depth for the shoring systems should be increased by at least 20 percent to obtain the design embedment depth. A factor of safety of 1.5 has been applied to the passive resistance values presented above.

9.8.2 Soil Nails

Soil-nailing is a method of shoring using grouted reinforcing bars (nails), which are typically spaced, horizontally and vertically, between 4 and 6 feet. Construction of a soil-nail wall involves 1) excavation, 2) installation of nails, and 3) construction of facing. The excavation proceeds in lifts that are generally 4 to 6 feet deep, depending upon the ability of the soil or rock to stand temporarily unsupported. In each excavation step, a row of nails, usually 3/4- to 1-1/2-inch-diameter steel bars, are placed in predrilled holes and grouted. We anticipate the length of the soil nails would be approximately equal to the height of the proposed excavation. After the installation of a row of nails and placement of drainage panels, the soil/rock surface is covered by facing, typically 4- to 6-inch-thick shotcrete reinforced with wire mesh. On the following day, the nails are bolted to steel plates, which are typically 8-inch square. The above steps are repeated to the bottom of the excavation.

Because localized seepage may be encountered near the bottom of the excavation, drainage panels should be installed to reduce the potential for weeping of groundwater through the wall. We recommend drainage panels cover at least half of the total soil-nail wall area. The drainage panels should outlet to PVC collector pipes that are connected to the storm drain system. Continuous drainage panels overlain by waterproofing elements should be placed on the outside of the shotcrete for the soil-nail wall to further reduce the potential for seepage through the wall where a permanent building wall will be adjacent to the soil-nail wall. The continuous drainage panels should drain to a perforated PVC collector pipe. The pipe should be surrounded on all sides by at least four inches of 3/4-inch crushed rock wrapped in filter fabric or Caltrans Class 2 permeable material. Alternatively, AdvanEDGE pipe (or equivalent) may be used in lieu of the PVC pipe surrounded by gravel.

The ultimate soil-nail friction will depend upon the installation methods and workmanship of the specialty contractor who performs the work. The allowable soil-nail friction should be computed by dividing the ultimate friction by a factor of safety of at least 2.0. For planning purposes, we recommend the soil-nail wall be designed using values of 35 degrees and 0 psf for the angle of internal friction (ϕ) and cohesion intercept (c), respectively. The total unit weight of the soil should be taken as 130 pcf. The soil-nail wall should be designed with a minimum factor of safety of 1.5 against global slope stability failure. Wire mesh and shotcrete should be applied to the exposed soil/rock face within 24 hours of excavation.

An increase in lateral design pressure for the shoring will be required where additional retaining structures or slopes will be located above the walls. The increase in pressure should be determined after the grading plan has been developed and surcharge loads are known.

The computed nail length should be confirmed by a proof-testing program under our observation or the observation of an engineer experienced in this type of work. Two verification tests should be performed prior to the start of production soil nailing. The verification tests should be performed on "sacrificial" nails and should be loaded to pullout failure to verify the ultimate soil-nail friction developed. If a factor of safety of less than 2.0 is determined from the verification tests, the allowable soil-nail friction should be reduced accordingly. The first two production nails and two percent of the remaining nails should be proof tested to 1.5 times the design load. If any nails fail to meet the proof-testing requirements, additional nails should be added to compensate for the deficiency, as required by the shoring designer.

9.9 Retaining Wall Design

The parking garage walls and northern building walls will retain up to 25 feet of engineered fill and/or native soil. We believe the two conditions behind the retaining walls will consist of relatively level backslope conditions or steep slopes with inclinations of 2:1 or greater. For static conditions and retained soil with a maximum slope inclination of 3:1, we recommend the walls be designed as restrained (no movement at the top of the wall) with level backfill conditions using an at-rest equivalent fluid weight of 60 pcf. For seismic conditions, permanent walls should be designed for the greater of at-rest pressures or an active equivalent fluid weight of 40 pcf plus a seismic pressure increment. The seismic pressure increment should consist of a uniform pressure of 12H in psf, where H is the height of the wall in feet.

Where the retained slope inclination is greater than 2:1, we recommend the walls be designed as restrained using an at-rest equivalent fluid weight of 75 pcf. For seismic conditions, these walls should be designed for the greater of at-rest pressures or an active equivalent fluid weight of 50 pcf plus the seismic pressure increment recommended previously for level backfill conditions. If intermediate final slope angles will be used above retaining walls, we will provide alternate design recommendations upon request.

Where an unrestrained portion of the wall extends above the building diaphragm, the wall can be designed for active conditions using an equivalent fluid weight of 45 pcf for static conditions. A uniform seismic pressure increment of 12H should be added for seismic conditions.

Where there will be vehicular traffic within a horizontal distance of 10 feet from the back of retaining walls, we recommend a vehicle surcharge be included in the design. The vehicle surcharge should consist of uniform pressure (rectangular distribution) of 100 psf applied over the upper 10 feet of the wall.

Where there is sufficient space behind the retaining wall, we conclude the earth pressure acting on the wall can be reduced by placing layers of geogrid in the backfill. We recommend retaining walls backfilled with geogrid-reinforced soil be designed using an equivalent fluid weight of 10 pcf. We recommend uniaxial geogrid (Tensar UX1400MSE or equivalent) be installed with the strong axis perpendicular to the wall. At junctions between adjacent pieces of geogrid normal to the wall, the adjacent pieces should be connected with hog ties to provide continuous layers throughout the entire reinforced soil mass. The geogrid should be spaced every two feet vertically and should be placed in continuous horizontal layers. The geogrid length should be at least equal to 70 percent of the wall height (measured above the adjacent ground surface below), with an additional four feet of geogrid at the back of the wall to wrap around the soil adjacent to the back of the wall. Therefore, for a 20-foot-high wall, each length of geogrid would be 18 feet long.

The foregoing design pressures assume that all walls will be properly backdrained. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the backside of the wall. We should review the manufacturer's specifications for proposed prefabricated drainage panel material to verify it is appropriate for the intended use. The drainage panels should extend down to a perforated PVC collector pipe that is connected to the storm drain system. The pipe should be surrounded on all sides by at least four inches of 3/4-inch crushed rock wrapped in filter fabric or Caltrans Class 2 permeable material. Alternatively, AdvanEDGE pipe (or equivalent) may be used in lieu of the PVC pipe surrounded by gravel.

Where wall backfill is required, it should meet the requirements presented in Section 9.1 for on-site or imported fill using light compaction equipment. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced. The lateral earth pressures presented above assume the backfill material will have low expansion potential, as expansive soil can exert large lateral earth pressures on adjacent structures. Therefore, highly expansive soil should either be chemically treated prior to use as backfill or not used as wall backfill. Wall backfill with a total thickness greater than five feet should be compacted to at least 95 percent compaction. Even with good compaction control, we believe 20 feet of wall backfill will settle between 1 and 1-1/2 inches over a period of several years after placement. Structural improvements and floor slabs over the wall backfill should be appropriately designed to withstand the expected settlement.

The retaining walls should be supported on footings designed in accordance with the recommendations presented in Section 9.2.1.

9.10 Pavement Design

9.10.1 Flexible Pavement Design

The State of California flexible pavement design method was used to develop the recommended flexible pavement sections for the asphalt-paved parking area. We expect the final soil subgrade will consist of native clay. The R-value test performed on a sample of clay soil collected from test pit TP-5 indicates the R-value of the soil is 7. We used this value for design.

For our calculations, we assumed a Traffic Index (TI) of 4.5 for automobile parking areas with occasional trucks and 5.5 for areas subjected to occasional garbage or delivery truck traffic. These TIs should be confirmed by the project civil engineer and/or Whole Foods, as appropriate. Table 5 presents our flexible pavement section recommendations.

TABLE 5
Recommended Pavement Sections

TI	Asphaltic Concrete (inches)[9]	Class 2 Aggregate Base R = 78 (inches)[10]
4.5	2.5	9
5.5	3	12

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas and the entire thickness of Class 2 aggregate base should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction.

9.10.2 Rigid Pavement Design

If rigid pavements are required, we recommend they be designed for 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 seven inches of Portland cement concrete over six inches of Class 2 aggregate base. The pavement section should rest on at least six inches of fill or native clay compacted to at least 95 percent relative compaction.

The compressive strength of the Portland cement concrete should be at least 3,000 psi at 28 days. Contraction joints should be constructed at 15-foot spacings. Where the outer edge of the rigid pavement meets asphalt pavement, the slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. We recommend concrete pavement be reinforced with a minimum of

---

9 Minimum asphalt thickness shall be 2.5 inches
10 Minimum aggregate base thickness shall be 6 inches

No. 4 bars at 16 inches on center in both directions. Recommendations for subgrade preparation and aggregate base compaction for rigid pavement are the same as those we have described for flexible pavement.

9.10.3 Interlocking Concrete Pavers

We anticipate interlocking, precast concrete pavers could by used for this project, particularly in entry and parking areas. Where pavers will receive vehicular traffic, we recommend they be at least 3.15 inches (80 millimeters) thick and placed on a 1- to 2-inch-thick sand leveling course. The aggregate base thickness given above in Table 5 should also be used beneath the pavers and sand leveling course. The subgrade and aggregate base beneath the pavers should be compacted in accordance with the recommendations previously provided for asphalt concrete pavements.

9.10.4 Concrete Flatwork and Pedestrian Pavers

In areas to receive concrete sidewalks or other flatwork, the native soil subgrade should be scarified, moisture-conditioned as appropriate (see Section 9.1.2), and compacted to at least 90 percent relative compaction. On-site sidewalks or concrete flatwork should be underlain by at least four inches of Class 2 aggregate base. The aggregate base should be moisture-conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction. City of Oakland sidewalks should also be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction in accordance with the City of Oakland standard specifications and details.

9.11 Seismic Design

The proposed buildings should be designed to the appropriate seismic codes. The project site is about 4.4 kilometers from the Hayward Fault, defined as a Type A fault per the 2001 California

Building Code (CBC). For design in accordance with the 2001 CBC seismic code, we recommend the following:

- Seismic Zone Factor 4
- Soil Profile S_D
- Near Source Factors N_a and N_v of 1.26 and 1.68, respectively.

9.12 Soil Corrosivity

Representative samples of native soil from the weak clay deposit (TP-6 at 3 feet) and the stronger clay (TP-7 at 5.5 feet) were sent to Environmental Technical Services of Petaluma, California for corrosivity testing. The samples were chosen to be near the anticipated underground utility elevation. The analytical test results are presented in Appendix D.

Based on the results of the tests, the weak clay from TP-6 is considered to be corrosive to buried iron, steel, Cast Iron, Ductile Iron, and galvanized steel pipe resulting primarily from a combination of a low resistivity and anticipated wet near-surface soils. The stronger clay from TP-7 is not considered to be corrosive to metallic utilities. The sulfate and chloride ion concentrations are considered insufficient in either sample to damage reinforced concrete structures and cement mortar coatings.

We recommend all buried metal utilities and steel located in the weak clay deposit be designed assuming they will be in contact with corrosive soil. The use of non-corrosive materials or corrosion inhibitors, such as cathodic protection or polyethylene protective film, should be considered for metallic underground utilities in contact with the weak clay deposit or similar fill. In addition, steel reinforcement bars used in CAPs should either be protected against corrosion or sized to allow for corrosion loss. The CAP design-build contractor should provide corrosion protection recommendations based on the results presented for the sample from TP-6 in Appendix D. Corrosion inhibitors are not required for metallic underground utilities in contact with the stronger native soil or similar fill. We should review the location of any underground metal utilities to determine which corrosion protection recommendations are appropriate. We conclude special protection is not warranted for concrete in contact with the ground or reinforcing steel embedded in concrete.

10.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

We should review the final project plans and specifications to check that they are in general conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during demolition, site preparation, placement and compaction of fill and backfill, and installation of building foundations and floor slabs. These observations will allow us to compare actual with anticipated soil conditions and check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

11.0 LIMITATIONS

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

REFERENCES

California Division of Mines and Geology (1997), Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.

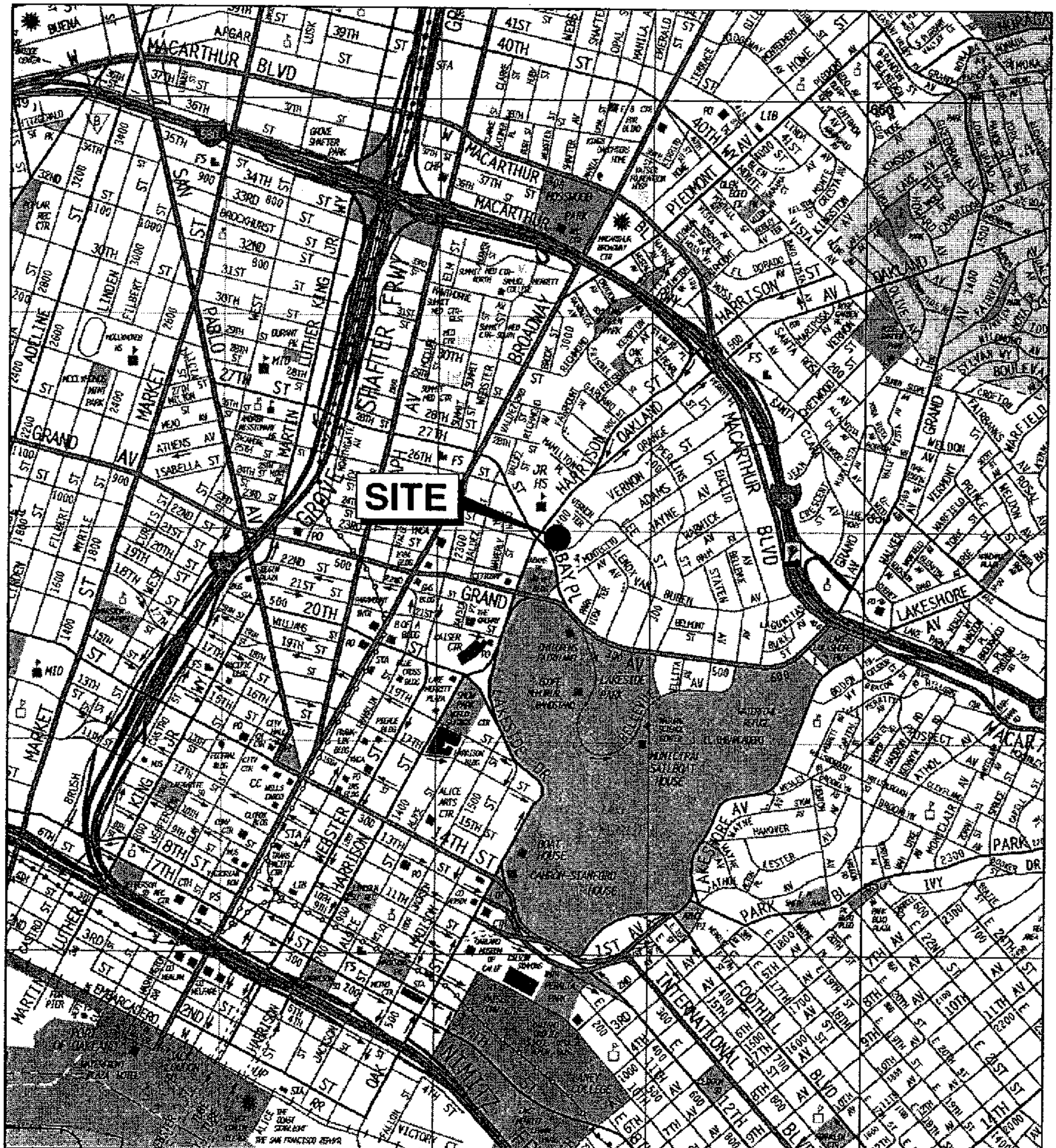
California Division of Mines and Geology (1982), State of California Special Studies Zones, Oakland West, Revised Official Map.

California Division of Mines and Geology (1966), Probabilistic seismic hazard assessment for the State of California, DMG Open-File Report 96-08.

International Conference of Building Officials (1997), Uniform Building Code, Volume 2 – Structural Engineering Design Provisions, Chapter 16.

Toppozada, T.R. and Borchardt G. (1998). "Re-evaluation of the 1936 "Hayward Fault" and the 1838 San Andreas Fault Earthquakes." *Bulletin of Seismological Society of America*, 88(1), 140-159.

Working Group on California Earthquake Probabilities (WGCEP) (1999), "Earthquake Probabilities in the San Francisco Bay Region: 2000 to 2030 – A Summary of Findings." Open File Report 99-517.



Base map: The Thomas Guide
Alameda County
1999

0 1/4 1/2 Mile

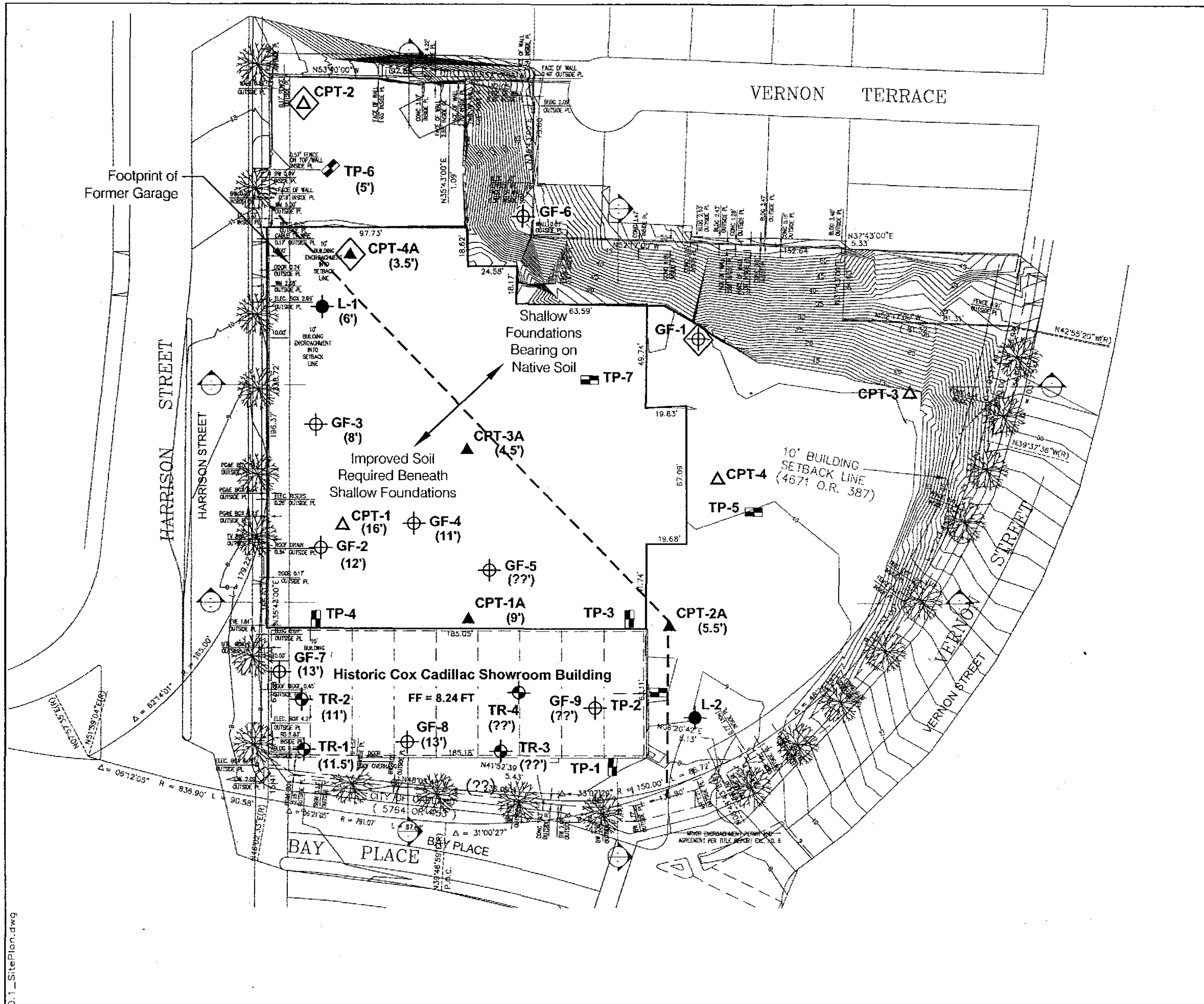
Approximate scale

COX CADILLAC SITE DEVELOPMENT
Oakland, California

SITE LOCATION MAP

Treadwell&Rollo

Date 05/21/04	Project No. 3830.01	Figure 1



EXPLANATION

CPT-1A ▲ Approximate location of cone penetration test by Treadwell & Rollo, Inc., February 2004

TR-1 Approximate location of boring by Treadwell & Rollo, Inc., May 2004

TP-1 Approximate location of test pit excavated by others and logged by Treadwell & Rollo, Inc., April 2004

L-1 Approximate location of boring by Lowney Associates, July 2000

CPT-1 △ Approximate location of cone penetration test by Lowney Associates, July 2000

GF-1 Approximate location of boring by Geoforensics, Inc., April 2001

10 Approximate ground surface elevation contours, feet

◇ Indicates groundwater rose to ground surface

(16') Approximate depth to bottom of weak native soil at location

Note: Finished floor elevation and elevation contours are referenced to City of Oakland datum.

True North Project North

0 50 Feet

Approximate scale

3830.1_SitePlan.dwg

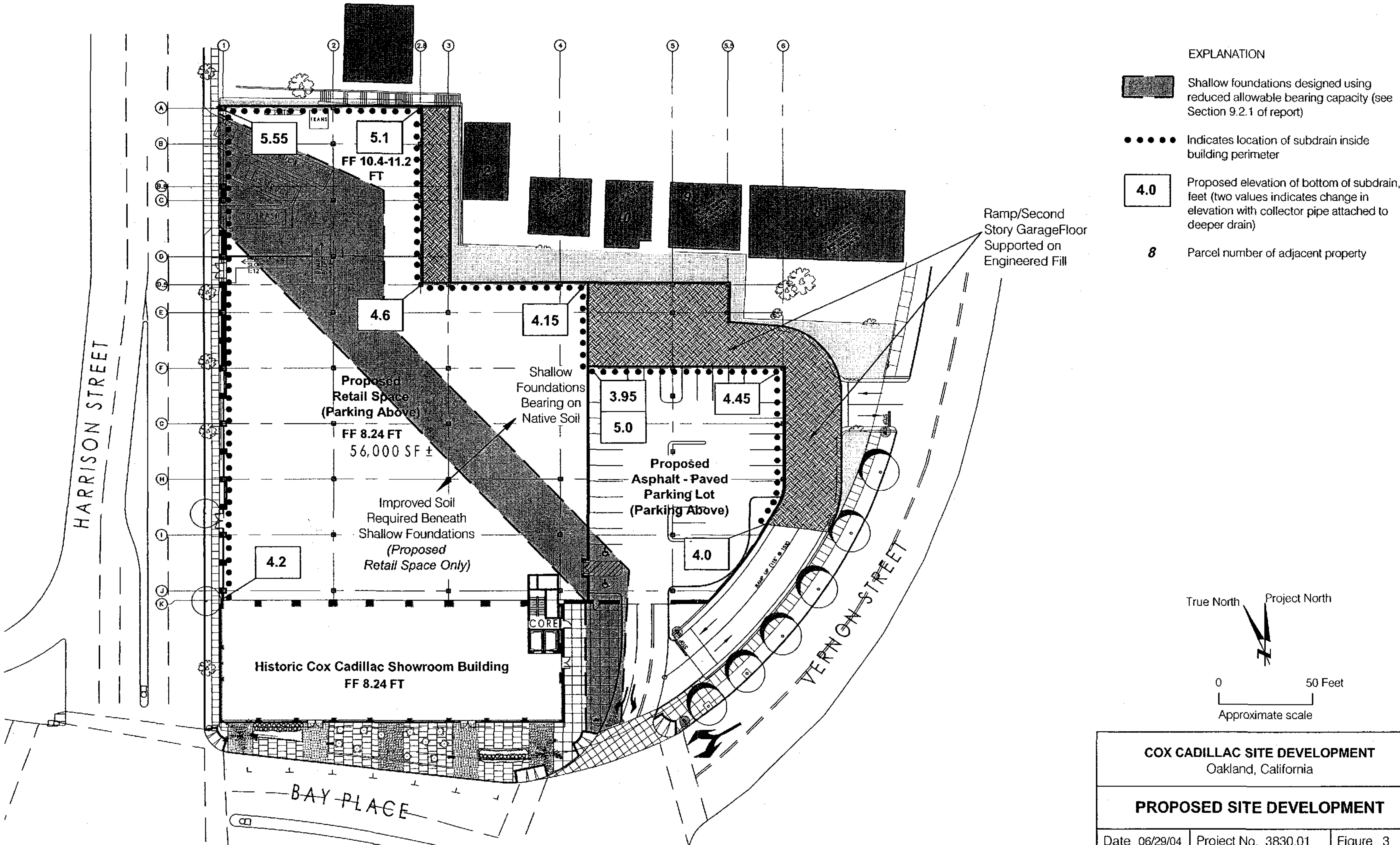
Reference: AutoCAD file "x-civil-topo.dwg," provided by Christiani Johnson Architects.

COX CADILLAC SITE DEVELOPMENT
Oakland, California

EXISTING SITE PLAN WITH TEST LOCATIONS

Date 05/26/04	Project No. 3830.01	Figure 2

Treadwell&Rollo



COX CADILLAC SITE DEVELOPMENT
Oakland, California

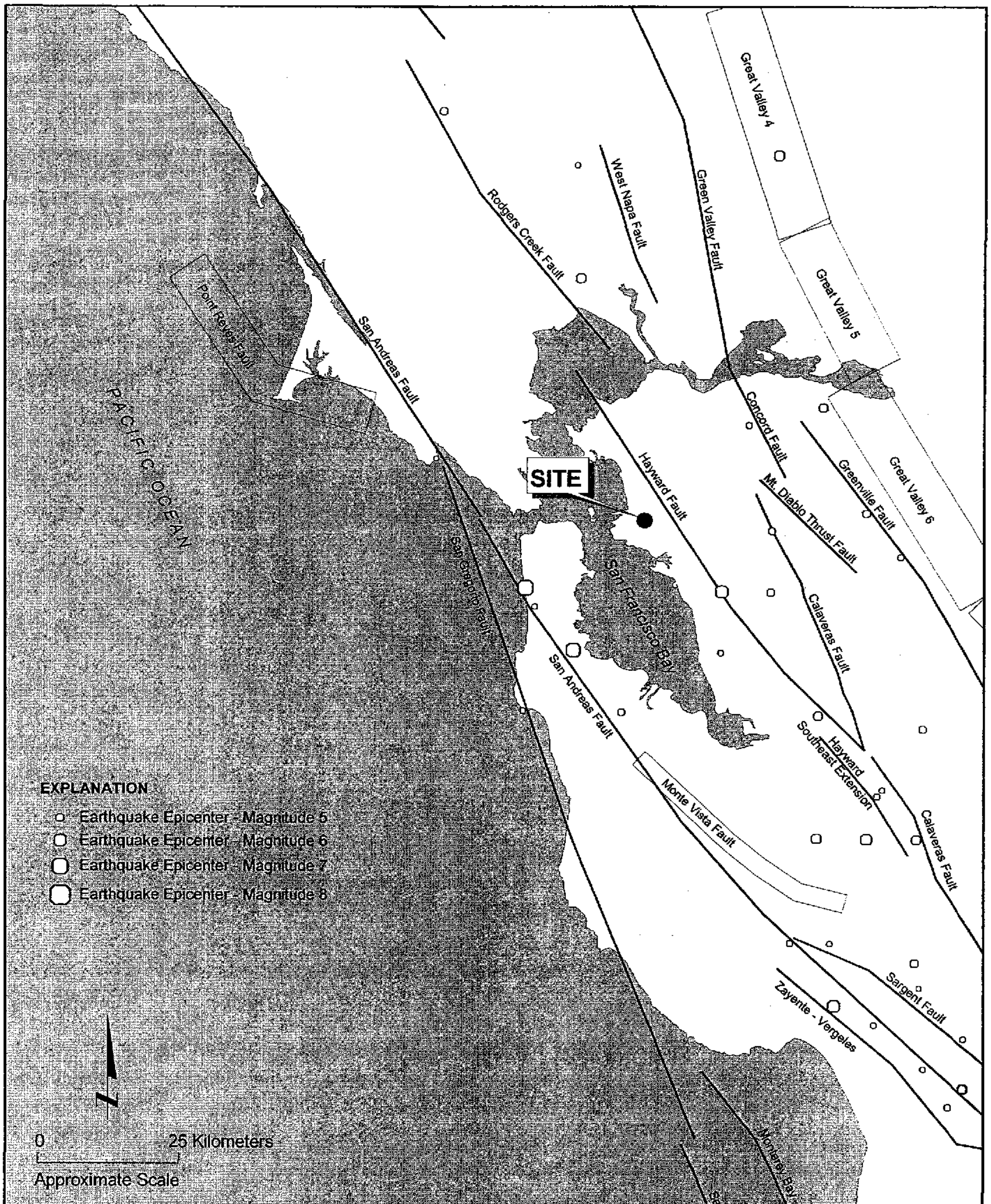
PROPOSED SITE DEVELOPMENT

Date 06/29/04	Project No. 3830.01	Figure 3

Treadwell&Rollo

3830.1_PropSiteDivelop.dwg

Reference: Sheet A.1, "First Floor, Cox Cadillac Site, Whole Foods Scheme, Oakland, California," prepared by MBH Architects, dated 29 April 2004.



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

COX CADILLAC SITE DEVELOPMENT Oakland, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
Treadwell&Rollo	Date: 05/05/04 | Project No. 3830.01 | Figure: 4

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

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

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

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

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

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

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

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

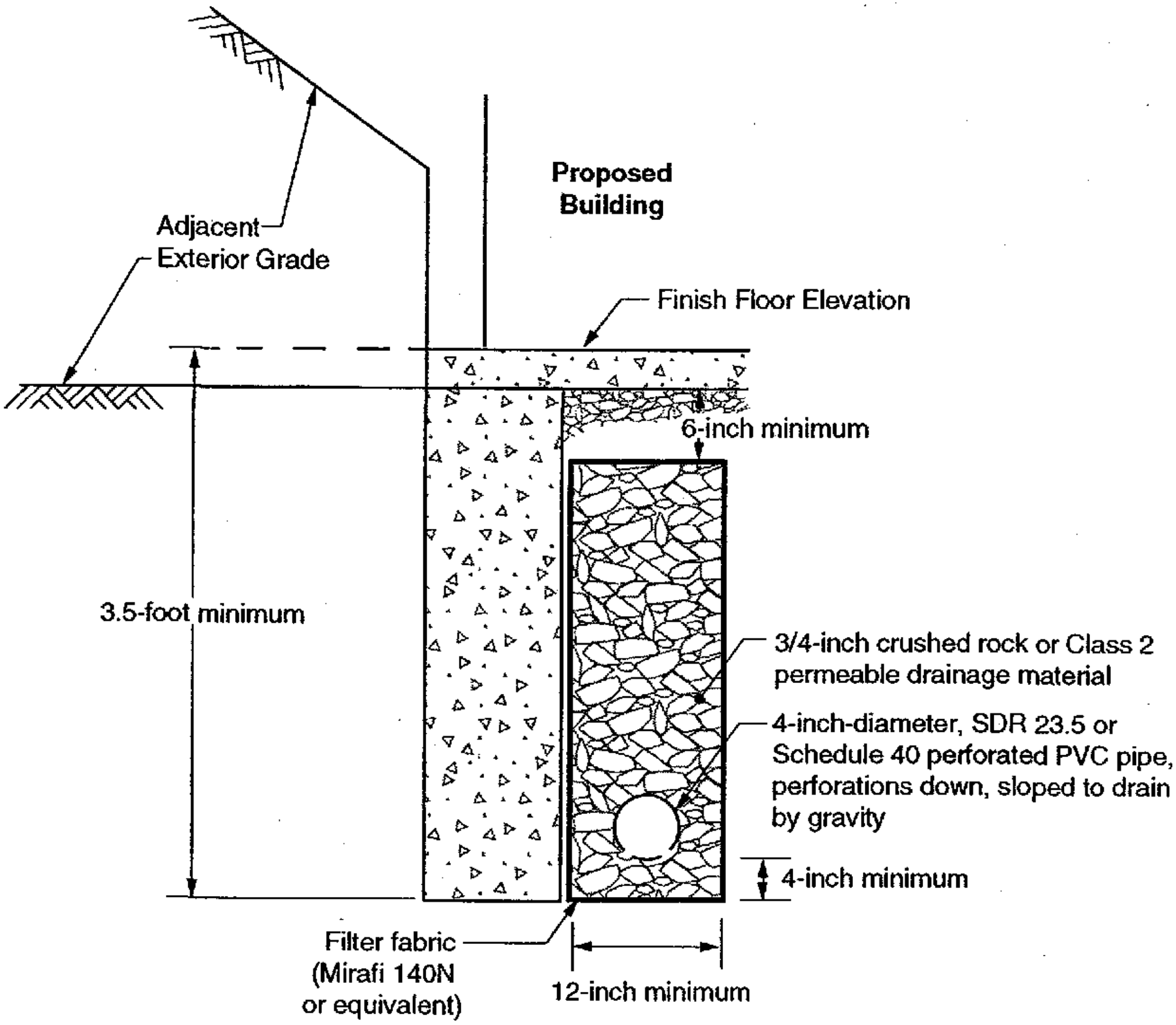
IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others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

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

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

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

COX CADILLAC SITE DEVELOPMENT Oakland, California	**MODIFIED MERCALLI INTENSITY SCALE**		
Treadwell&Rollo	Date: 05/05/04	Project No. 3830.01	Figure: 5



Not to scale

COX CADILLAC SITE DEVELOPMENT
Oakland, California

Treadwell&Rollo

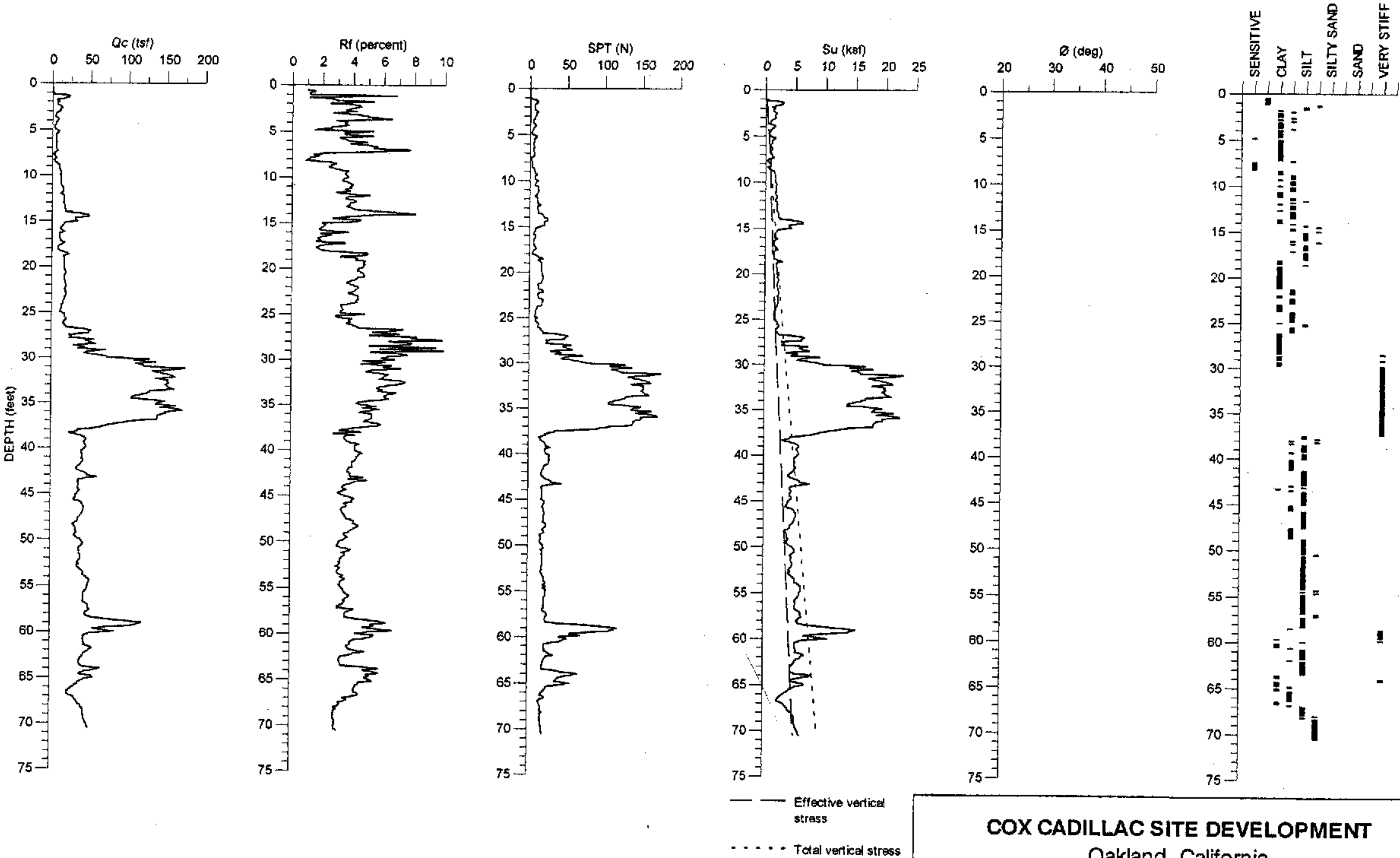
SUBDRAIN DETAIL

Date 05/21/04 | Project No. 3830.01 | Figure 6

Treadwell&Rollo

APPENDIX A

Logs of Cone Penetration Tests and Borings



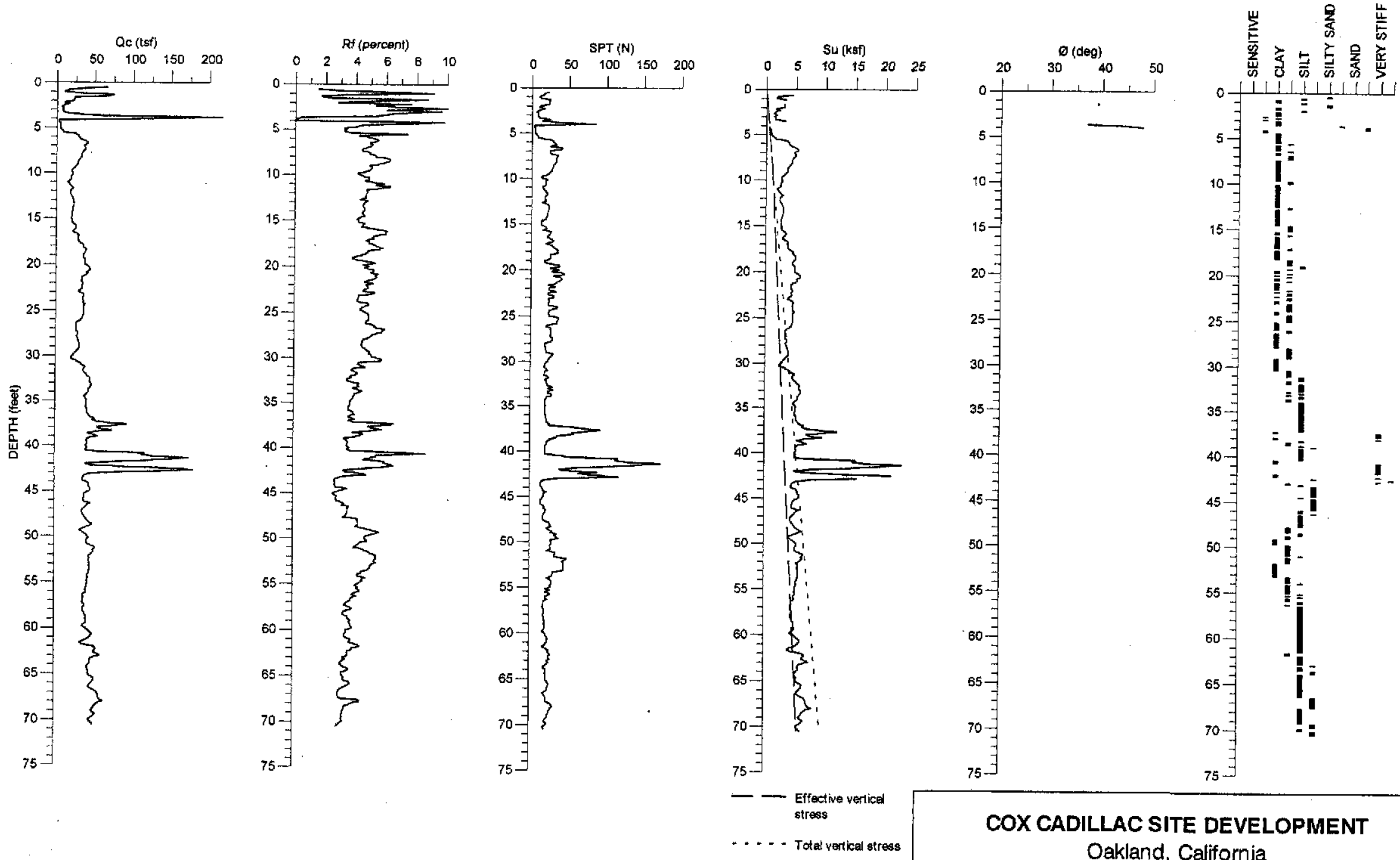
Terminated at 70.5 feet
Groundwater estimated at 6.7 feet.
Date performed: 2/3/04.
Elevation: 9.5 feet, datum: City of Oakland Datum.

COX CADILLAC SITE DEVELOPMENT
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-1A

Date 05/05/04	Project No. 3830.01	Figure A-1

Treadwell&Rollo



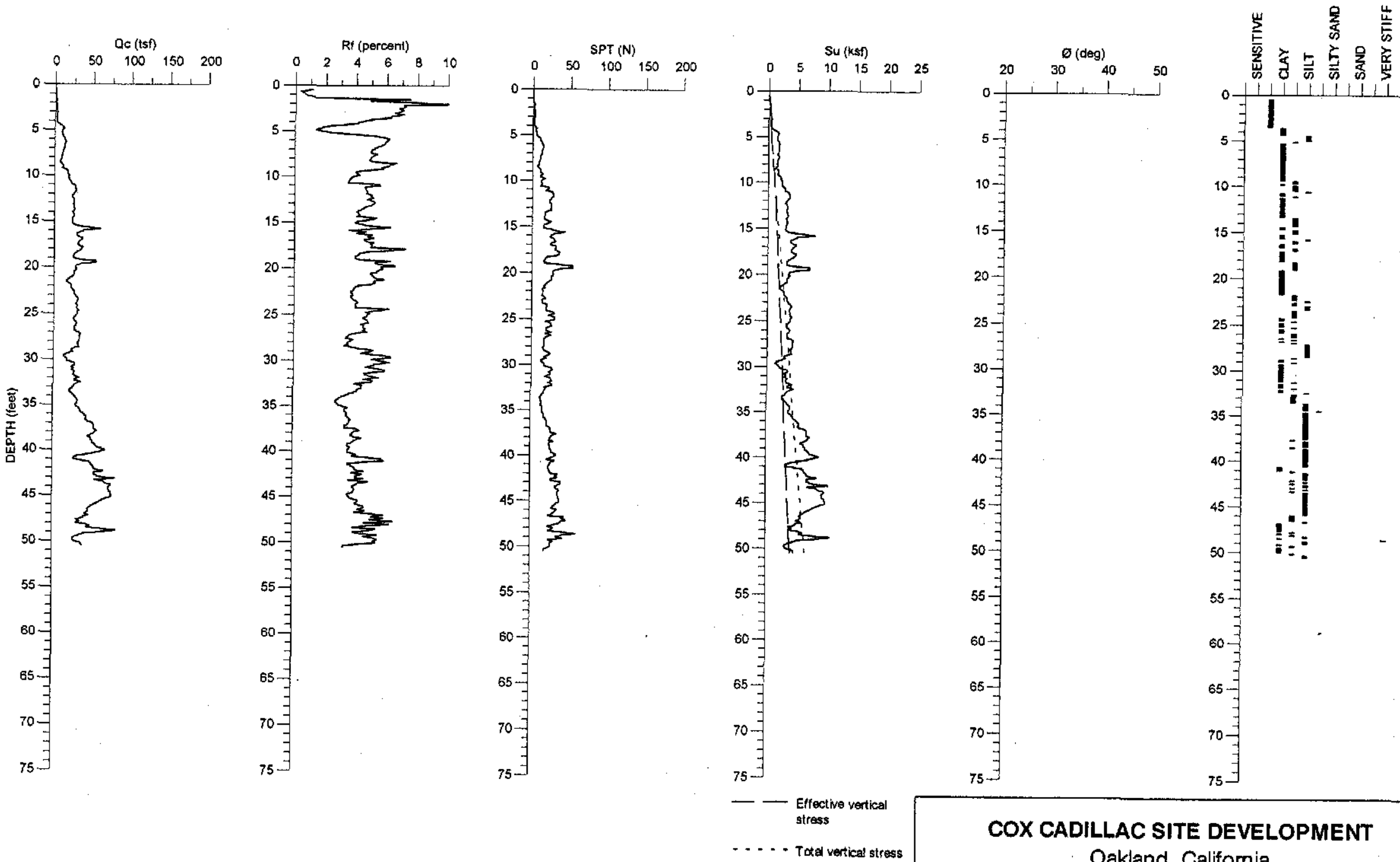
Terminated at 70.5 feet
Groundwater estimated at 9 feet.
Date performed: 2/3/04.
Elevation: 9.5 feet, datum: City of Oakland Datum.

COX CADILLAC SITE DEVELOPMENT
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-2A

Date 05/05/04	Project No. 3830.01	Figure A-2

Treadwell&Rollo



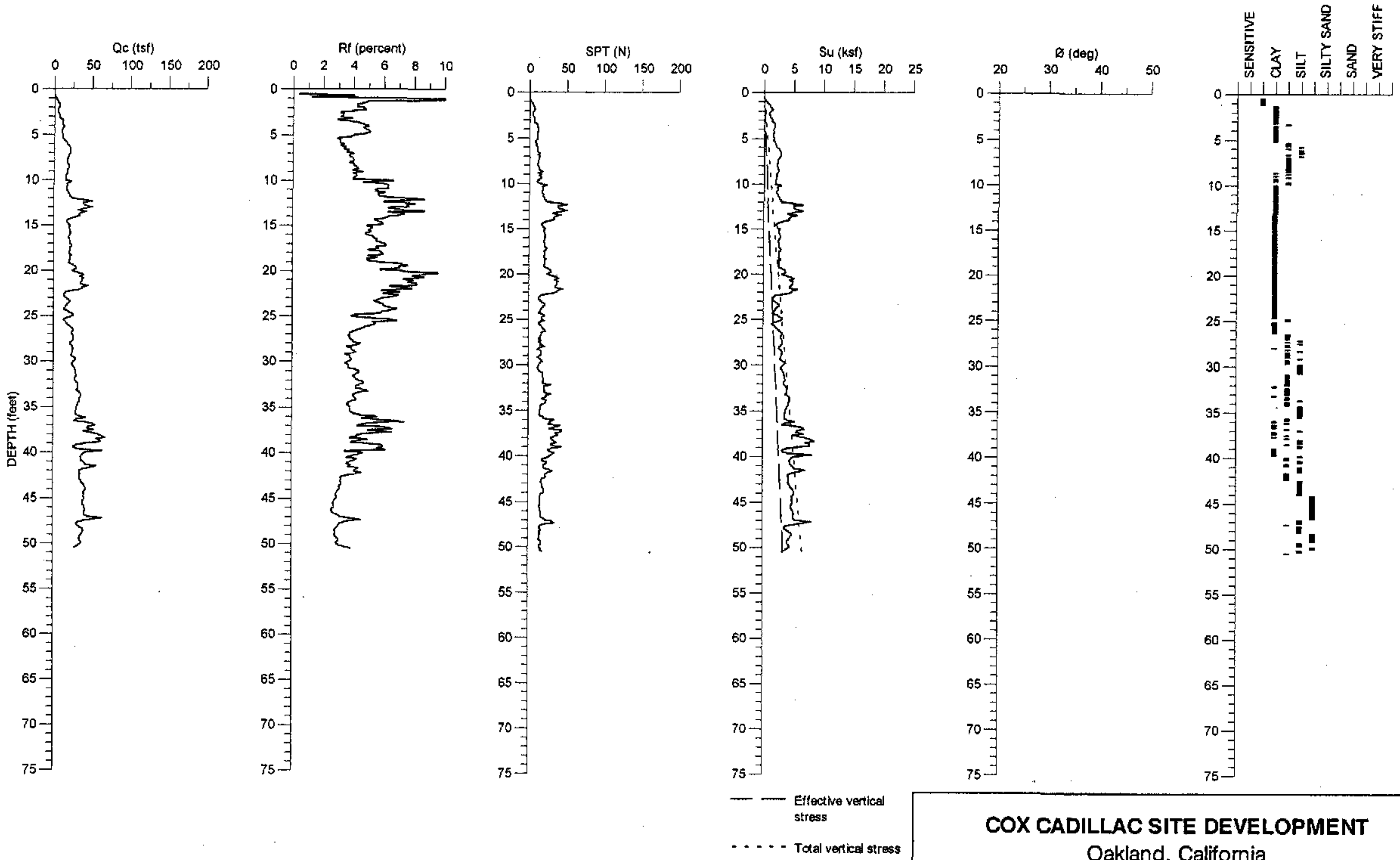
COX CADILLAC SITE DEVELOPMENT
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-3A

Date 05/05/04	Project No. 3830.01	Figure A-3

Treadwell&Rollo

Terminated at 50.5 feet
Groundwater estimated at 10.5 feet.
Date performed: 2/3/04.
Elevation: 10 feet, datum: City of Oakland Datum.



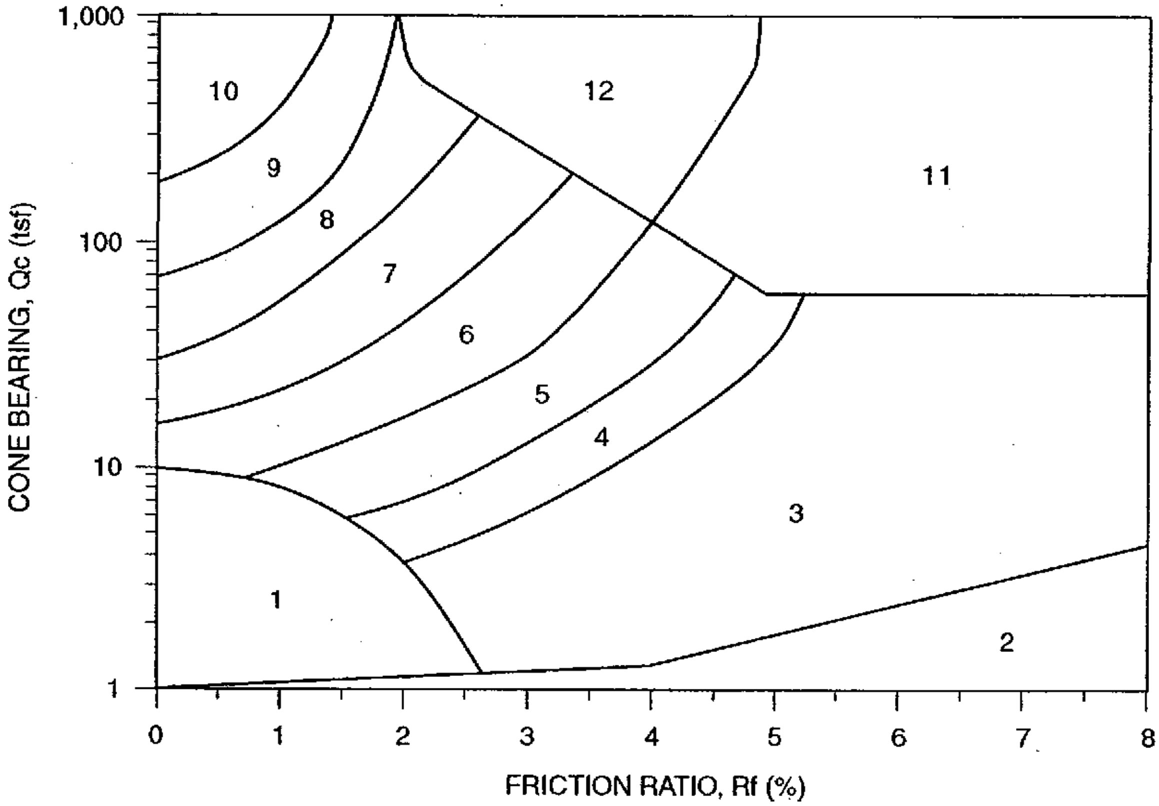
Terminated at 50.5 feet
Groundwater estimated at 0.1 feet.
Date performed: 2/3/04.
Elevation: 10 feet, datum: City of Oakland Datum.

COX CADILLAC SITE DEVELOPMENT
Oakland, California

CONE PENETRATION TEST RESULTS
CPT-4A

Date 05/05/04	Project No. 3830.01	Figure A-4

Treadwell&Rollo



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).

COX CADILLAC SITE DEVELOPMENT
Oakland, California

Treadwell&Rollo

SARIMIENTO

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 02/17/04	Project No. 3830.01	Figure A-5

PROJECT:	COX CADILLAC SITE DEVELOPMENT Oakland, California	Log of Boring TR-1
		PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2
Date started:	5/8/04
Date finished:	5/8/04
Drilling method:	Mobile B-24 truck mounted rig, 6-1/2-inch-diameter hollow-stem augers
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Safety
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by:	A. Blaisdell

Ground Surface Elevation: 8.24 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				Six inches PCC with clay tile	FILL (0–11.5 ft)						
0.5–4			SP-SC	SAND with CLAY (SP-SC) brown, very loose, moist, trace gravel, many brick and rock fragments in upper 6 inches							
2	S&H	2									
4	SPT	2		⊻ (5/8/04, 10:30 AM)							
4–5.5			SC	CLAYEY SAND (SC) dark gray, very loose, wet							
5.5–11.5			CH	SANDY CLAY (CH) gray, very soft, wet							
6	SPT	2									
8	S&H	1		trace gravel, lost lower 6 inches of sample							
10	ST			no recovery							
11.5–15.5			CL	GRAVELLY CLAY (CL) olive-brown, stiff, wet							
12	S&H	11									
15	S&H	17		very stiff, less gravelly							

LABORATORY TEST DATA (no entries)

Boring terminated at a depth of 15.5 feet
Boring backfilled with neat cement grout.
Groundwater was measured at a depth of 4 feet.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on City of Oakland datum.

Treadwell&Rollo

Project No.: 3830.01 | Figure: A-6

TEST GEOTECH LOG 383001.GPJ TR.GDT 6/29/04

PROJECT:	COX CADILLAC SITE DEVELOPMENT Oakland, California	Log of Boring TR-2
		PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: A. Blaisdell

Date started: 5/8/04 | Date finished: 5/8/04

Drilling method: Mobile B-24 truck mounted rig, 6-1/2-inch-diameter hollow-stem augers

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION — Ground Surface Elevation: 8.24 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				Six inches PCC with clay tile						
0.5–5			SP-SC	SAND with CLAY (SP-SC) brown, loose, moist, trace gravel (FILL)						
2	S&H	4								
4	SPT	3		∇ (5/8/04, 1:05 PM) very loose, wet						
5–11			CH	CLAY with SAND (CH) dark gray, soft, wet, coarse sand, trace fine gravel (FILL)						
6	SPT	4								
8	S&H	2		very soft, heavy organics, contains stiffer clods within overall soft matrix, with gravel						
10	S&H	3								
11–13			CL	SANDY CLAY (CL) olive-brown, stiff, wet						
12	S&H	10								

Boring terminated at a depth of 13 feet
Boring backfilled with neat cement grout.
Groundwater was measured at a depth of 4 feet.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on City of Oakland datum.

Treadwell&Rollo

Project No.: 3830.01 | Figure: A-7

PROJECT: COX CADILLAC SITE DEVELOPMENT Oakland, California	Log of Boring TR-3 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: A. Blaisdell

Date started: 5/8/04 | Date finished: 5/8/04

Drilling method: Mobile B-24 truck mounted rig, 6-1/2-inch-diameter hollow-stem augers

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Safety

Sampler: Standard Penetration Test (SPT)

LABORATORY TEST DATA

Ground Surface Elevation: 8.24 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				Six inches PCC with clay tile						
0.5–2				RUBBLE FILL heterogeneous mix of sand, brick and concrete (FILL)						
2–3				12 inches PCC (FILL)						
3–3.7				8-inch void below slab (FILL)						
4–6	SPT	4	SP-SM	▽ (5/8/04, 11:33 AM) SAND with SILT (SP-SM) dark brown, loose, wet, heavy brick fragments (FILL)						
6–6.7	SPT	50/0"		dark gray-brown, loose to medium dense (FILL)						
6.7–7.2				WOOD (FILL)						
7.2–7.5				BRICK (FILL)						

Boring met practical refusal during drilling at 7.5 feet.

Boring backfilled with neat cement grout.
Groundwater was measured at a depth of 4 feet.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on City of Oakland datum.

Treadwell&Rollo

Project No.: 3830.01 | Figure: A-8

TEST GEOTECH LOG 383001.GPJ TR.GDT 6/29/04

PROJECT: **COX CADILLAC SITE DEVELOPMENT** Oakland, California

Log of Boring TR-4

Boring location: See Site Plan, Figure 2

Logged by: A. Blaisdell

Date started: 5/8/04 | Date finished: 5/8/04

Drilling method: Mobile B-24 truck mounted rig, 6-1/2-inch-diameter hollow-stem augers

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Safety

Sampler: Sprague & Henwood (S&H)

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 8.24 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				Six inches PCC with clay tile						
0.5–3.5			CL	SANDY CLAY with GRAVEL (CL) mottled yellow-brown, brown, and gray, soft to medium stiff, moist (FILL)						
2	S&H	4								
3.5				∇ (5/8/04, 12:45 PM)						
3.5–4.8			GP	GRAVEL (GP) gray, medium dense, wet, angular to sub-angular (FILL)						
4	S&H	13								
4.8–9			GC	CLAYEY GRAVEL (GC) gray, medium dense, wet, angular to sub-angular, clay in gravel matrix is soft, wood at 6 feet (FILL)						
6	S&H	16								
8	S&H	31		gray-green						

Boring met practical refusal during drilling at 7.5 feet; sampler advanced to 9 feet.

Boring backfilled with neat cement grout.
Groundwater was measured at a depth of 4 feet.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on City of Oakland datum.

Treadwell&Rollo

Project No.: 3830.01 | Figure: A-9

TEST GEOTECH LOG 383001.GPJ TR.GDT 6/29/04

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	Range of Grain Sizes: 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

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

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
- Analytical laboratory sample
- Sample taken with Direct Push sampler

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

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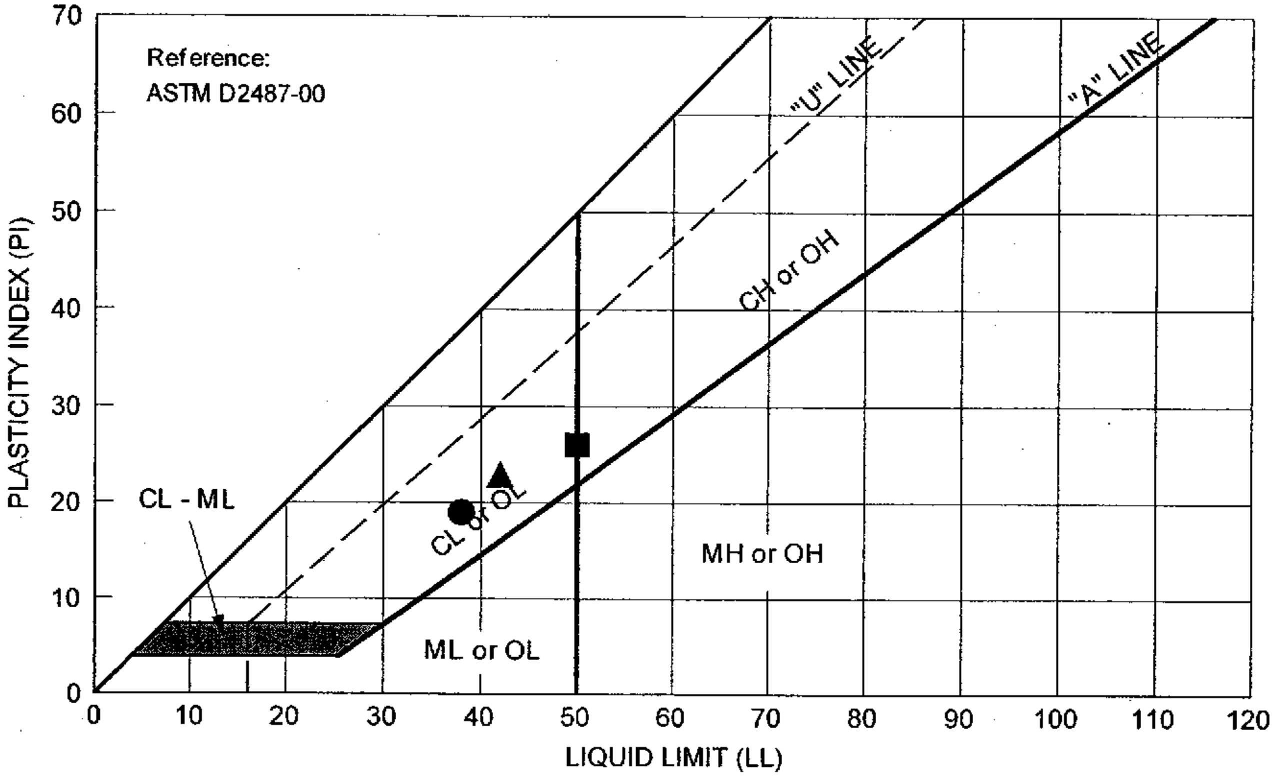
APPENDIX B

Laboratory Test Results



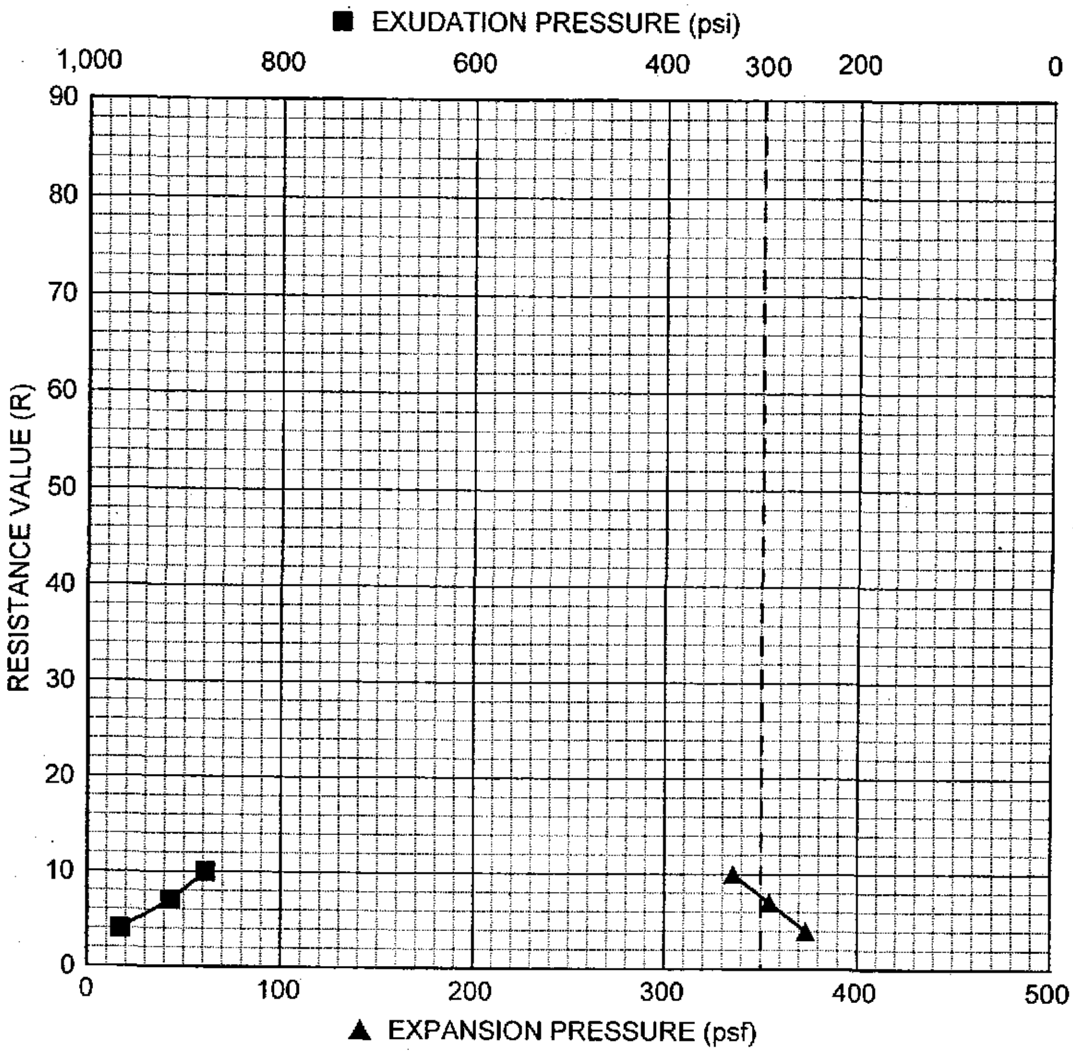
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	TP-4 at 2.5 feet	CLAY with SAND (CL), orange, orange-brown, and dark brown [FILL]	29.4	38	19	---
▲	TP-6 at 2 feet	CLAY with SAND (CL), dark gray	31.2	42	23	---
■	TP-7 at 2.5 feet	CLAY with SAND (CL/CH), orange-brown	26.7	50	26	---

COX CADILLAC SITE DEVELOPMENT
Oakland, California

Treadwell&Rollo

PLASTICITY CHART

Date 05/05/04 | Project No. 3830.01 | Figure B-1



Specimen ID:	A	B	C	D
Water Content (%)	20.5	21.5	22.0	—
Dry Density (pcf)	105.1	102.4	101.7	—
Exudation Pressure (psi)	329	291	253	—
Expansion Pressure (psf)	61	43	17	—
Resistance Value (R)	10	7	4	—

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
TP-5 at 2.5 to 3.5 feet	SANDY CLAY (CL), mottled orange-brown and olive-brown	—	43 psf	7

COX CADILLAC SITE DEVELOPMENT
Oakland, California

Treadwell&Rollo

RESISTANCE VALUE TEST DATA

Date 05/05/04	Project No. 3830.01	Figure B-2

Treadwell&Rollo

APPENDIX C

Logs of Borings and Cone Penetration Tests from Lowney and GeoForensics Investigations

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 Inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				gravelly sandy CLAY		
5	1 - 1		33	fine sandy CLAY/clayey fine SAND - mottled grey & orange-brown; slightly moist to moist; very stiff/medium dense	98.4	25.8
10	1 - 2		45	gravelly clayey SAND - orange-brown; slightly moist; medium dense	105.1	21.7
15	1 - 3		48	clayey SAND - grey-brown; slightly moist to moist; medium dense to dense		
				sandy CLAY - dark grey & black; wet; very stiff to hard		
20	1 - 4		40		110.5	18.4
25				Groundwater encountered at 8 feet, rose to surface after 1.5 hours. Bottom of boring at 20 feet Drilled on 04/05/01 Logged by dd/ba Mobile B-24 drilling rig Modified California sampler 140# hammer		
30						
35						

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Figure A1 - Log of Boring GF-1

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 Inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				silty CLAY with sand - mottled grey & orange-brown; slightly moist		
5				CLAY - green-grey; slightly moist; soft		
	2 - 1		11		94.8	29.3
10				clayey SILT - black; organic; very moist to wet; soft	98.1	23.0
15	2 - 2		21	sandy CLAY - dark grey & black; wet; stiff		
20	2 - 3		28	sandy CLAY with trace fine gravel - mottled orange & grey-brown; slightly moist; very stiff	101.7	24.5
25						
30						
35						

Groundwater at 12 feet after 1 hour (not stabilized).
Bottom of boring at 21.5 feet
Drilled on 04/05/01
Logged by dd/ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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Figure A2 - Log of Boring GF-2

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 Inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
5	3 - 1		9	CLAY - dark grey-black; slightly moist; medium stiff	94.2	26.0
10	3 - 2		21	sandy CLAY - dark grey-black; slightly moist; stiff	101.0	24.7
15	3 - 3		27	clayey fine SAND/fine sandy CLAY - mottled orange & grey-brown; slightly moist; medium dense/very stiff		
20	3 - 4		29	fine sandy CLAY with trace fine gravel - mottled orange & grey-brown; slightly moist; very stiff		
25						
30						
35						

Groundwater encountered at 13 feet, rose to 10.5 feet after 1 hour.
Bottom of boring at 20 feet
Drilled on 04/05/01
Logged by ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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Figure A3 - Log of Boring GF-3

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 Inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				silty CLAY with trace sand & fine gravels - tan-brown; slightly moist; soft		
	4 - 1		8		93.6	25.4
5				sandy CLAY - mottled orange and reddish dark brown; moist; soft	98.5	23.0
10	4 - 2		11	CLAY - black; slightly moist; medium stiff	99.0	24.0
15	4 - 3		25	sandy CLAY with few gravels - grey; slightly moist to moist; very stiff	118.6	15.8
				gravelly SAND - brown; wet; loose	108.0	19.0
20	4 - 4		25			
				sandy CLAY with few fine gravels - mottled orange & grey-brown; slightly moist; very stiff		
25	4 - 5		28	as above - stiff	109.1	18.0
30	4 - 6		30	CLAY with fine sand & reeds? - mottled orange & grey-brown; slightly moist; very stiff		
35				Groundwater at 6.5 feet after 1 hour. Bottom of boring at 30 feet Drilled on 04/05/01; Logged by ba Mobile B-24 drilling rig Modified California sampler; 140# hammer		

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Figure A4 - Log of Boring GF-4

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				4" CONCRETE SLAB		
	5 - 1			sandy CLAY - mottled orange & brown; firm; moist		
5				CONCRETE 3 feet thick		
				Practical refusal at 6 feet.		
10						
15						
20						
25						
30						
35						

No groundwater encountered.
Bottom of boring at 6 feet
Drilled on 04/05/01
Logged by dd/ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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Figure A5 - Log of Boring GF-5

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 Inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				sandy CLAY/clayey SAND - dark brown; slightly moist; firm to stiff		
5	6 - 1		42		94.9	14.6
				clayey gravelly SAND - orange-brown; slightly moist; medium dense		
10	6 - 2		88		107.0	18.0
				as above, but with lenses of silty fine SAND with trace clay - medium dense to dense		
15				fine sandy CLAY - orange-brown; moist; very stiff		
	6 - 3		50/4"	silty fine SAND - orange & yellow-brown; dense	102.0	20.0
20						
25				No groundwater encountered. Bottom of boring at 19.5 feet Drilled on 04/05/01 Logged by dd Minute Man portable drilling rig Modified California sampler 70# hammer		
30						
35						

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Figure A6 - Log of Boring GF-6

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				6" CONCRETE SLAB SAND - buff; loose; very moist/wet (excavated prior to drilling)		
5	7 - 1		10	sandy CLAY with few gravels - dark grey-brown; saturated; medium stiff		
10						
	7 - 2		70	CLAY - mottled orange & grey-brown; slightly moist; hard		
15						
	SPT 7 - 3		43	as above; very stiff		
20						
25				Groundwater encountered at 5 feet. Bottom of boring at 19.5 feet Drilled on 05/09/01 Logged by BA Mobile B-24 drilling rig Modified California & Split Spoon samplers 140# hammer		-
30						
35						

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Figure A - Log of Boring GF-7

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				6" CONCRETE SLAB		
				SAND		
				CONCRETE		
5						
	8 - 1		8	CLAY - dark grey & black; moist; medium stiff		
10						
	8 - 2		15	sandy CLAY - grey; moist; stiff		
15						
	no recovery		31	sandy CLAY - very stiff		
20						
25				Groundwater encountered at 7 feet Bottom of boring at 19.5 feet Drilled on 05/09/01 Logged by ba Mobile B-24 drilling rig Modified California sampler 140# hammer		
30						
35						

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Figure A - Log of Boring GF-8

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				6" CONCRETE		
				CONCRETE SLAB		
5				Materials removed by Levine-Fricke prior to GeoForensics drilling.		
10						
	9 - 1		46	CLAY with sand - mottled orange & grey-brown; slightly moist; very stiff		
15						
20						
25						
30						
35						

Groundwater reported at 5 feet.
Bottom of boring at 14.5 feet
Drilled on 05/09/01
Logged by ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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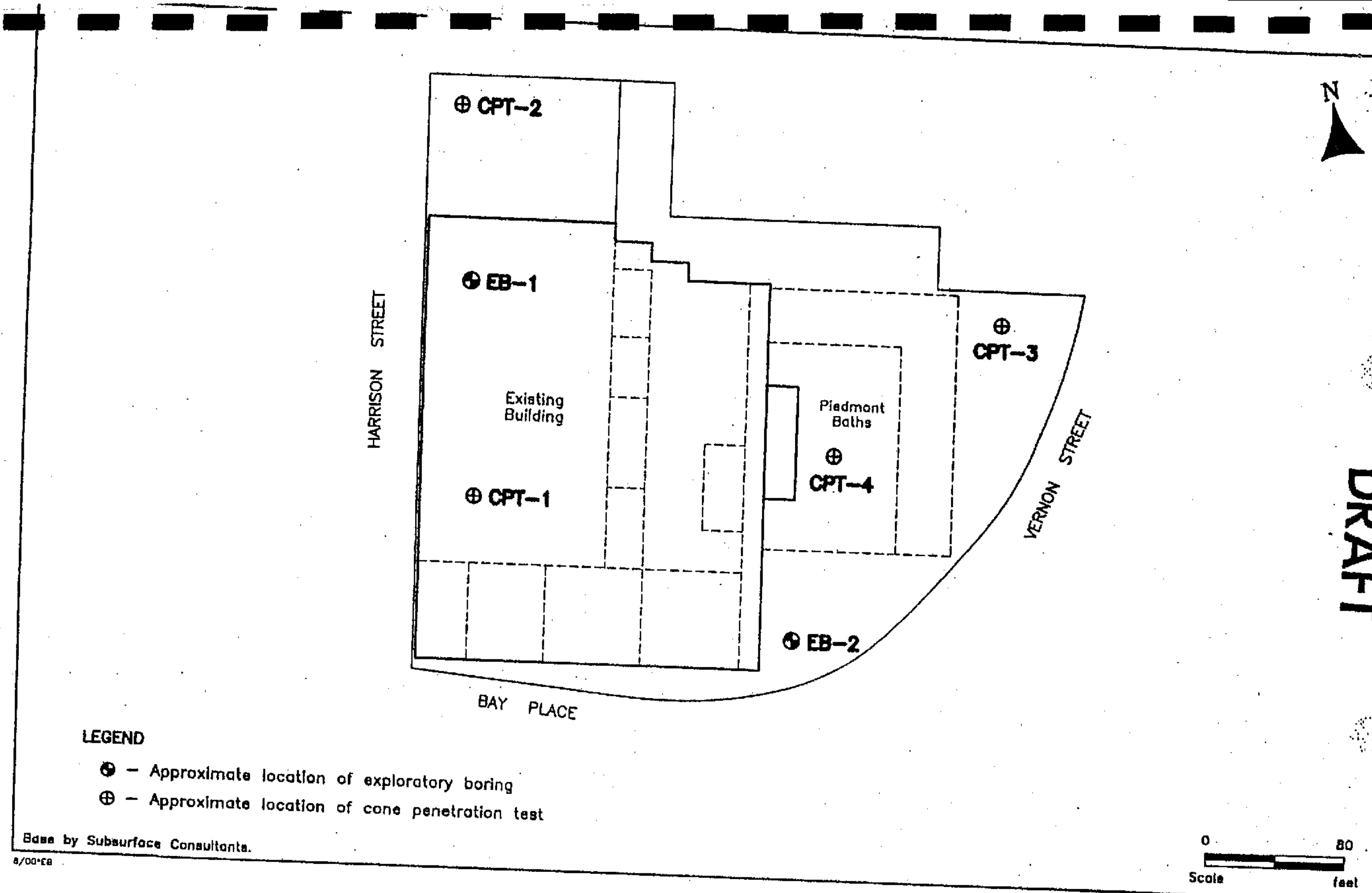
Figure A - Log of Boring GF-9



SITE PLAN

230 BAY PLACE

Oakland, California

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 2
596-71

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, rotary-wash auger drilling equipment, and truck-mounted cone penetration test (CPT) equipment. Two approximately 5-inch-diameter exploratory borings (EB-1 and EB-2) were drilled, and four CPTs (CPT-1, 2, 3 and 4) were hydraulically pushed on July 27, 2000, to maximum depths of 40 to 41 feet. The approximate locations of the exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix. The CPT data is also attached.

The locations of borings and CPTs were determined by approximate measurements from site and building features. Elevations of the borings were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Most of the soil samples were obtained with a 2.5-inch I.D. Modified-California split barrel sampler. Modified-California penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall; the sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). In addition, 2.0-inch I.D. samples were obtained using a Standard Penetration Test (SPT) split barrel sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the samplers the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring and CPT logs and related information show subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

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

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4"	3"	12"
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION

MODIFIED CALIFORNIA

D&M UNDERWATER SAMPLER

SHELBY TUBE

NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

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DRAFT

FIGURE A-1

EXPLORATORY BORING: EB-1

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
0	SURFACE ELEVATION:					
	2½ inches concrete over 6 inches old concrete					
	SILTY CLAY (CL) (FILL) medium stiff to stiff, moist, black with brown mottles, organics, moderate plasticity	CL, FILL	6	24	100	
5	**SILTY CLAY (CL)** stiff, moist, dark gray, some sand and gravel, strong hydrocarbon odor	CL	13	20	110	
	SILTY CLAY (CL) stiff, moist, gray and brown mottled, trace gravel, moderate plasticity		25	18	114	
10	Plasticity Limit = 21, Liquid Limit = 35		37	18	110	
			23	18	111	
15						
		CL	39	17		
20						
	gravel and sand in cuttings					
			14	29		
25						
			31	30		
30						

DRAFT

Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

LA_CORP.GDT 8/4/00 MY' FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 1500

BORING TYPE: 4-7/8 INCH ROTARY WASH

LOGGED BY: DGJ

START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71

PROJECT: 230 BAY PLACE

LOCATION: OAKLAND, CA

COMPLETION DEPTH: 39.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30		**SILTY CLAY (CL)** stiff, moist, gray and brown mottled, trace gravel, moderate plasticity	CL						
					63	X	23			
	35									
					53	X	25	104		△ ~2.0; ○ ~3.5
	40		Bottom of Boring at 39 feet							
	45									
	50									
	55									
	60									

DRAFT

GROUND WATER OBSERVATIONS:

NO FREE GROUNDWATER ENCOUNTERED

LA_CORP.GDT 8/4/00 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

SURFACE ELEVATION:

Depth (FT)	Material Description and Remarks	Soil Type	Penetration Resistance (Blows/Ft.)	Moisture Content (%)	Dry Density (PCF)	Percent Passing No. 200 Sieve
0	3 inches asphalt concrete					
	CLAYEY SAND (SC) [FILL] dense, moist, yellowish-brown, fine, trace fine gravel	SC, FILL	40	13	114	
5	SAND (SP) dense, wet, greenish-gray, fine to medium, trace clay, trace gravel	SP	38	14	118	13
	SILTY CLAY (CL) very stiff, moist, yellowish-brown, gray mottles, moderate plasticity		27	25	100	
10	Plasticity Index = 47, Liquid Limit = 72		30	35	89	
15			52	18	114	
20		CL	31	24		
25			38	19	107	
30			68	18		

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Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

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Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
	30		**SILTY CLAY (CL)** very stiff, moist, yellowish-brown, gray mottles, moderate plasticity							
					51	X	25	100		○
	35			CL						
					84	X	20			
	40		Bottom of Boring at 39 feet							
	45									
	50									
	55									
	60									

DRAFT

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

Treadwell&Rollo

APPENDIX D

Soil Corrosivity Analysis Results

ETS

Environmental Technical Services

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

1343 Redwood Way
Petaluma, CA 94954
(707) 795-9605 / FAX 795-9384

Serving people and the environment so that both benefit.

COMPANY:	Treadwell & Rollo, 501 14th Street, 3rd Floor, Oakland, CA 94612			ANALYST(S)	SUPERVISOR
ATTN:	Andrew R. Blaisdell	PROJ. NO.:	DATE RECEIVED	DATE of COMPLETION	W. Zuo	D. Jacobson
PROJ. NAME:	Cox Cadillac	3830.01	4/30/04	5/7/04	G. Hundt	LAB DIRECTOR
LOCATION:	Oakland, California.					G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
00594-1	CC1/O	TP-6 @ 3.0'	7.40	1750	[570]	225	74
00564-2	CC2/O	TP-7 @ 5.5'	7.52	3170	[315]	51	55
Method	Detection	Limits —>	—	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
00594-1	CC1/O	TP-6 @ 3.0'		0.111		+471.6	
00564-2	CC2/O	TP-7 @ 5.5'		0.012		+826.4	
Method	Detection	Limits —>	—	0.1	0.1	1	0.1

******************************** COMMENTS ********************************

Resistivities are over 1,500 & 3,000 ohm-cm which is good, and soil reactions (i.e., pHs) are mildly alkaline which also helps; chloride and sulfate levels are not high; but sulfide is variable. The CalTrans times to perforation are as follows: for 18 ga steel the time is a good 31.4 yrs, and for 12 ga it goes to 69.1 yrs; and for CC2/O the respective times are even better at 40.1 yrs, and 88.1 yrs. Neither chloride nor sulfate are high enough to be of direct concern. Chloride is not high enough to impact contained steel reinforcement; and sulfate should not have a direct negative impact on concrete, mortar, grout or cement. One sulfide is under 0.1 ppm, which is good, but the other is just over 0.1 ppm. And while both redoxes are over +400 mV, one is relatively close to this lower limit. Addition of lime would be on no benefit whatsoever because pHs are already alkaline. Greater longevitiy would necessitate heavier gauge steel or other actions (e.g, wrapping pipe, special fill, etc.). Finally as far as standard concrete & related materials are concerned, it would be best to aerate the CC1/O soil to reduce sulfide concentration (by converting it to sulfate); or use sulfide resistant mixes. Last, be sure that if there is to be any impermeable slab coating (e.g. epoxy, vinyl, etc.]) that adequate sub-slab vapor barriers are specified.

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

Treadwell&Rollo

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Richard D. Rodgers,
Geotechnical Engineer