GEOTECHNICAL INVESTIGATION
1731 – 1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

La Corneta Taqueria
San Francisco, California

1 December 2008
Project No. 2766.03

Treadwell Rollo

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GEOTECHNICAL INVESTIGATION
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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Treadwell & Rollo, Inc. for the proposed La Corneta Palace at 1731-1741 Powell Street in San Francisco (Site Location Map, Figure 1). The project site is at the western corner of the intersection of Powell Street and Columbus Avenue. It is bound by Powell Street to the east, Columbus Avenue to the northeast, a two-story wood framed building and a concrete/asphalt parking lot to the north, a two-story concrete/masonry building to the west, a three stories and a one-story wood frame building to the south. Sidewalk grades vary between Elevation 61 and 62.5 feet[1] along Columbus Avenue and between 62.6 and 65 feet along Powell Street.

In July 2000, we performed a geotechnical investigation at the project site. The proposed project at that time included seismic strengthening and renovation of the existing theatre. However, the strengthening and renovation were not carried out. Instead, the project was scaled down to construct a new basement for storage, measuring about 11 by 12 feet in plan, adjacent to the existing basement near the front of the building.

The project architect is Naylor and Chu, Inc. of San Francisco; and the project structural engineer is Santos Urrutia, Inc.

1.1 Existing Improvements

The site is approximately rectangular in shape with plan dimensions of about 90 by 148 feet and is currently occupied by a 2- to 4-story building constructed in 1907. It was opened in 1909 as the Washington Square Theatre and was extensively remodeled in the late 1930's and reopened as the Palace in 1937. In 1995, the building was completely renovated for use as a movie theater. The renovations included alterations to accommodate a new projection screen, adding a new cast-in-place concrete floor at the second level, and constructing a new five-inch-thick concrete slab-on-grade.

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1 Elevations are obtained from plan titled "As-Built Survey at the Pagoda theatre for Leirum Corporation" prepared by Martin M. Ron Ass dated 23 February 2000 and are referenced to San Francisco City Datum (SFCD).

The existing finished floor elevations at the main theater level, main lobby, and theatre stage, respectively are about Elevation 60.0, 64.5, and 61.8 feet. Two small basement areas are located at the front and rear of the building. The plan dimension for the front basement is approximately 26 by 73 feet with a floor at Elevation 56 feet and the rear basement is approximately 34.5 by 73 feet with a floor at Elevation 51.5 feet. The west basement wall (rear basement) extends 10 feet below the basement slab.

The perimeter walls of the building are steel-framed with brick infill while the basement walls are concrete. Two narrow tunnels or air-ducts extend from the rear basement towards the front of the building; the tunnels do not appear to extend to the front basement. Two-story brick structures were constructed on the northwest and southwest corners of the site. These enclose the stairways that exit to the alleyways. The alleyway on the south side of the building is gently sloping towards west with finished grades varying between Elevation 65 and 60 feet. The alleyway to the north is generally level at approximately Elevation 61 feet.

We have reviewed the following plans for the building:

- portions of the original architectural plans and section, prepared by Mr. A. Mendelman, Architect, not dated
- portions of architectural plans from the 1995 renovation, source and date unknown.

The plans indicate the building is founded on continuous and isolated spread footings. The footings appear to be bottomed at about 7 to 15 feet below the theater level slabs-on-grade. However, the actual elevations of footing cannot be determined from the drawings. The footings are deepest near the rear basement, and step up approaching Powell Street. The slab-on-grade for the rear basement is approximately 10 feet below the theater level slab-on-grade; one drawing indicated the rear wall of the basement is founded 10 feet below the basement slab. Efflorescence on the basement walls suggests the basement floor may be near or below the groundwater table.

On 18 January 2001, we observed the excavation of a basement expansion approximately 11 by 12 feet in plan. The basement expansion was adjacent to the existing front basement, a brick sewer was encountered the base of the excavation. The sewer had been plugged with concrete and appeared to be abandoned. In addition, a six-inch-diameter joint sewer and storm drain pipe (SS/SD pipe) was observed along the north side of the new basement. Where exposed, the top of this pipe varied from about Elevation 56.5 to 55.7 feet on the west and east ends, respectively. An active and an abandoned roof drain are located along the east side of the excavation, adjacent to the existing basement wall. An abandoned pipe was also observed buried in the subgrade soil adjacent to the roof drain line. Along the south wall of the excavation, a footing for a small structural post was exposed. The post appeared to be founded on a five-inch-thick concrete pad, measuring 3.5 by 4 feet in plan dimensions. The bottom of the pad was at about 22 inches below grade (58.2 feet, SFCD), and appeared to be founded on an older concrete footing about 3 ½ feet square. The footing was bottomed in stiff native soil.

1.2 Proposed Improvements

We understand the proposed improvements include 1) demolishing and replacing most of the existing structural components and facilities (sheetrock wall, furred column and the 2-story brick stairwells) but preserving the building façade and 2) constructing a five-story mixed use building over one level basement. The first floor will be occupied by a restaurant, retail space and residential lobby area. The remaining four levels will contain residential units. The footprint of the basement and first floor is shown on the attached Site Plan, Figure 2. The finished floor for the restaurant, retail and resident entry (Porte Cochere) will be at Elevations 62.3 feet, 64.1 feet and 65.1 feet, respectively. The floor of the below grade garage will be at Elevation 48.3 feet. Site grading will require cuts of approximately 5 feet at the rear of the building and up to 15 feet at the front. The existing alleys will remain and receive minor site grading with finished grade at 62.3 feet. No information regarding the building loads is available at the time of our investigation but we judge the load will be moderate.

In addition, the sidewalk along Columbus Avenue and Powell Street fronting the building will receive new pavement and new curb and gutter. A new PG&E vault will be constructed below sidewalk near the southeast corner of the site; we anticipate the vault will extend 4 to 6 feet below street pavement.

2.0 SCOPE OF SERVICES

In accordance with our proposal dated 5 September 2008, our scope of work included advancing one boring to supplement the previous investigation of subsurface conditions at the site. On the basis of the field investigation and a review of previous geotechnical investigations in the vicinity, we performed engineering analyses to develop recommendations and conclusions regarding:

- seismic hazards, including ground rupture, liquefaction, lateral soil displacement, and differential compaction
- appropriate seismic hazard mitigation measures, if necessary
- appropriate foundation type(s) for the proposed new construction

- design criteria for the recommended foundation type(s)
- estimates of total and differential foundation settlement
- temporary shoring system(s) and underpinning
- construction monitoring
- lateral earth pressures for design of basement walls and shoring systems
- site preparation and grading, including criteria for fill quality and compaction
- corrosion potential of near-surface soil
- 2007 California Building Code (CBC 2007) maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at one-second period, S_{M1}, adjusted for site class effects
- construction considerations

3.0 FIELD INVESTIGATION

3.1 Previous Investigation

Treadwell & Rollo performed a geotechnical investigation at the site in July 2000. The investigation included advancing two cone penetration tests (CPTs), designated CPT-1 and CPT-2, performing three dynamic penetrometer tests (DPTs), designated DPT-1 through DPT-3, and advancing two hand-augured borings, designated B-1 and B-2, at the locations shown on Figure 2.

3.1.1 Cone Penetration Tests

The CPTs were performed on 29 February 2000 by Gregg In-Situ, Inc. using portable "ram-set" CPT equipment. The ram-set CPT consists of an approximately three-foot-square piece of equipment containing a hydraulic ram. The ram-set CPT is bolted to a concrete slab at the test location; the reaction between the slab and the CPT equipment generates a maximum downward capacity of approximately 12 tons. CPT-1 was performed near the northwest building corner and was advanced to refusal at a depth of 22 feet, corresponding to Elevation 75.5 feet. CPT-2 was performed near the southeast corner of the building and was advanced to refusal at a depth of 16 feet, corresponding to Elevation 86 feet. Direct-push soil samples were collected from CPT-2 for visual classification between depths of 7 and 8 feet.

The CPTs were performed by hydraulically pushing a 1.75-inch-diameter (15 square centimeters), cone-tipped probe into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measure soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, are recorded by a computer while the test is conducted. Accumulated data are 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 and friction ratio by depth, as well as interpreted SPT N-values, soil shear strength parameters and an interpreted soil classification, are presented on Figures B-1 and B-2. The classification chart for the CPT logs is presented on Figure B-3.

3.1.2 Dynamic Penetrometer Tests

On 29 February 2000, Treadwell & Rollo personnel performed dynamic penetrometer tests DPT-1 through DPT-3 at the locations shown on Figure 2. DPT-1 and DPT-2 were advanced to depths of 12 and 16 feet below the top of the concrete sidewalk, respectively; DPT-3 was advanced to a depth of about 1-1/2 feet, where it met refusal due to an obstruction. DPT-1 was advanced adjacent to CPT-1 so that a site-specific correlation between the CPTs and DPTs could be determined. The tests were performed in general accordance with the recommendations of the penetrometer manufacturer, Triggs Technologies, Inc.[2] The penetrometer test consists of driving a 1.4-inch-diameter, cone-tipped probe into the ground with a 35-pound hammer falling 15 inches. The cone tip has a projected area of 10 square centimeters. The cone is advanced using 1.1-inch-diameter steel rods; the rods slip out of the disposable cone tip during extraction. The blows used to drive the probe are recorded in 10-centimeter (4-inch) increments, and can be converted to dynamic tip resistance (q_d) using the "Dutch Formula." These values may be converted to equivalent Standard Penetration Test (SPT) N-values for use in determining liquefaction potential. Logs of the unfactored (raw) penetrometer data are presented on Figures B-4 though B-6.

3.1.3 Hand-Augured Borings

Two hand-augured borings, designated B-1 and B-2, were advanced on 29 February 2000 at the approximate locations shown on Figure 2. The purpose of the borings was to investigate soil conditions beneath the theater-level slab and collect samples for laboratory analysis. The borings met refusal on concrete rubble or other debris at a depth of 1 to 2 feet below the existing slab. At both locations, we encountered about 1/2 to 1-1/2 feet of silty sand fill beneath the existing slab-on-grade.

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2 Triggs, Fred J., and Simpson, Paul D, *A Portable Dynamic Penetrometer for Geotechnical Investigations*, Dynamic Penetrometer Product Manual.

3.2 Current Investigation

3.2.1 Field Investigation

Prior to performing our field investigation, we obtained the required permit from San Francisco Department of Public Health (SFDPH). To check that the boring location was clear of existing utilities, we contacted Underground Service Alert (USA) and retained a private utility locator.

The approximate boring location is shown on the Site Plan, Figure 2. The boring was drilled inside the existing building. It was drilled on 8 October 2008, by Access Soil Drilling, Inc. of San Mateo, to a depth of approximately 31 feet using a portable Minuteman drill rig equipped with a 4-inch solid stem auger. Our field engineer logged the boring and obtained representative samples of the soil encountered for visual classification and laboratory testing. Log of the boring is presented in Appendix A as Figure A-1. The material encountered was classified according to the soil classification system described on Figure A-2.

Soil samples were obtained using the following split-barrel samplers:

- Sprague and Henwood (S&H) sampler with a 3.0-inch outside diameter and a 2.5-inch inside diameter, with 2.43-inch inside diameter liners
- Standard Penetration Test (SPT) sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter, without liners.

The samplers were driven with a 140-pound, safety or downhole hammer falling about 30 inches. To account for sampler size and hammer energy, the blow counts required to drive the S&H and SPT sampler the final 12 inches of an 18-inch drive were converted to approximate SPT blow counts (N-values) using a conversion factor of 0.6 and 1.0, respectively, and are presented on the boring log.

Upon completion of drilling, the hole was backfilled using cement grout, as required by the SFDPH. The soil cuttings were left on site adjacent to the boring location.

4.0 LABORATORY TESTING

Laboratory testing was performed on selected samples to determine the physical properties of the subsurface soils. We re-examined the soil samples in our office to confirm field classifications. Representative samples were delivered to a laboratory and were tested to measure moisture content, fines content (percent passing the US No. 200 sieve), Atterberg Limits. The laboratory tests are presented on the boring logs.

One representative soil sample from B-3 at 15 feet was submitted to CERCO Analytical, Inc. for a corrosivity analysis. The corrrosivity test results are attached in Appendix C and summarized in Section 7.6.

5.0 SITE AND SUBSURFACE CONDITIONS

The existing building occupies most of the site; the building has about 30 feet of frontage on Columbus Avenue and 50 feet of frontage on Powell Street. The north and south sides of the site are occupied by 7-1/2-foot-wide, concrete-paved alleyways.

The regional geology map (Figure 3) indicates that the northeastern side of the site is underlain by fill. Based on the results of the borings, CPTs and DPTs, we judge the site is blanketed by 4.5 to 10 feet of fill consisting of medium dense sand and stiff clay with varying amounts of silt and the fill thickness increases towards northeast. However, DPT-2 indicates the fill may be up to 15 feet deep at the DPT-2 location. The fill layer encountered in CPT-2 consists of mostly sandy material and appeared to be consistent with Boring B-3. However, CPT-1 encountered clayey material. The upper portions of the fill contain bricks, concrete, and other debris, the log for CPT-2 indicates the upper few feet of fill in this area may contain loose gravel or large voids.

The fill is underlain by medium stiff to very stiff sandy clay and dense to very dense silty sand that extend to the maximum depth explored (31 feet), the soil samples obtained from Boring B-3 indicated the material at depth may be residual soil[3] or completely weathered sandstone. Based on the results of borings we have reviewed at nearby sites and the geology map, weathered sandstone of the Franciscan formation may be present within 40 to 50 feet below ground surface (bgs).

Groundwater was measured in the two CPT locations (T&R, 2000) at a depth of eight feet. This depth corresponds to Elevation 51 and 56.5 feet in CPT-1 and CPT-2, respectively. Boring B-3 (T&R, 2008) encountered groundwater at 54.5 feet which is consistent with previous investigation. The results of the

3 Residual soil consists of soil that has resulted from weathering and decomposition of underlying bedrock.

groundwater measurements indicate groundwater flows towards northwest. Groundwater levels at the site are expected to fluctuate seasonally in response to rainfall. The groundwater measurements at the CPT locations made during our field investigation were made during the wet winter months and were allowed to stabilize for 1 to 2 hours; we judge the groundwater levels measured are near the high groundwater level at the site.

If a single design groundwater level is desired, a groundwater level of Elevation 56.5 feet should be used.

6.0 SEISMIC CONSIDERATIONS

Regional seismicity and faulting, fault rupture and associated geologic hazards are discussed in this section.

6.1 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, Hayward, San Gregorio, and Calaveras Faults. These and other faults of the region are shown on Figure 4. For the active faults within 40 kilometers, the distance from the site and estimated mean characteristic Moment magnitude[4] [Working Group on California Earthquake Probabilities (WGCEP) (2003) and Cao et al. (2003)] are summarized in Table 1.

4 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.

TABLE 1
Regional Faults and Seismicity

Fault Name	Distance (km)	Direction from Site	Mean Characteristic or Maximum Moment Magnitude
San Andreas – 1906 Rupture	12.9	West	7.90
San Andreas – Peninsula	12.9	West	7.15
San Andreas – North Coast South	14	West	7.45
North Hayward	16	East	6.5
Total Hayward	16	East	6.9
Total Hayward-Rodgers Creek	16	East	7.3
South Hayward	18	East	6.7
Northern San Gregorio	18	West	7.2
Total San Gregorio	18	West	7.4
Rodgers Creek	32	North	7.0
Mt Diablo – MTD	34	East	6.7
Total Calaveras	35	East	6.9
Concord/Green Valley	38	East	6.7
Point Reyes	40	West	6.8
Monte Vista-Shannon	43	Southeast	6.8
West Napa	43	Northeast	6.5

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 470 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, in the Santa Cruz Mountains with an M_w of 6.9, approximately 97 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 2002 the Working Group on California Earthquake Probabilities (WGCEP 2003) at the U.S. Geologic Survey (USGS) predicted a 62 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2031. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2003) Estimates of 30-Year Probability (2002 to 2031) of a Magnitude 6.7 or Greater Earthquake

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

6.2 Liquefaction, Lateral Spreading and Differential Compaction

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[5], lateral spreading[6], post-liquefaction settlement[7], and cyclic

5 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.

6 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial soil is typically displaced in "blocks" that are transported downslope or in the direction of a free face by earthquake and gravitational forces.

7 Post-liquefaction settlement is a phenomenon in which a previously liquefied sand layer settles into a denser soil arrangement after dissipation of pore water pressures.

differential compaction[8]. We used the results of the borings and CPTs to evaluate the potential of these phenomena occurring at the project site.

The project site is within an area designated by the California Geologic Survey (CGS) (formerly known as California Division of Mines and Geology) as a zone of potential liquefaction (State of California Seismic Hazard Zones – San Francisco North Quadrangle, 17 November 2000) as shown on Figure 6. Consequently, we performed an analysis of the liquefaction potential at the site.

The liquefaction studies were performed in accordance with the methodology presented in the publication titled Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, prepared by the National Center for Earthquake Engineering Research (NCEER, 1997) and recent studies by Seed et al. (2003). Our evaluation was based on a Moment Magnitude, $M_w = 7.9$ earthquake with a peak ground acceleration, $a_{max} = 0.40g$ from the designed mapped values of spectral accelerations.

Based on the site specific information obtained from the boring, CPTs and DPTs, and results of our engineering analyses, we conclude the soil encountered beneath the proposed structure has sufficient strength to resist liquefaction during a major earthquake. Therefore, we judge the risk of other geologic hazards associated with liquefaction, including lateral spreading, to be low.

The fill encountered at the site is generally clayey; however, possible voids or loose materials were encountered above a depth of 3-1/2 feet in CPT-2. Therefore, we conclude some loose sandy or gravelly fill may be present beneath portions of the site. In addition, based on Boring B-3, a medium dense clayey sand layer of approximately six feet thick is present below the existing slab. We anticipate total settlement on the order of 1 to 2 inches of settlement may occur in isolated locations near sidewalk and the alleyways due to cyclic densification during a large earthquake on one of the nearby faults.

8 Cyclic soil densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, resulting in ground surface settlement.

7.0 DISCUSSION AND CONCLUSIONS

We conclude that from a geotechnical engineering standpoint the site can be developed as proposed, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns for the project site include:

- selection of an appropriate foundation system for the support of the planned structure
- presence of groundwater table above basement excavation
- presence of undocumented fill
- maintaining vertical and lateral capacities of existing column footings at the perimeter during basement excavation
- construction related issues include:
  - Depth of excavation for the below grade level and need to maintain lateral support during excavation.
  - Impact of surcharge loads from adjacent structures on temporary shoring and permanent basement walls.
  - Dewatering during basement excavation

These issues and their impact on the design and construction of the planned structure are discussed in the following sections.

7.1 Foundations and Settlement

The primary geotechnical issue is the presence of shallow groundwater. The groundwater will impose hydrostatic uplift on the basement floor and lateral pressure on the basement walls. Groundwater can also be problematic during construction of the basement.

On the basis of our investigation, we conclude the proposed basement walls and columns can be supported on conventional spread footings with slab-on-grade or a mat foundation. However, with a design groundwater level at 8 feet above the proposed basement level, the hydrostatic pressure may exceed the distributed building loads. Therefore, tension elements, such as ground anchors, may be required. A mat foundation of approximately 4 to 5 feet thick should be sufficient to resist hydrostatic uplift but it may not be practical. We have provided herein recommendations for design of both spread footings and a mat so a cost comparison can be made between the two options.

Either footings or a mat should be bottomed in very stiff to hard sandy clay and/or very dense silty sand. Based on the allowable bearing pressures presented later in this report, we estimate total settlement under moderate building loads will be ½ inch, with differential settlement on the order of ¼ inch or less in 25 feet; for the mat foundation, total settlement of about ¼ inch may occur depending upon applied mat pressure at different locations. We estimate differential settlement of a mat between adjacent columns should be less than ¼ inch.

Furthermore, if seismic building loads result in high uplift forces, tiedowns may be required for the mat foundation.

7.2 Undocumented Fill

Based on our review of the regional geology map, it appeared that approximately half of the site is underlain by fill. The borings, CPTs and DPTs results indicated that the northeastern side of the site may be underlain by up to 15 feet of medium dense clayey sand fill. We understand most of the fill will be removed during excavation for the proposed basement and the foundations will be bottomed in native material. However, the design of the temporary shoring, underpinning piers and basement walls is subject to the thickness of the fill. Due to limited information available at this time to precisely determine the fill thickness across the site, we assume the fill to be 15 feet thick.

7.3 Groundwater

Below grade walls, mat foundation or floor slab will extend about 8 feet below the groundwater level and should be designed to resist lateral and uplift hydrostatic pressures using a design groundwater level at Elevation 56.5 feet. Below grade walls and slabs should be waterproofed.

7.4 Tiedown Anchors

We judge the use of tiedown anchors is the most suitable method to resist temporary or permanent uplift if building loads are insufficient. Tiedown anchors consist of relatively small-diameter, drilled, concrete- or grout-filled shafts with steel bars embedded in the concrete or grout. The tiedown anchors develop their resistance from friction between the sides of the shaft and the surrounding soil.

Because portable drill and CPT rigs were used for site exploration, the depth explored may not be sufficient to develop a profile for high capacity anchors. We judge the tiedown anchors will extend into hard sandy clay and very dense silty sand as a minimum. The lower portion of the tiedown may be embedded in sandstone or claystone of Franciscan bedrock if they extend beyond about Elevation 30 feet. Recommendations for design of tiedown anchors are presented in a subsequent section of this report.

7.5 Construction Considerations

7.5.1 Demolition

We anticipate a significant quantity of crushed concrete and other debris will be generated during demolition of the existing concrete slabs-on-grade and basements. This material should be removed or stockpiled for later use, if approved by the architect and recommendations presented in this report for gradation of the material are incorporated into the project specifications.

7.5.2 Excavation

The results of the CPTs indicate the materials to be excavated will include clay fill with sand and silt as well as silty sand. Based on the results of CPT-2, we anticipate loose sand with gravel and some voids may be present in the fill, especially in the upper few feet of the excavation. We anticipate the site can be excavated with conventional equipment.

We anticipate the subgrade exposed at the base of the excavation will be wet and subject to disturbance under equipment loads and construction workers. To protect the base of the excavation and to provide a relatively smooth surface for waterproofing, a concrete mud slab should be cast on the subgrade for the mat or slab-on-grade (if footings are used) prior to placing reinforcing steel. The mud slab will provide a firm working surface on which to place reinforcing steel and waterproofing.

7.5.3 Underpinning

We anticipate underpinning will be required around the perimeter footings to be saved except possibly at the existing rear basement wall where footings supposedly extend below the proposed basement level. However, information is not sufficient at this time to indicate if perimeter columns are supported on individual footings or on continuous footings. We recommend test pits be excavated to verify the footing dimensions and conditions before construction. If the columns are supported on individual spread footings, underpinning piers should be constructed in sequence underneath the footings without undermining the footings.

Typical underpinning techniques include hand excavated piers and slant-drilled piles. The selection, design, construction, and performance of the underpinning system should be the responsibility of the contractor. The contractor should be familiar with applicable local, state, and federal regulations for the current OSHA Excavation and Trench Safety Standards. The contractor should be solely responsible for the design of underpinning. We should review the design and observe the installation of these systems to confirm that our recommendations have been properly incorporated, and that conditions in the field are as we assumed when determining the underpinning parameters.

If underpinning piers are used, they should extend 2 feet below the planned excavation depth into competent material. Settlement of the piers should be minimal if no significant load will be imposed to the existing footings. However, the existing footings should be monitored during excavation and construction of the basement to verify that unacceptable vertical and lateral movement does not occur.

Beside hand-excavated underpinning piers, conventional slant-drilled piers have been used to temporarily support buildings on other projects where similar soil conditions are present. Slant-drilled piers are constructed by drilling a cylindrical hole within a hand excavated access pit.

The slant-drilled pier holes are typically two feet in diameter and once drilled, a steel beam is lowered in the shaft, underneath the existing foundation and filled with concrete. Because the site is underlain by fill consisting of loose to medium dense sand with varying amounts of fines, caving or excessive lateral deformations toward the shaft cavity can occur. Therefore, either casing or slurry should be used during drilling. Slant-drilled piers should be designed to resist lateral movements as well as support vertical loads.

7.5.4 Shoring

?

The excavation for the proposed basement and foundation varies in depth between approximately 6 (near the rear of the building) and 19 feet (main lobby area). Because there is insufficient space to slope the sides of the excavation, shoring will be required. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

- protection of surrounding improvements, including streets, utilities, and adjacent structures
- proper construction of the shoring system to reduce potential for ground movement

- cost
- constructability due to high groundwater level

On the basis of our findings and our experience with sites having similar soil conditions, we evaluated the use of 1) soil nails, 2) underpinning piers with lagging in between and 3) mixed-in-place soil/cement walls. These are discussed in the following sections:

Soil nailing is a method of shoring using grouted reinforcing bars (nails), which are typically spaced between 4 and 6 feet, horizontally and vertically. Facing, usually consisting of a four-inch-thick layer of shotcrete reinforced with wire mesh, stabilizes the excavation face between nails. Soil nails are passive (not tensioned) soil reinforcements that are placed in sufficient quantities within the soil to create a coherent gravity mass. Horizontal displacements may be greater than those associated with tieback construction, and therefore, soil nail shoring may not be feasible adjacent to the existing structures.

We judge underpinning piers with timber lagging is feasible but would require extensive dewatering before the piers can be installed. Additionally, it would be difficult to install lagging in areas where perched water is encountered. Perched water can transport soil through the lagging resulting in the creation of voids behind the lagging.

Mixed-in-place, closely spaced, soil/cement columns would likely be the most watertight shoring systems and thus require the least dewatering. In addition, mixed-in-place soil/cement walls would be relatively rigid and could significantly limit lateral deflections and ground subsidence related to the excavation. The columns could be slant drilled beneath footings that need to be underpinned. The disadvantages of these systems are cost and space requirements. Unless used as underpinning, these systems will require a width of about three feet around the perimeter of the site.

How low?

Lateral resistance may be mobilized by extending the shoring below the bottom of the excavation and using internal braces or tiebacks. However, tiebacks will have relatively low capacities in the fill. Because the depth of excavation is relatively shallow, tiebacks with low capacities may still be feasible. If tiebacks are used to provide lateral support for the shoring, care should be taken to locate utilities and other possible underground obstructions prior to installation. Information regarding the below-grade portion of the adjacent structures and their foundation is not available at this time; encroachment permits will be required to install tiebacks below existing adjacent buildings. If encroachment permit cannot be obtained, internal bracing is an option.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. However, the shoring should be designed by a structural engineer knowledgeable in this type of construction, and we should review the design to confirm it incorporates our concerns regarding the shoring.

7.5.5 Excavation Monitoring

During excavation, the shoring system may yield and deform, which could cause surrounding improvements to settle and move laterally. The magnitude of shoring movements and resulting ground deformations are difficult to estimate because they depend on many factors, such as soil conditions, type of shoring system and the contractor's skill in installing the shoring. Considering the excavation will extend up to 15 feet into very dense silty sand/ hard clay, we anticipate the horizontal and vertical deformation for a properly installed tied back / braced shoring system may be on the order of 1 to 1½ inch. Potential deformations should be estimated by the shoring designer.

A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements and the existing building. The contractor should install surveying points to monitor the movement of shoring and settlement of adjacent structures and existing building during excavation. The monitoring should provide timely data which can be used to modify the shoring system if needed.

7.5.6 Dewatering

The proposed excavation will extend up to 11 feet below the design groundwater level. Seepage of water through the walls of the excavation may cause erosion or sloughing of the underpinning pits or softening of the basement subgrade. Because we anticipate the fill encountered will be variable (based on CPT-1 and CPT-2 results), the amount of seepage into the excavation is difficult to predict. To reduce the amount of water seeping into the excavation, temporary dewatering will be required.

The groundwater level at the site should be lowered to a depth of at least three feet below the bottom of the planned maximum excavations and maintained at this level until sufficient weight and/or uplift capacity is available to resist the hydrostatic uplift forces on the bottom of the structure. The project structural engineer should evaluate when the dewatering can be stopped.

The efficiency of the dewatering system will depend to some extent on the type of shoring system used. For example, a soil/cement mix wall would likely be relatively more water-tight than soldier beam/underpinning piers with timber lagging, thus requiring less dewatering.

The selection and design of the dewatering system should be the responsibility of the contractor. The contractor will need to obtain a dewatering permit from the City and County of San Francisco (City) for discharging water into the local municipal waste water collection system. The dewatering permit requires chemical testing for characterizing the water to be discharged. No chemical testing of the groundwater has been performed by us to-date. Currently there is a fee for disposing of construction generated water in the City's waste water collection system.

Excessive site dewatering could result in subsidence of the immediate area. Therefore, adjacent buildings and streets should be monitored for vertical movement, while dewatering is in progress.

7.6 Corrosivity Evaluation

CERCO Analytical of Pleasanton, California performed a corrosivity test on a representative sample at a depth of 15 feet below the existing slab-on-grade in Boring B-3. Corrosion potential was determined based on the nominal resistivity measurement (100 percent saturation), electrical conductivity, chloride ion concentration, sulfate ion concentration, soluble sulfide concentration, pH, and redox potential.

The results of corrosivity testing as well as a summary describing the corrosion characteristics of the near surface soil and protection recommendations are included in Appendix C. We recommend a corrosion expert be consulted during the design phase for the most economical and effective corrosion protection.

8.0 RECOMMENDATIONS

In accordance with our scope of work, recommendations for site preparation, foundations, temporary shoring, tiedown, underpinning, below grade walls and slabs, dewatering, and seismic design are presented in the following sections.

8.1 Site Excavation, Subgrade Preparation and Backfill

We anticipate the soil removed from the basement excavation will generally consist of a mix of silty sand and sandy clay with varying amounts of sand, silt, and gravel. We judge the soil excavated from the basement can be reused if approved by the project architect, and provided it contains no rocks, lumps or rubble larger than three inches in greatest dimension and can be compacted to the desired degree of compaction. Remnants of underground utilities (e.g. existing brick sewer line), building debris, and other obstructions may be encountered during excavation. Soil excavated from below the groundwater level will likely be wet and require drying before it can be used as backfill. However, because of the limited space available at the site, we judge it may not be feasible to stockpile the soil removed from the basement for reuse as backfill for other site improvements, and imported soil may be required.

All fill placed at the site, including the excavated on-site material or imported fill, should contain no rocks or lumps larger than three inches in greatest dimension. Imported fill should have a plasticity index (PI) less than 12, a liquid limit less than 40, and be free of organic material. Samples of imported material should be submitted to the geotechnical engineer for approval and testing at least 72 hours before delivery to the site. Fill should be placed in lifts no greater than eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[9]. Imported soil with less than 10 percent fines (material passing the No. 200 sieve) should be compacted to at least 95 percent relative compaction. For work performed in City and County of San Francisco streets, the upper three feet of subgrade and the aggregate base in pavement areas should be compacted to at least 95 percent relative compaction.

Demolition debris can be used as backfill provided it is free of bricks, organic material, wood, or other deleterious material. Concrete or other debris used as backfill should be crushed to no greater than four inches in greatest dimension, with no more than 50 percent of the debris by weight greater than two inches in greatest dimension. Demolition debris should be mechanically compacted in lifts no greater than 12 inches in loose thickness. Where demolition material is to be placed as backfill, the lower five feet of the excavation should be backfilled with engineered fill meeting the requirements presented for general site fill. The engineered fill should be compacted to at least 95 percent relative compaction over the entire five-foot depth.

The soil exposed at the subgrade should be graded to produce a level, non-yielding surface. To provide a smooth surface a layer of gravel or lean concrete may be used. Because the proposed foundation extends below the groundwater level, waterproofing the base of the mat (if used) or floor slab will be required. Waterproofing should be placed in accordance with the manufacturer's specifications. The waterproofing should be covered by a mud slab (a layer of low strength concrete). The mud slab should reduce the potential for disturbing the underlying subgrade and protect the waterproofing from damage during mat construction. The mud slab should also provide a firm, smooth surface on which to place the reinforcing steel for the mat or floor slab. A waterproofing specialist should design the waterproofing system.

9 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-91 laboratory compaction procedure.

We should check the subgrade prior to placing the mud slab or waterproofing for proper bearing. Subgrade areas where loose/soft material is encountered should be removed and replaced with lean concrete. Overexcavation may be required on the northeast corner of the site due to the presence of fill and should be replaced with lean concrete. Where temporary slopes are to be cut, we recommend that they be no steeper than 1.5:1 (horizontal to vertical) in fill and 1:1 in native material.

Areas to receive fill should be scarified to a depth of at least six inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to between 88 and 93 percent relative compaction.

Fill and backfill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 95 percent relative compaction.

Backfill for utility trenches and other excavations is also considered fill, and it should be compacted according to the recommendations presented above. If imported clean sand or gravel (material with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. The materials excavated from the trenches can be reused to backfill those trenches, provided they can be compacted to the desired degree of compaction. Material excavated from utility trenches will likely be wet and require drying before it can be used as backfill. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

8.2 Foundations

8.2.1 Footings

Conventional spread footings can be used to support the proposed building at basement level. We understand the existing perimeter column footings will be underpinned during excavation and bottomed in very dense sand or hard clay. The existing and new footings can be designed with an allowable bearing pressure of 6,500psf (pounds per square foot) for dead plus live load. These pressures may be increased by one-third for total load conditions, including wind and seismic forces.

Continuous perimeter footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches square. Interior footings and continuous footings should be bottomed at least 2 feet below the lowest adjacent soil subgrade.

The foundation excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If water seeps into the base of the footings, they should be covered with a three-inch mud slab. We should check foundation excavations prior to placement of reinforcing steel.

8.2.2 Mat Foundation

As an alternative, a mat foundation bearing on very dense silty sand or hard clay can be used to support the proposed structure. The average, allowable dead plus live load bearing pressure for the mat should not exceed 6,500 psf. For total loads, including wind or seismic loads, the allowable bearing pressures can be increased by one third.

For design of the mat using a subgrade modulus method, we recommend using a subgrade modulus of 90 pounds per cubic inch (pci). This value is for a maximum bearing pressure of 6,500 psf and ½ inch of settlement. This value can be increased by 1/3 for total loads including seismic forces.

8.2.3 Lateral Resistance

Lateral forces can be resisted by a combination of passive pressure on the vertical faces of the foundations and below-grade walls, provided the walls are appropriately designed for the pressures. Additional resistance can be mobilized as friction along the base footing or mat. Passive resistance may be calculated using an equivalent fluid weight of 150 pcf for the basement footings or mat foundation below the groundwater table. The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using base friction coefficients of 0.30 for the basement footings. These values include a factor of safety of at least 1.5. Frictional resistance should be verified once the type of waterproofing has been determined.

8.3 Underpinning

Where hand-excavated piers are used to underpin the foundations of the façade to be saved, they should be designed to gain support through end bearing using an allowable bearing pressure of 6,500 psf. Piers should be bottomed at least 2 feet below the bottom of the planned excavation. The soil between the piers should be retained by shotcrete facing or wood lagging. If wood lagging is used, any voids behind the lagging created during excavation should be filled with grout immediately.

The bottom of the piers should be free of standing water, debris, and disturbed materials prior to placing concrete. We should check the excavation prior to placement of reinforcing steel to confirm the exposed soil is suitable to support the design bearing pressures. If loose or soft soil or undesirable material is encountered, it should be removed and the overexcavation backfilled with lean or structural concrete to the bottom of the pier.

Slant-drilled piers have been used on other projects where similar soil conditions are present. Slant drilled piers would gain their capacity in friction and should extend at least 15 feet beneath the proposed excavation. If slant-drilled piers are used in lieu of hand-excavated piers, we recommend they be designed using a skin friction of 400 psf in medium dense fill and 800 psf in the very dense silty sand and very stiff sandy clay. The soil between piers can be retained by additional soil-cement columns or timber lagging.

Underpinning piers should be design to resist lateral pressure (restrained condition) presented in Table 3 of Section 8.5. Lateral pressure from the retained soil and building loads can be resisted by the passive resistance of the piers, and if necessary, by tiebacks. Figure 7 presents our design recommendations for passive resistance and tiebacks. Passive resistance acting on the hand-excavated and slant-drilled piers should be assumed to act over one pier width and twice pier width, respectively.

Furthermore, a surcharge pressure should be added to the design pressures for both hand-excavated and slant-drilled piers whenever an adjacent footing falls above an imaginary line extending upward at a 45-degree angle from the base of the proposed piers. The lateral pressure due to individual footings is complex and should be determined on a case-by-case basis. We can provide parameters for building surcharge pressures once the adjacent building loads and foundation type are known.

The underpinning piers will be permanent structures, the structural engineer should therefore evaluate if the vertical and lateral capacity of the underpinning piers would be sufficient for the existing footings that will be used for the proposed improvements if new load is added.

8.4 Basement Floor

If individual footings will be used as foundations for the proposed improvements, the floor slab at the basement resting on competent soil can be designed as slabs-on-grade. However, to adequately resist uplift pressures, tiedown anchors may be required. Alternatively, the top of a mat foundation may be used as the lowest basement floor, or a thin layer of concrete (topping slab) may be placed directly above the mat to provide a smooth wearing surface.

Permanent waterproofing will be required beneath the proposed basement mat foundation/footings and floor slab and along the basement walls due to the planned construction below the groundwater table. We recommend a waterproofing consultant be retained to determine the most appropriate system for this project. Installation of waterproofing should be performed in accordance with the manufacturer's recommendations. The acceptability of pouring a mud slab on the foundation subgrade prior to application of the waterproofing membrane should be checked with the manufacturer.

8.5 Permanent Basement Walls

We recommend all basement walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. In addition, the basement wall should also be designed to resist surcharging from the footings of the adjacent properties.

Because the project site is in a seismically active area, we recommend the basement walls be designed to resist additional loads associated with seismic forces. We recommend designing the walls to resist the more critical condition of either the at-rest pressure, or the active pressure plus a seismic pressure increment corresponding to a rectangular distribution of 10H (in psf) where H is the height of the wall in feet. Basement walls should be designed for the pressures presented in Table 3, where H is the height of the wall in feet. Additional surcharge loads from foundations supporting the adjacent structures should be included in the design. We can provide the additional lateral pressure due to the building surcharge when we receive the information.

TABLE 3
Lateral Earth Pressures

Soil Type and Conditions	Static Conditions		Seismic Conditions
	Unrestrained Walls	Restrained Walls	
Fill above water table	40 pcf	60 pcf	40 pcf + 10H psf
Fill below water table	85 pcf	90 pcf	85 pcf + 10H psf
Native Soil below the water table	80 pcf	85 pcf	80 pcf + 10H psf

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

The lateral earth pressures given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend down to the design groundwater elevation (10 feet below the existing ground surface). We should check the manufacturer's specifications regarding the proposed prefabrication drainage panel material to confirm it is appropriate for its intended use.

Another acceptable alternative is to backdrain the wall with Caltrans Class 2 permeable material at least one foot wide extending down to the base of the wall. Filter fabric should be placed between the gravel drain and the natural ground. This system is usually not used where shoring is used as the backside form for the walls.

To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls unless the manufacturer of the waterproofing directs otherwise.

Wall backfill should be compacted to at least 90 percent relative compaction using light compaction equipment. Wall backfill with less than 10 percent fines, or deeper than five feet, should be compacted to at least 95 percent relative compaction for its entirety. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

8.6 Shoring

Shoring may be cantilevered where the depth of excavation permits, and either tied back or internally braced where necessary. Tie back or braced shoring should be designed to resist the pressures presented on Figure 7. Cantilevered shoring should be designed using the pressures presented on Figure 8. The design pressures are based on the assumption the site will be dewatered to 3 feet below the bottom of the excavation during construction.

If traffic will occur within 10 feet of the shoring depth, a uniform surcharge load of 100 psf should be added to the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth in feet. Construction equipment should not be allowed within 15 feet of the edge of the excavation, unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be determined after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated by the shoring designer to check if they are acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement and possible damage to adjacent structures and streets.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on its design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

8.6.1 Tieback Design Criteria and Installation Procedure

Design criteria for tiebacks are presented on Figure 7. As shown on the figure, tiebacks should derive their load-carrying capacity from the rock behind an imaginary line sloping upward from a point 0.2H feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the excavation depth in feet.

Allowable capacities of the tiebacks will depend upon the drilling method, tieback-hole diameter, grout pressure, and workmanship. Because specialty contractors who install the tiebacks use different types of installation procedures, the skin friction of the tieback will vary. For estimating purposes, we recommend using the allowable skin friction values presented on Figure 7. These values are for pressure-grouted tiebacks and include a factor of safety of 1.5. Higher allowable skin friction values may be used if confirmed with pre-production performance tests. All tiebacks should have a minimum bonded and unbonded length of 15 feet.

Solid flight augers should not be used for tieback installation. We recommend a smooth cased tieback installation method (such as a Klemm type rig) be used.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with his installation method. The computed bond length should be confirmed by a performance- and proof-testing program under our observation. Tieback testing should be performed after grout has been allowed to set up to obtain a compressive strength of at least 3,000 pounds per square inch (psi) at 28 days. Replacement tiebacks should be installed for tiebacks that fail the load test.

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. Recommendations for tieback testing are presented in Section 8.4. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer at the expense of the contractor.

8.6.2 Internal Bracing

As discussed in Section 7.5.4, tiebacks may not be feasible if encroachment permits cannot be obtained. Internal bracing such as horizontal struts or inclined rakers can be used. The lateral earth pressure diagram presented on Figure 7 can be used for internal bracing. These pressures are based on the assumption the interior and exterior of the excavation will be dewatered to 3 feet below the bottom of excavation. Control of ground movement will depend as much on the timeliness of installation of lateral restraint / raker as on the design. Internal bracing should be installed as close to the time of excavation as possible. Excavation should not proceed below a level of bracing until the bracing at that level has been installed and locked off. Jacking (preloading) of the bracing against the sides of the excavation can reduce movement of the shoring.

The contractor or his designer should be responsible for determining the type and size of bracing/ rakers required to resist the recommended pressures. We should review the shoring plans and a representative from our office should observe the installation of the shoring system.

8.7 Tiedown Anchors

Tiedown anchors may be used to provide uplift resistance across portions of the mat or slab-on-grade where the uplift pressure will exceed the anticipated building loads. The hydrostatic uplift load should be computed using a design groundwater at Elevation 56.5 feet. Tiedown anchors typically consist of relatively small-diameter, drilled, concrete or grout-filled shafts; high strength bars with a minimum stressing length (free length) of 15 feet and a bond length of 15 feet should be used as tensile reinforcement in the anchors. The anchors will develop their uplift resistance from friction between the sides of the shaft and the surrounding rock.

Tiedown anchors should be spaced at least four-shaft diameters center-to-center, with a minimum of four feet and should not be spaced more than the width that the slabs-on-grade can resist the hydrostatic uplift. The ultimate bond strength between the anchor and soil will depend on the installation procedure. For planning purposes, however, we recommend using an allowable skin friction of 800 psf. The actual bond strength should be determined by the shoring contractor or his designer. Higher values may be obtained depending upon the techniques employed by the contractor and the results of pullout tests. The tiedown anchors will be installed below the water table; therefore, the contractor should be prepared to use an auger-cast system or to case the holes if caving soil is encountered.

Special attention should be given to waterproofing the connections between the tiedown anchors and the mat. Because the tiedowns will be permanent, encapsulated tendons should be used (double corrosion protection). Corrosion protection requirements regarding the bonded and unbonded length, and stressing anchorage are outlined below:

- encapsulations used to provide an additional corrosion protection layer over the tendon bond length should consist of a grout filled, corrugated plastic sheathing, or grout filled deformed steel tube; the prestressing steel can be grouted inside the encapsulation prior to inserting the tendon into the drill hole or after the tendon has been placed; centralizers or grouting techniques should provide a minimum of ½ inch of grout cover over the encapsulation
- a sheath filled with corrosion inhibiting compound or grout, or a heat shrinkable tube internally coated with a mastic compound should be used to provide corrosion protection of the unbonded length
- the trumpet should be sealed to the bearing plate and overlap the unbonded length corrosion protection by at least four inches; it should be completely filled with a corrosion inhibiting compound or grout
- all stressing anchorages permanently exposed to the atmosphere should be grout-filled; stressing anchorages encased with at least two inches of concrete do not require a cover
- if water is present in the shaft, concrete should be placed using a tremie system. The first two production tiedowns and two percent of the remaining tiedowns should be performance-tested to 1.5 times the design load. All other tiedowns should be proof-tested to 1.5 times the design load. The anchors should be tested as recommended in Section 8.8. After testing, all anchors should be loaded and locked off to a portion of their design load as determined by the structural engineer and indicated on the structural drawings and/or specifications.

8.8 Tieback and Tiedown Anchor Testing

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

8.8.1 Performance Tests

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

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

The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiedowns that should be creep tested. Creep tests should be performed in accordance with the latest edition of "Recommendations for Prestressed Rock and Soil Anchors" of Post-Tensioning Institute.

8.8.2 Proof Tests

A proof test is a simple test that is used to measure the total movement of the tieback/tiedown during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the load should be maintained and the observation is continued until the creep rate can be determined. The proof test results should be compared to the performance test results. Any significant variation from the performance test results will require performance testing on the tieback/tiedown.

We should evaluate the results of performance and proof tests to check that the tiebacks/tiedowns can resist the design load. For any tiebacks/tiedowns that fail to meet the performance and proof testing requirements, additional tiebacks/tiedowns should be installed to compensate for the deficiency, as required by the shoring designer and project structural engineer.

8.8.3 Acceptance Criteria

The geotechnical engineer should evaluate the tiebacks/tiedowns test results and determine whether the tiebacks/tiedowns are acceptable. A performance- or proof-tested tiebacks/tiedowns with a ten-minute hold is acceptable if the tiebacks/tiedowns carry the maximum test load with less than 0.04/0.08 inch movement, respectively, between one and ten minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. In addition the total deflection of the tiedowns should not exceed ¾ inch at the design load.

A performance- or proof-tested tiebacks/tiedowns with a 60-minute hold is acceptable tiebacks/tiedowns movement between 6- and 60-minute reading is less than 0.08 inch, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

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

8.9 Dewatering

The collected water from dewatering should be directed to sumps where it should be pumped into the City's municipal waste water collection system. The dewatering system should be in operation until sufficient building weight and/or uplift capacity are available to resist the hydrostatic uplift pressure. Elevator and sump pits should be locally dewatered.

If the pumped groundwater is disposed of in the City storm drain, it is likely the discharge will have to be metered. The volume of water discharged should be monitored and a record of the amount be submitted to the owner.

Adjacent site improvements should be monitored for vertical movement caused by the dewatering. Furthermore, groundwater levels outside the excavation should be monitored through wells while dewatering is in progress. Should settlement or groundwater drawdown which is deemed potentially damaging to surrounding improvements be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells, or alter the dewatering program to reduce the drawdown to an acceptable level.

8.10 Temporary Slopes

Where space permits, the sides of the excavation may be sloped. Based on the results of our field investigation, we judge the soil exposed in the slope cuts will generally consist of clay with varying amounts of sand and silt. We recommend temporary slopes not exceed inclinations of 1:1 (horizontal to vertical) where cohesive clay and silt is exposed in the slope cut. If loose sand or gravel is encountered in slope, it may be necessary to flatten to slope to an inclination of 1.5:1

Contractors should be familiar with applicable local, state, and federal regulations for temporary sloping, including the current OSHA Excavation and Trench Safety Standards. The contractor should be solely responsible for the design of temporary construction slopes. The design of the temporary slopes should be reviewed by the geotechnical engineer, and construction of temporary slopes should be observed by the geotechnical engineer.

8.11 Utilities and Utilities Trenches

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench excavations should be shored and braced to prevent cave-ins and/or in accordance with safety regulations. Where trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in lifts of eight inches or less, moisture-conditioned to near the optimum moisture content, and compacted to at least 90 percent relative compaction. Beneath streets and sidewalks, the upper three feet of fill should be compacted to at least 95 percent relative compaction. If fill with less than 10 percent fines is used, the entire depth of the fill should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements resulting in damage to the pavement section.

8.12 Concrete Flatwork

Concrete sidewalks including proposed alleyways and other exterior flatwork should be underlain by at least four inches of Class 2 aggregate base conforming to the most recent version of the Caltrans

Standard Specifications. Prior to placement of aggregate base, the soil subgrade should be scarified to a depth of six inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. The aggregate base should also be compacted to at least 90 percent relative compaction.

8.13 Seismic Design

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

- Maximum Considered Earthquake (MCE) S_s and S_1 of 1.50g and 0.615g, respectively.
- Site Class C
- F_a and F_v of 1.0 and 1.3, respectively
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.50g and 0.80g, respectively.

Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.00g and 0.533g, respectively.

9.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, Treadwell & Rollo, Inc. should review the project plans and specifications to check that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation; installation tiedown anchors, underpinning; excavation for the proposed basement and foundations; installation of building foundations; and placement and compaction of fill and backfill. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and 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

2007 California Building Code, Title 24, Part 2, Volumes 1 and 2.

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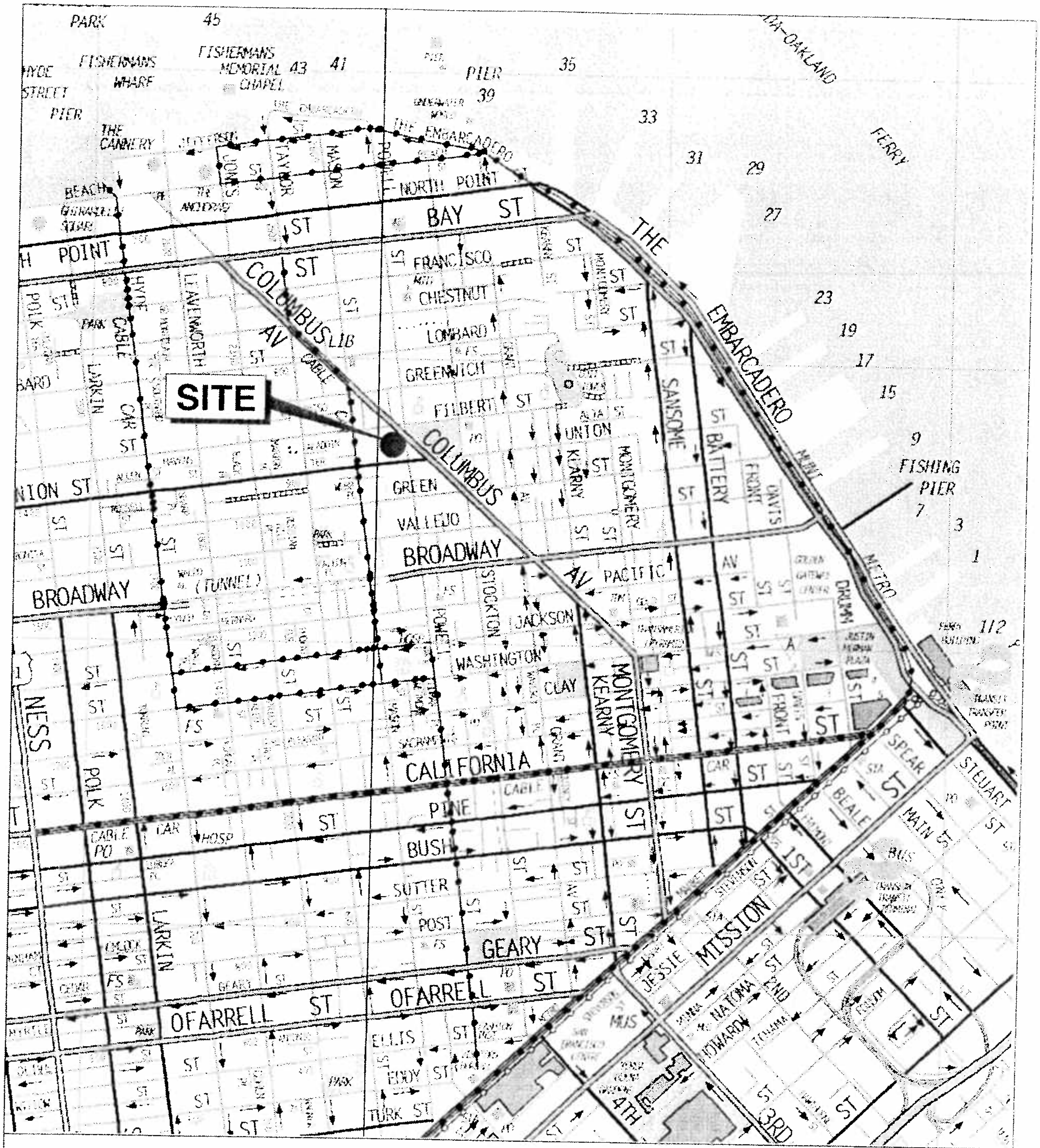
FIGURES



Base map: The Thomas Guide
San Francisco County
1999

0 1/4 1/2 Mile

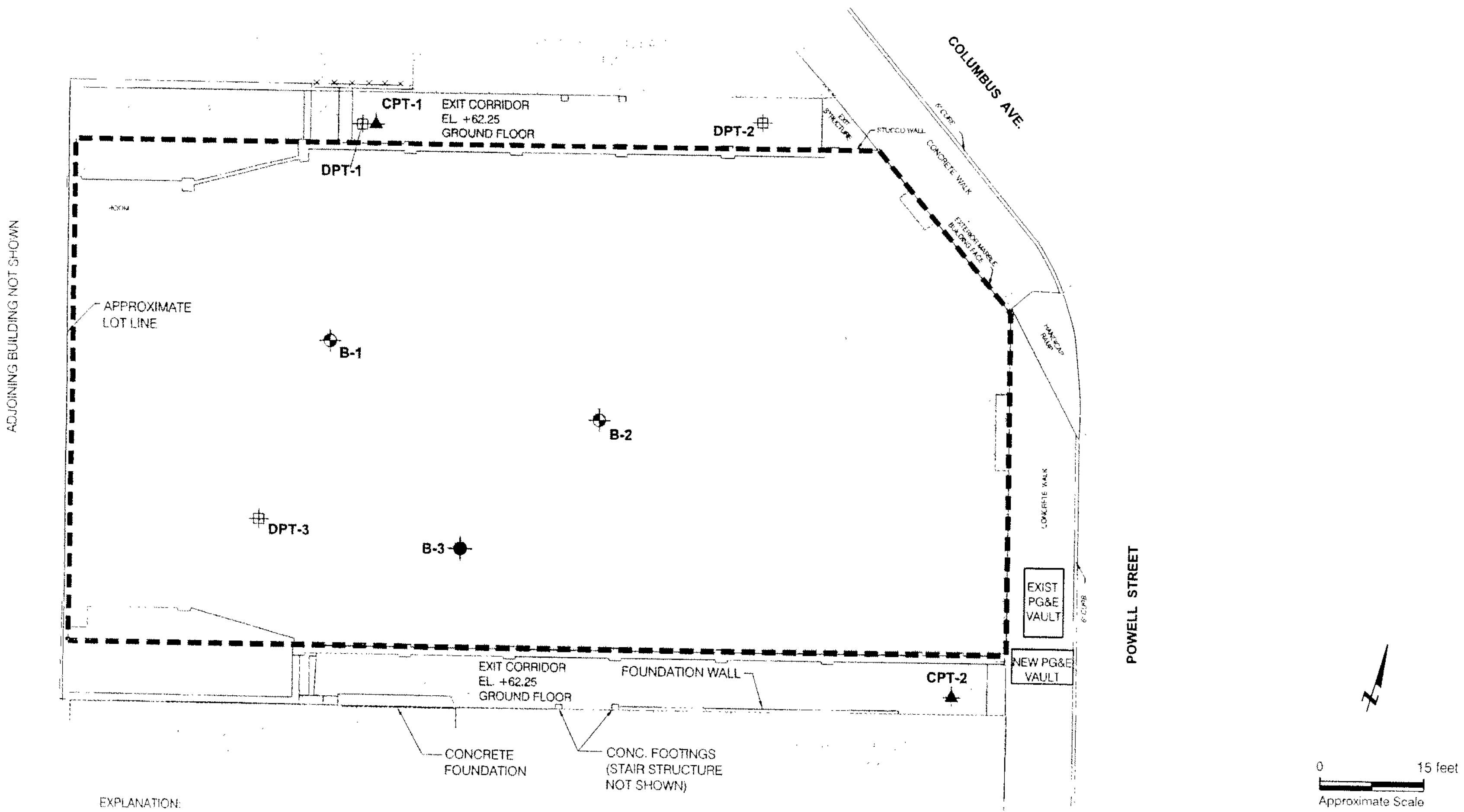
Approximate scale

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

Treadwell&Rollo

SITE LOCATION MAP

Date 11/13/08 | Project No. 2766.03 | Figure 1



EXPLANATION:

B-3 — Approximate location of boring by Treadwell & Rollo, Inc., October 2008

B-1 — Approximate location of hand-augered boring by Treadwell & Rollo, February 2000

CPT-1 — Approximate location of cone penetrometer test performed by Treadwell & Rollo, February 2000

DPT-1 — Approximate location of dynamic penetrometer test performed by Treadwell & Rollo, February 2000

Proposed basement finished floor elevation +48.25 feet (SFCD)

New Alleyway at grade +62.25 feet (SFCD)

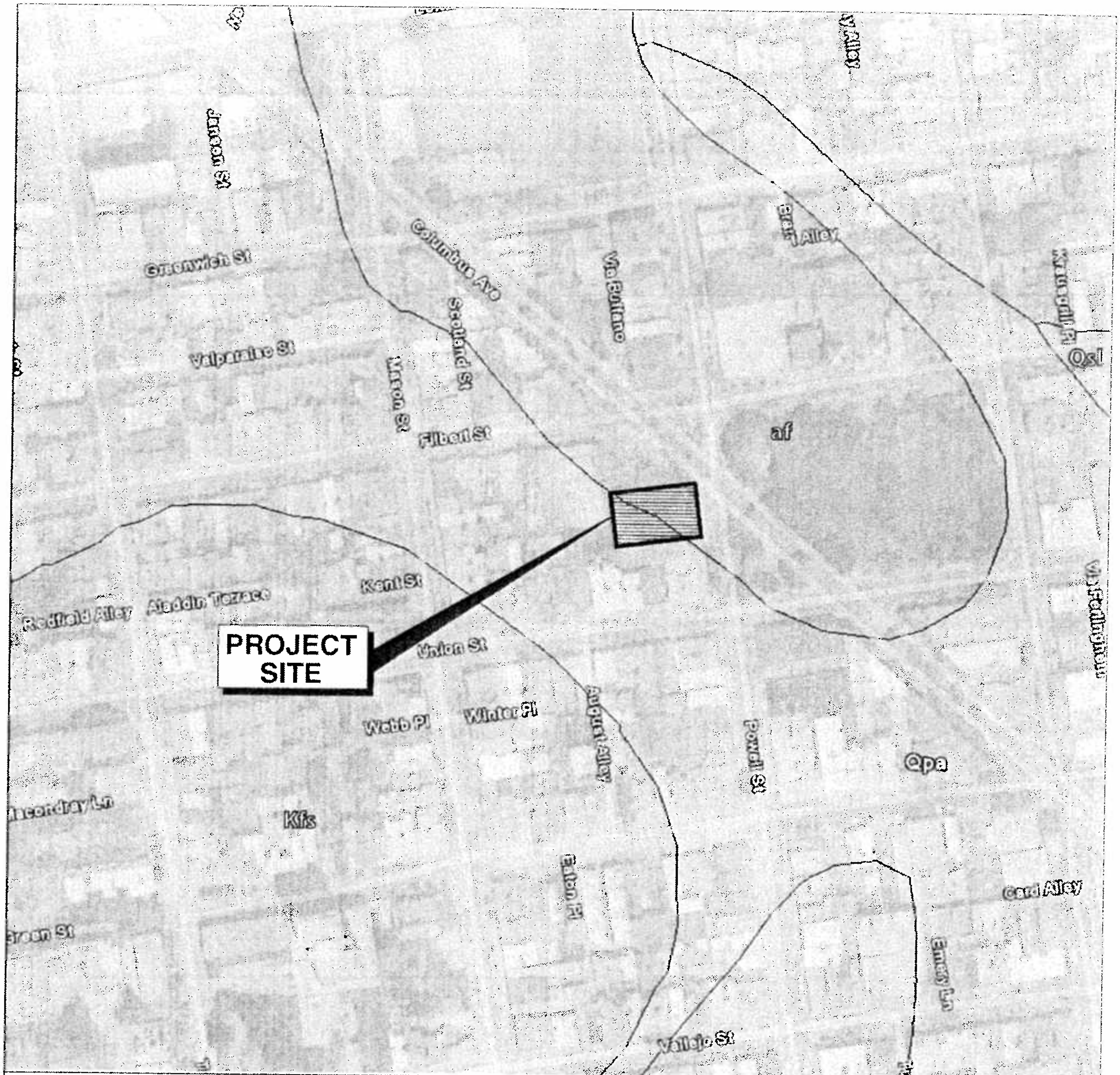
Reference
1. "As-built Survey at the Pagoda Theater for the Leirum Corporation, First Floor (Sheet 1 of 3)", (S-4379b dwg), prepared by Martin M. Ron Associates, dated 23 February 2000
2. "Garage Floor Plan, Sheet A 200", prepared by Naylor & Chu, Architects, dated 15 October 2008.

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

SITE PLAN

Date 11/26/08	Project No. 2766.03	Figure 2

Treadwell&Rollo



Base map: Google Earth and USGS San Francisco Bay Regional Geology and Geologic Hazards Map, San Francisco County, 2008.

EXPLANATION

af	Artificial Fill
Qsl	Hillslope Deposits (Quaternary)
Qpa	Alluvium (Pleistocene)
Kfs	Franciscan Complex sedimentary rocks (Cretaceous)

Geologic contact dashed where approximate and dotted where concealed

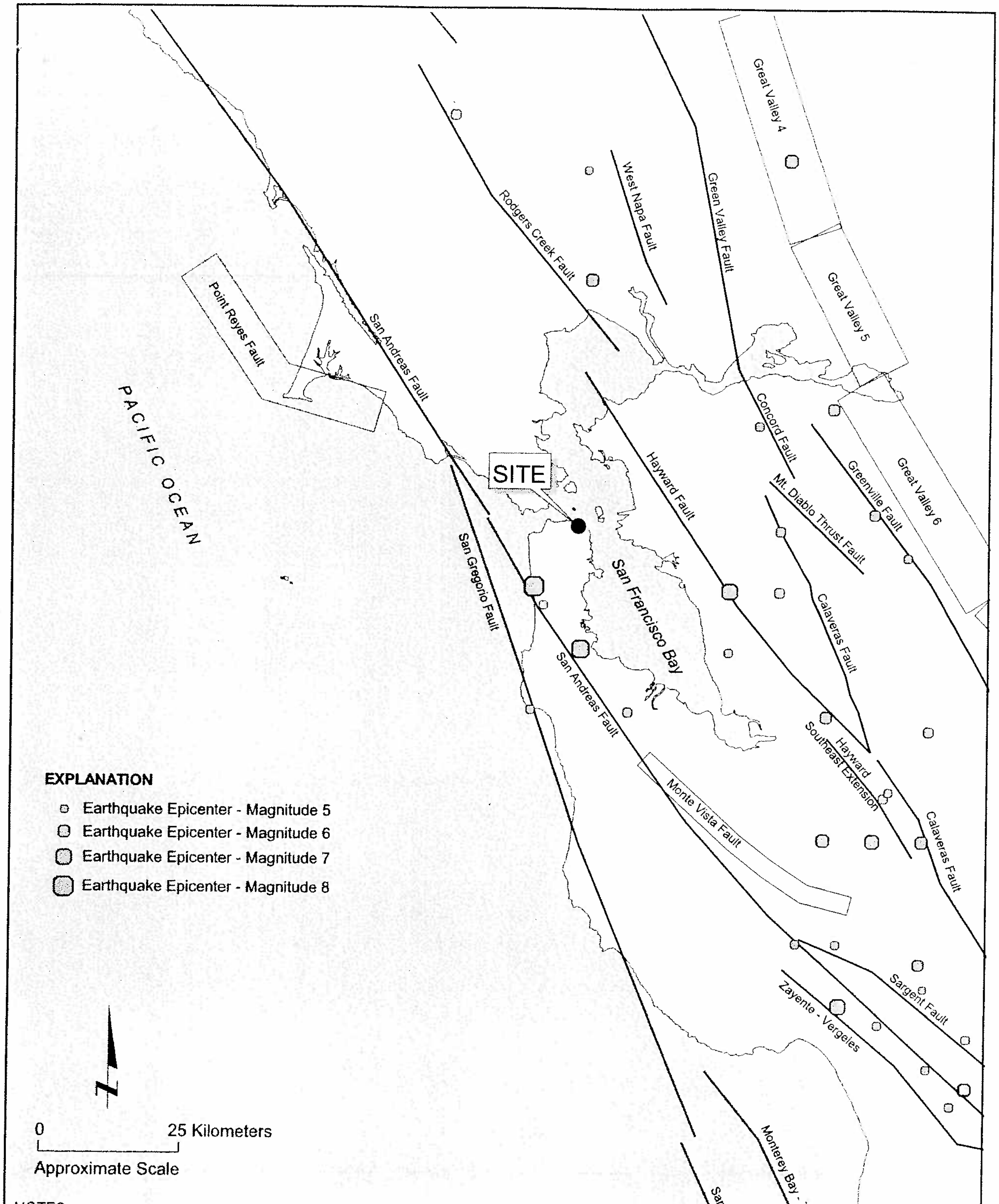
0 250 Feet
Approximate scale

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

Treadwell&Rollo

REGIONAL GEOLOGIC MAP

Date 11/14/08 | Project No. 2766.03 | Figure 3

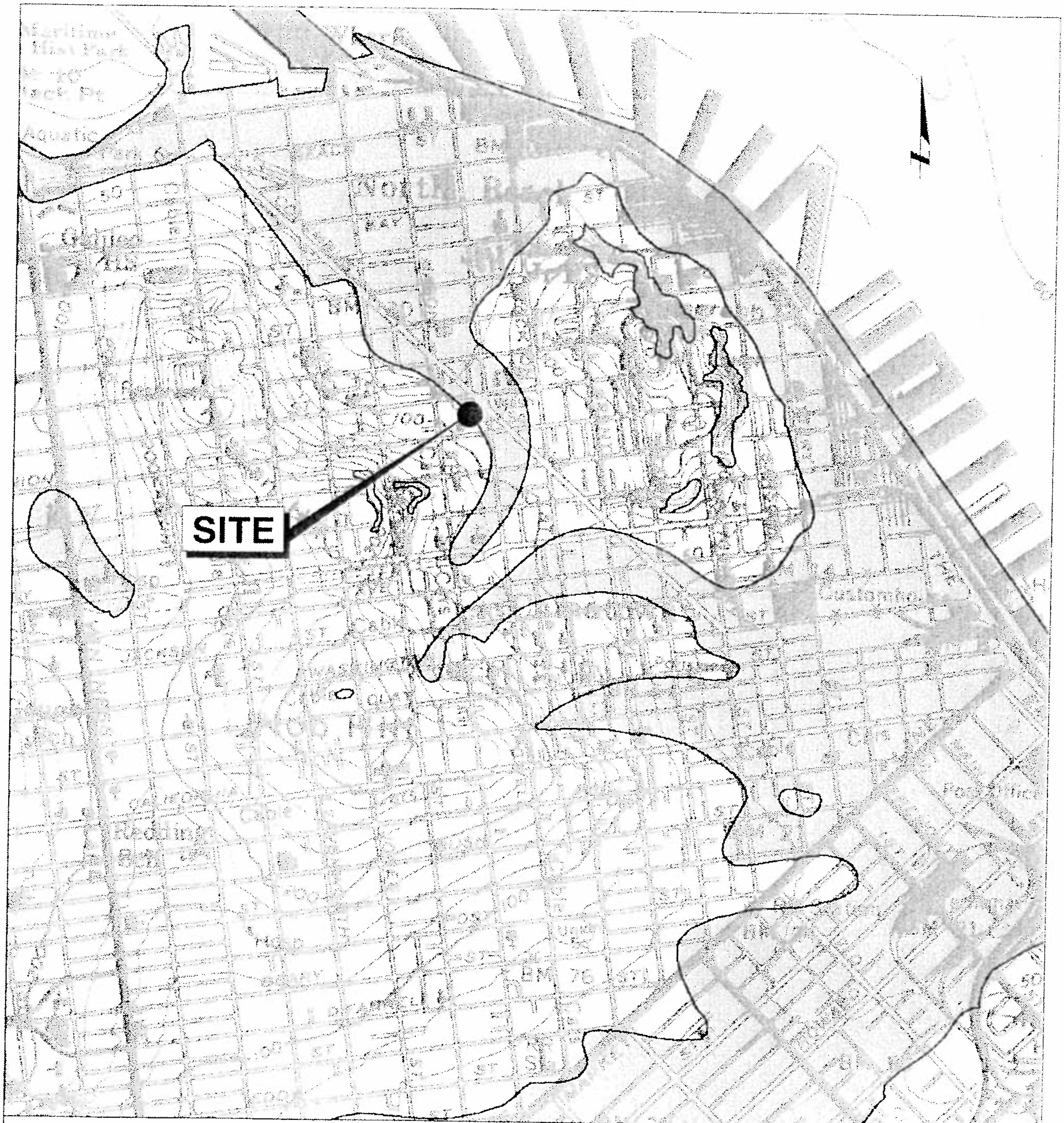


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

1731-1741 POWELL STREET LA CORNETA PALACE San Francisco, California	MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA
Treadwell&Rollo	Date: 11/26/08 | Project No. 2766.03 | 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.	

1731-1741 POWELL STREET **LA CORNETA PALACE** San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**		
Treadwell&Rollo	Date 11/26/08	Project No. 2766.03	Figure 5



EXPLANATION

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

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

0 1,000 2,000 Feet

Approximate scale

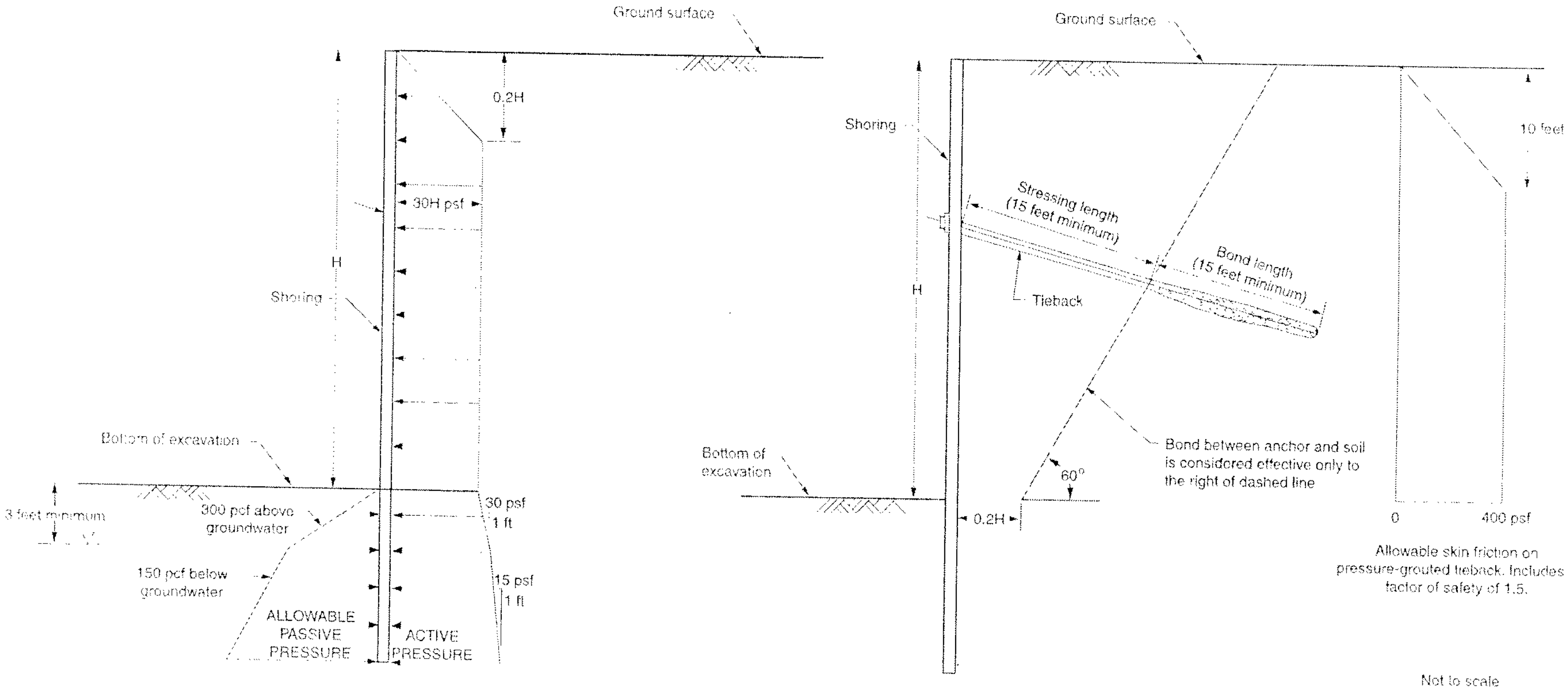
Reference:
State of California "Seismic Hazard Zones"
City and County of San Francisco
Released on November 17, 2001

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

Treadwell&Rollo

REGIONAL SEISMIC HAZARDS MAP

Date 12/01/08 | Project No. 2766.03 | Figure 6



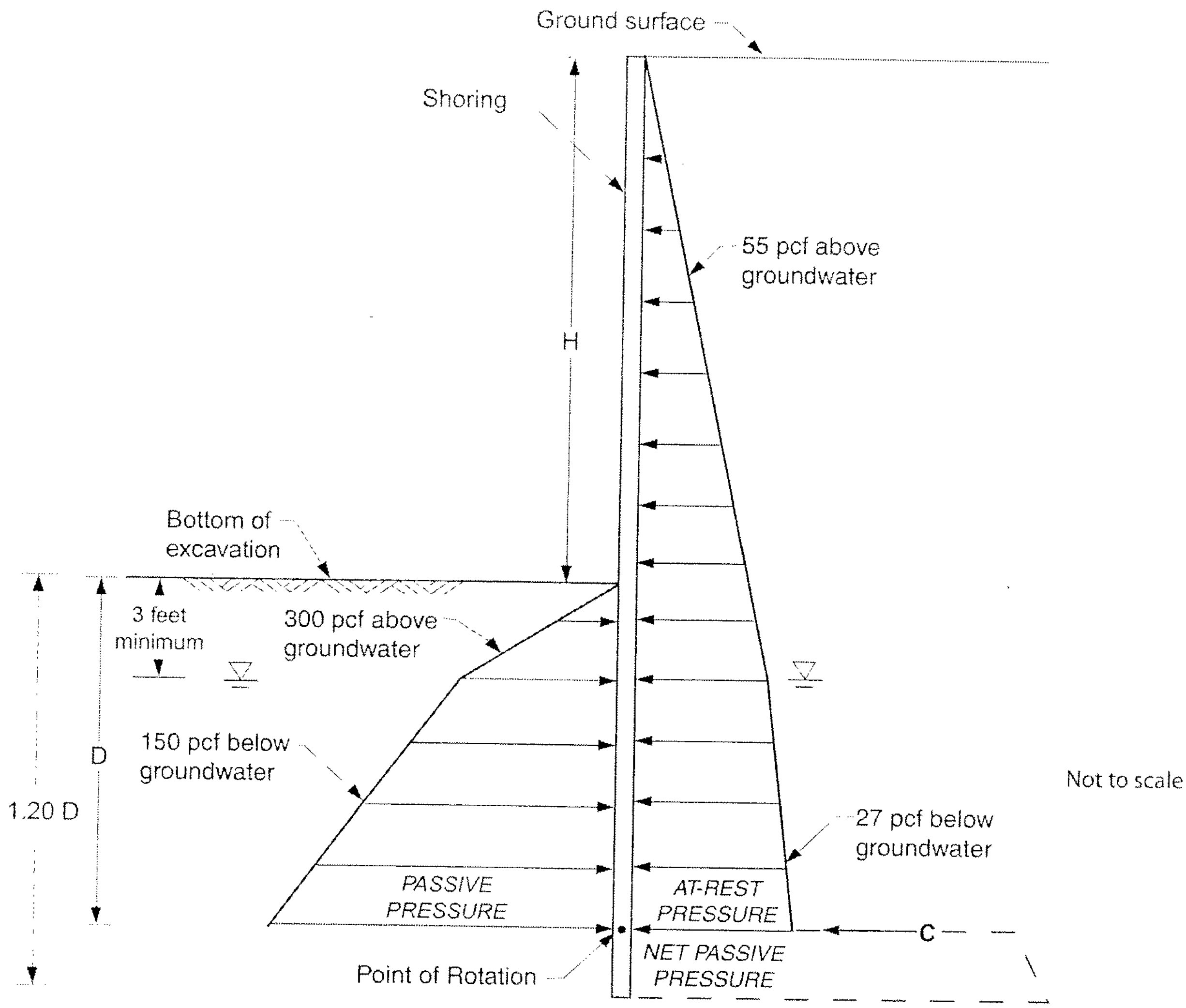
Notes:
1. Passive pressures include a factor of safety of about 1.5.
2. The passive pressures shown are based on the assumption that groundwater level is at least three feet below the bottom of the excavation.
3. For slanted drilled piers spaced at more than three times the slanted drilled piers diameter, the passive pressure should be assumed to act over three diameters.
4. For hand-excavated piers, the passive pressure should be assumed to act over one pier diameter.
5. Active pressure should be assumed to act over one pile diameter.
6. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
7. The recommended pressures do not include surcharges from adjacent buildings.

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

DESIGN PARAMETERS FOR TIED-BACK/ INTERNAL BRACING SOIL/CEMENT COLUMNS OR UNDERPINNING PIER-AND-LAGGING TEMPORARY SHORING SYSTEM

Date 11/26/08	Project No. 2766.03	Figure 7

Treadwell&Rollo



Notes:
1. Simplified pressure diagram is presented above. The net passive pressure on the right side of shoring below the point of rotation is replaced by a concentrated force C at the point of rotation.
2. Passive pressures include a factor of safety of about 1.5.
3. Passive pressures may be assumed to act over the pier spacing or three times the pier diameter, whichever is smaller.
4. Surcharge pressure, due to construction equipment and existing footings, if any, should be added to the above shoring pressure.
5. At-rest pressure below the excavation should be assumed to act over one pier diameter (for structural concrete).
6. Calculated embedment depth, D, should be increased by at least 20 percent to obtain the design depth of penetration.
7. The recommended pressures do not include surcharges from adjacent foundations. Surcharge pressure from adjacent foundations should be added to the above shoring pressures.
8. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

Treadwell&Rollo

LATERAL EARTH PRESSURES FOR CANTILEVER SHORING SYSTEM

Date 11/26/08 | Project No. 2766.03 | Figure 8

APPENDIX A

Log of Boring and Classification Chart

Log of Boring B-3

PROJECT: 1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: K. Lease

Date started: 10/8/08 | Date finished: 10/8/08

Drilling method: (4-inch) Solid Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

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

Ground Surface Elevation: 60 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	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
0–6				SC	3-inch Concrete Slab; CLAYEY SAND (SC) brown, medium dense, dry, brick fragments and concrete debris (FILL)						
3	S&H	20, 8, 12	12	SC							
5.5					∇						
6	S&H	10, 23, 33	34	SC					15	18.3	
6–31				SM	SILTY SAND (SM) yellow brown, dense, wet, with occasional gravel						
11	S&H	20, 56, 50/3.5"	64/9.5"	SM	very dense				15	18.5	
16	SPT	37, 55, 50/4"	105/10"	SM					13	24.8	
21	SPT	31, 50/5"	50/5"	SM							
26	SPT	37, 50/4"	50/4"	SM					6	25.9	
30.5	SPT	65, 50/3"	50/3"	SM							

Boring terminated at a depth of 30.75 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 5.5 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

Project No.: 2766.03 | Figure: A-1

TEST GEOTECH LOG 276603.GPJ TR.GDT 12/1/08

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

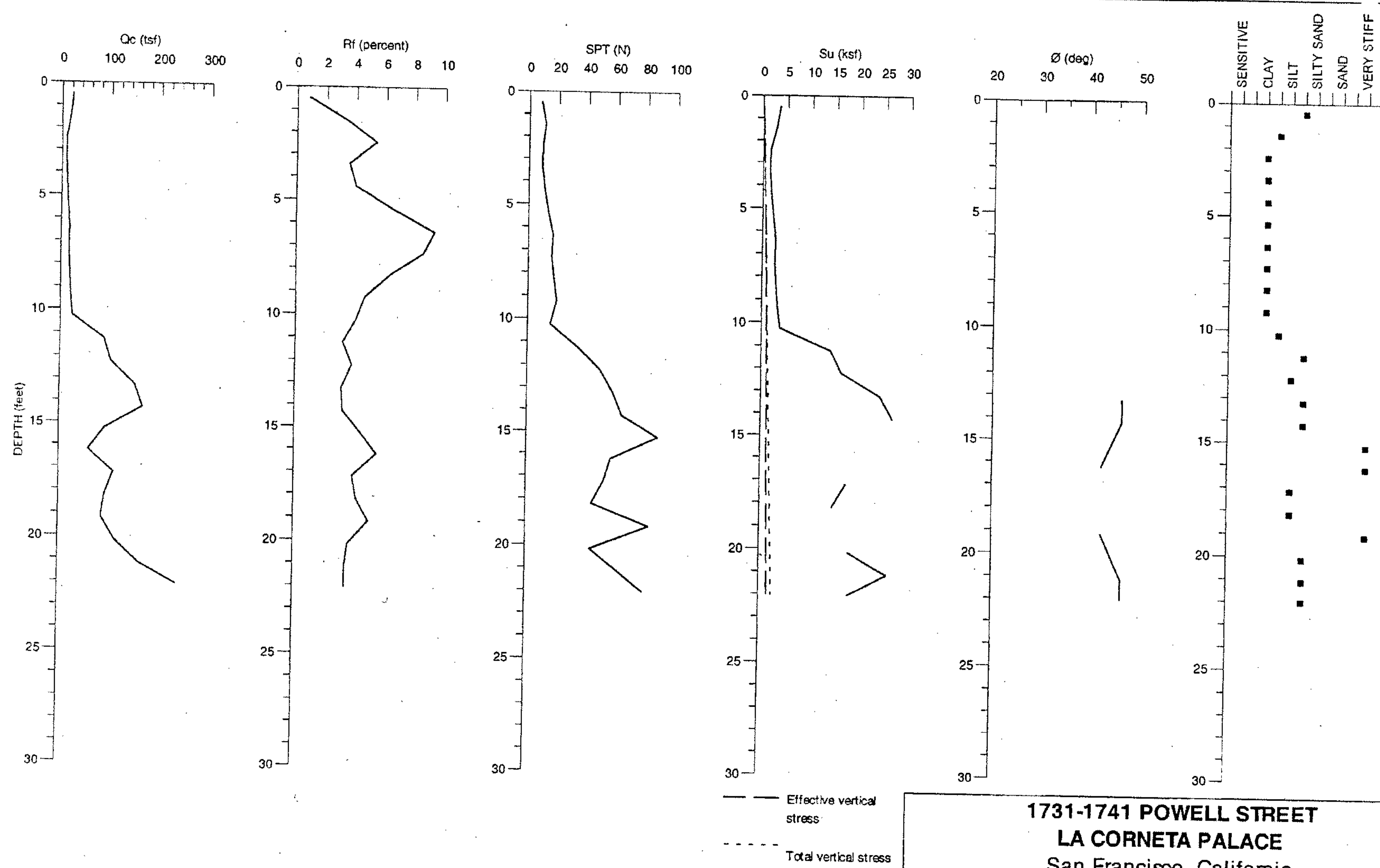
Treadwell&Rollo

CLASSIFICATION CHART

Date 11/13/08 | Project No. 2766.03 | Figure A-2

APPENDIX B

Logs of Cone Penetration Tests and Dynamic Penetrometer Tests from Previous Investigations by Treadwell & Rollo



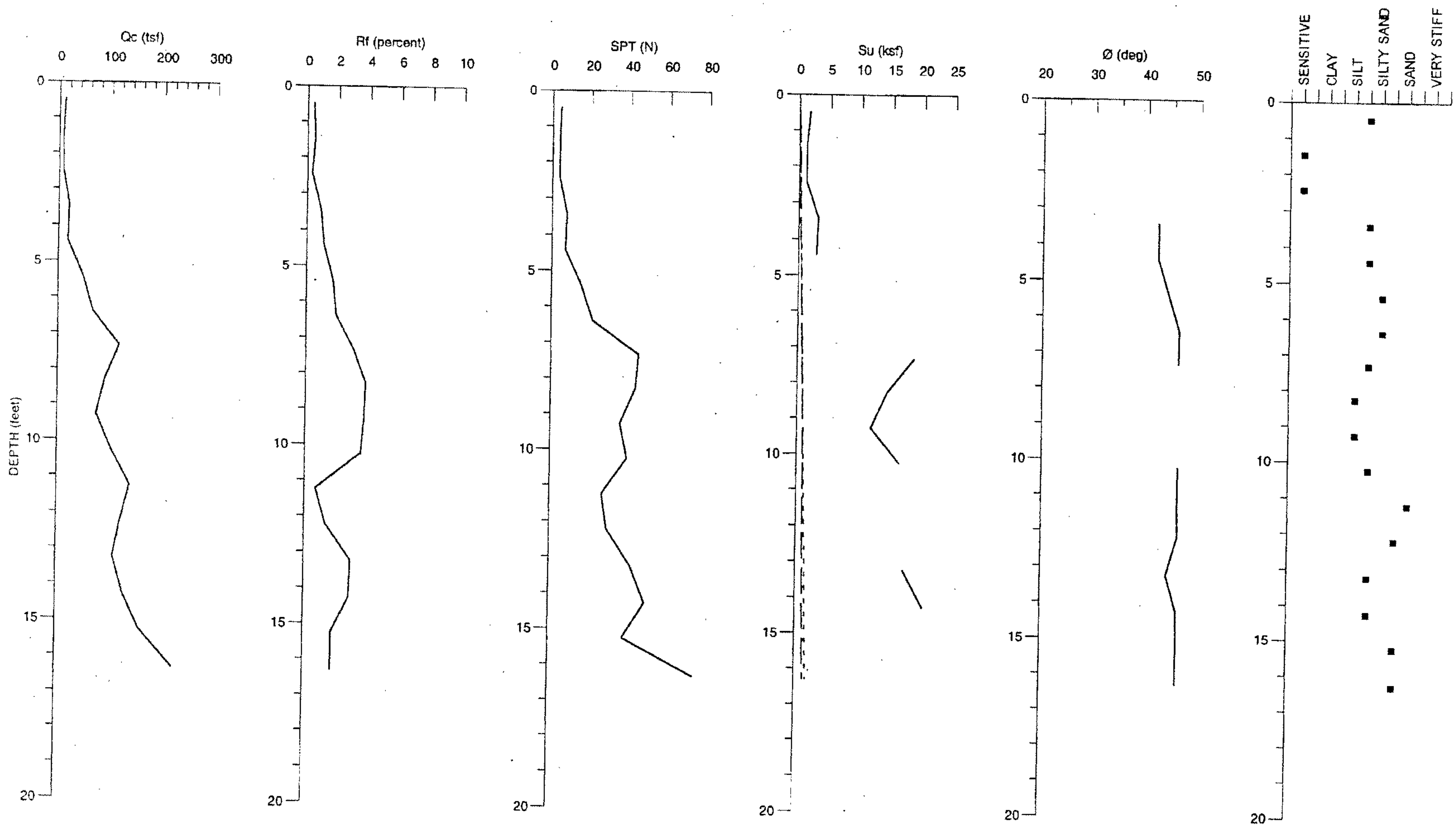
Terminated at 22.05 feet
Groundwater estimated at 8 feet.
Date performed: 29 February 2000.
Elevation: 59.85 feet, based on San Francisco City datum.

1731-1741 POWELL STREET
LA CORNETA PALACE
San Francisco, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 11/26/08	Project No. 2766.03	Figure B-1

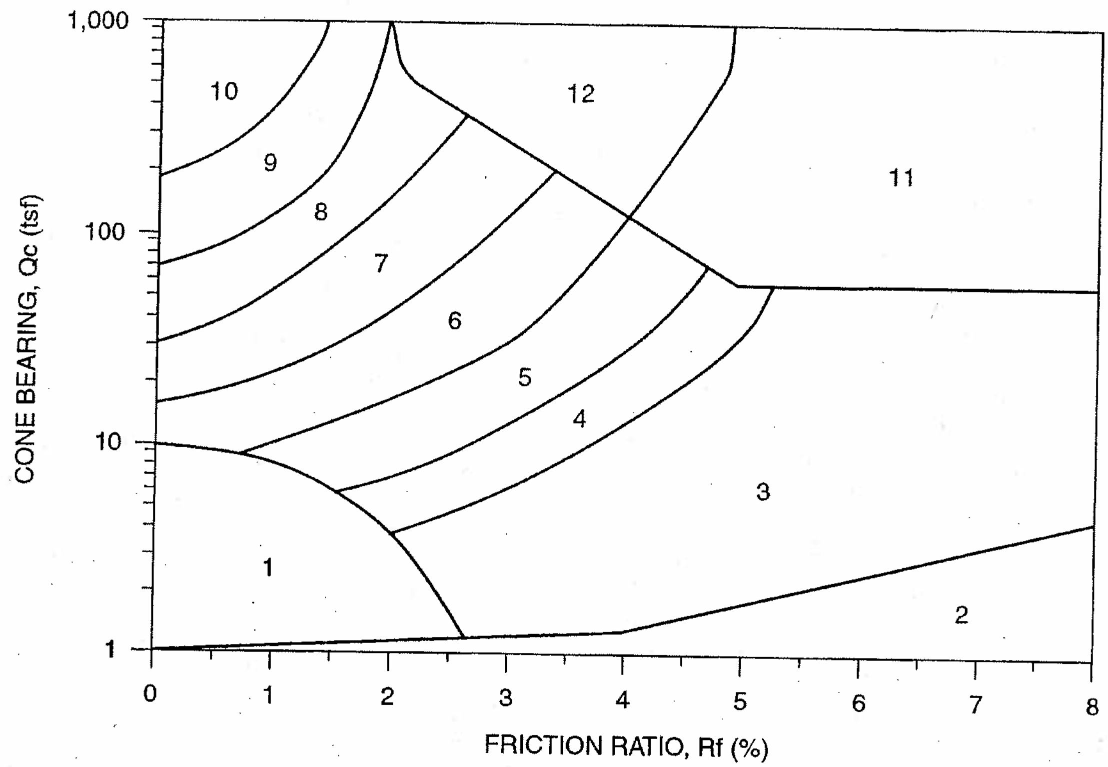
Treadwell&Rollo



Terminated at 16.31 feet
Groundwater estimated at 8 feet.
Date performed: 29 February 2000.
Elevation: 64.5 feet, based on San Francisco City datum.

1731-1741 POWELL STREET **LA CORNETA PALACE** San Francisco, California		
CONE PENETRATION TEST RESULTS **CPT-2**		
Date 11/26/08	Project No. 2766.03	Figure B-2

Treadwell&Rollo



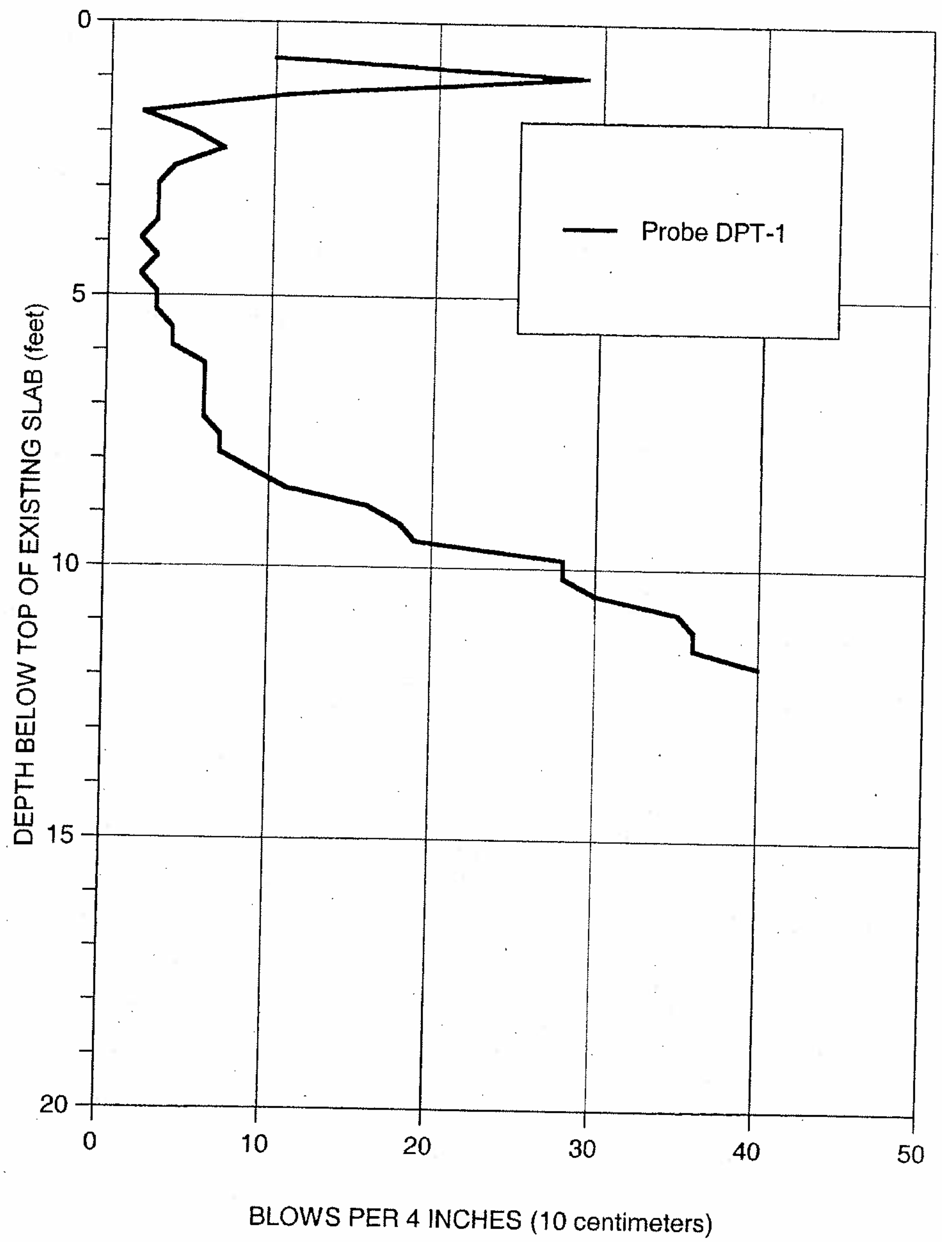
ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for Qc 9 tsf)	Sensitive Fine-Grained
2	1	15 (10 for Qc 9 tsf)	Organic Material
3	1	15 (10 for Qc 9 tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
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.

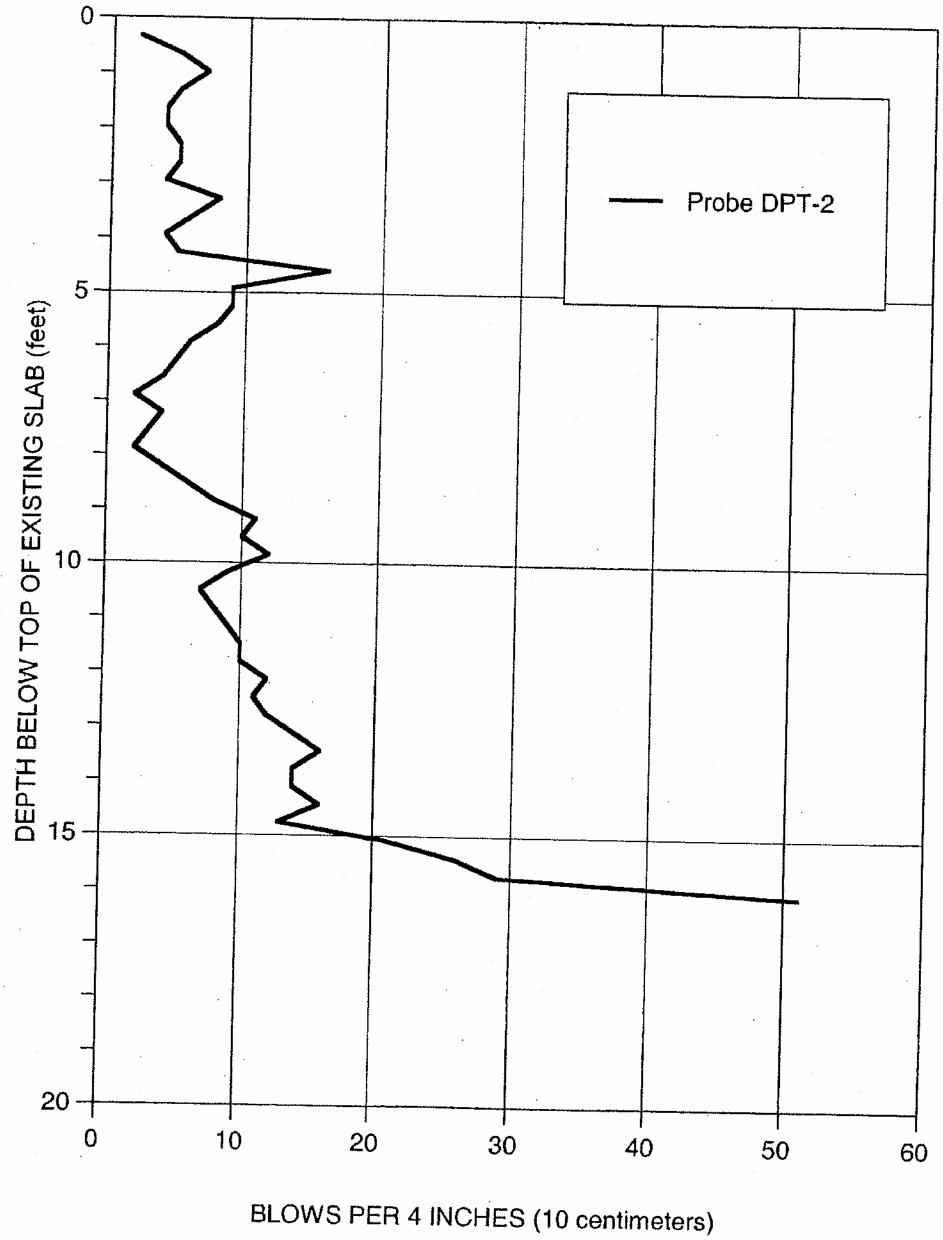
References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud Qc 9).
Estimated from local experience (fine-grained soils Qc > 9).

1731-1741 POWELL STREET **LA CORNETA PALACE** San Francisco, California	**CLASSIFICATION CHART FOR CONE PENETRATION TESTS**
Treadwell&Rollo	Date 11/26/08 · Project No. 2766.03 · Figure B-3



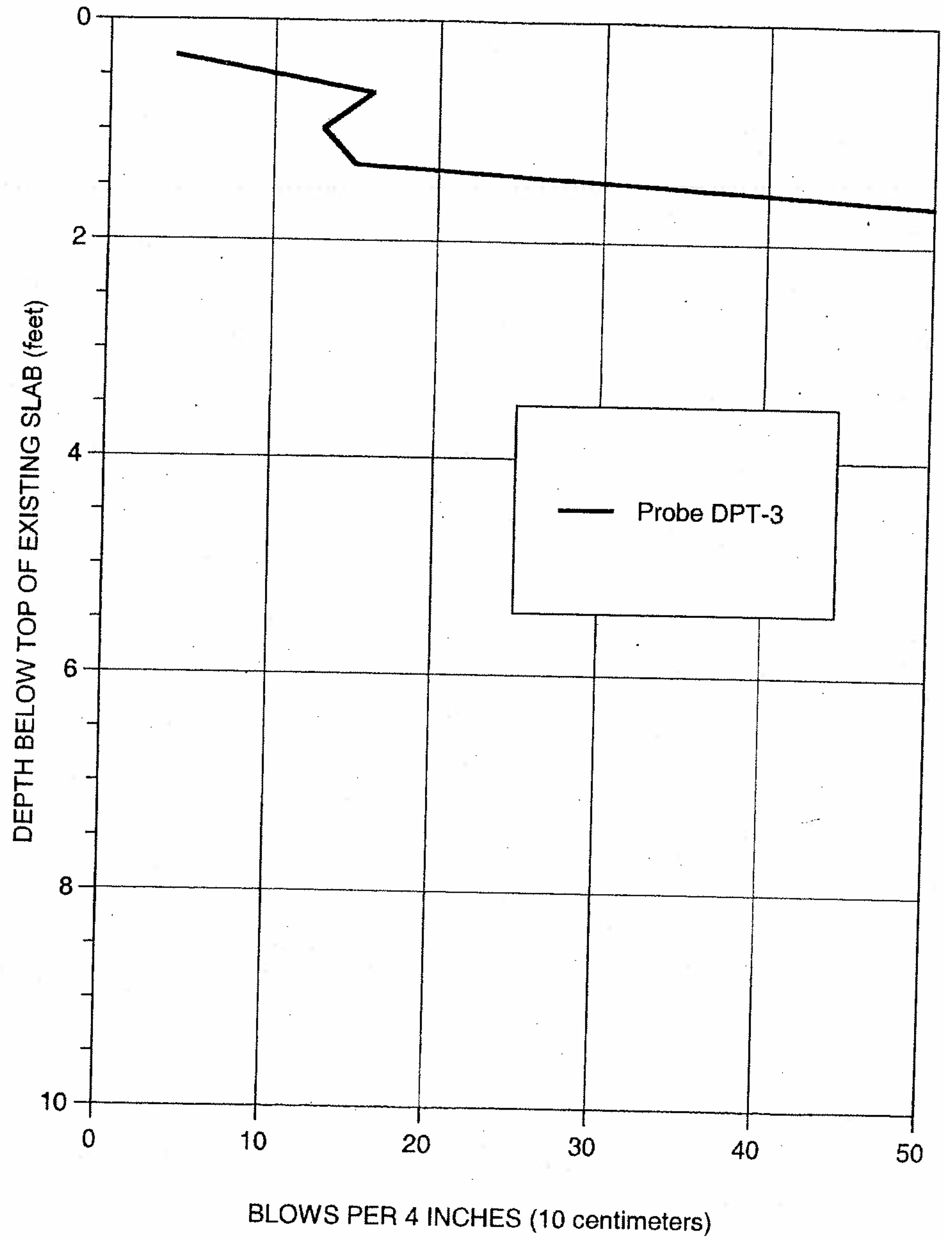
Elevation = 59.8 feet, based on San Francisco City datum.

1731-1741 POWELL STREET LA CORNETA PALACE San Francisco, California	RESULTS OF DYNAMIC PENETROMETER TEST DPT -1		
Treadwell&Rollo	Date 11/26/08	Project No. 2766.03	Figure B-4



Elevation = 61.0 feet, based on San Francisco City datum.

1731-1741 POWELL STREET LA CORNETA PALACE San Francisco, California Treadwell&Rollo	RESULTS OF DYNAMIC PENETROMETER TEST DPT-2	
Date 11/26/08	Project No. 2766.03	Figure B-5



Elevation = 60.0 feet, based on San Francisco City datum.

1731-1741 POWELL STREET LA CORNETA PALACE San Francisco, California	**RESULTS OF DYNAMIC PENETROMETER TEST DPT-3**
Treadwell&Rollo	Date 11/26/08 | Project No. 2766.03 | Figure B-6

APPENDIX C

Soil Corrosivity Test Data

California State Certified Laboratory No. 2153

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

25 November, 2008

Job No.0811055
Cust. No.10727

Mr. Timothy Wong
Treadwell & Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111

Subject: Project No.: 2766.03
Project Name: 1731 Powell Street, San Francisco
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Wong:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on November 10, 2008. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 20 mg/kg. Because the chloride ion concentration is less than 300 mg/kg, it is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 370 mg/kg and is determined to be sufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The pH of the soil is 8.3 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 460-mV, which is indicative of aerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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Client: Treadwell & Rollo
Client's Project No.: 2766.03
Client's Project Name: 1731 Powell Street, San Francisco
Date Sampled: 8-Oct-08
Date Received: 10-Nov-08
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 25-Nov-2008

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0811055-001	B-3 @ 15'	460	8.3	-	950	-	20	370

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	21-Nov-2008	21-Nov-2008	-	25-Nov-2008	-	21-Nov-2008	21-Nov-2008

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

Cheryl McMillen
Laboratory Director

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

DISTRIBUTION

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QUALITY CONTROL REVIEWER:

Maria G. Flessas
Principal