GEOTECHNICAL INVESTIGATION
BLOCK 30
MISSION BAY
San Francisco, California

Alexandria Real Estate Equities, Inc.
San Francisco, California

17 October 2007
Project No. 4086.16

Treadwell&Rollo
Environmental and Geotechnical Consultants

Treadwell&Rollo

17 October
Project 4086.16

Ms. Terezia Nemeth
Alexandria Real Estate Equities, Inc.
700 Owens Street, Suite 500
San Francisco, CA 94158

Subject: Geotechnical Investigation
Proposed Biotechnology Development
Block 30
Mission Bay
San Francisco, California

Dear Ms. Nemeth:

Treadwell & Rollo, Inc. is pleased to present this geotechnical investigation report for the development proposed on Block 30 at Mission Bay in San Francisco. Copies have been distributed as indicated at the end of this report.

The proposed development will consist of a steel-framed 6-story biotechnology/laboratory building. It will occupy the majority of the site. Subsurface conditions at the site consist of heterogeneous fill, underlain by Bay Mud, sand, stiff clay, and Franciscan Complex bedrock. We recommend the building be supported on driven piles gaining support in the soil or bedrock below the Bay Mud. This summary omits the detailed recommendations; therefore, anyone relying on the report must read it in its entirety.

The recommendations contained in the report are based on a limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found during construction. We should be retained to observe foundation installation, site grading, and compaction of utility trench backfill.

We appreciate the opportunity to assist you with this exciting and challenging project and look forward to working with you during final design and construction.

Sincerely yours,
TREADWELL & ROLLO, INC.

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed development of Block 30 in Mission Bay, San Francisco, as shown on Figure 1. Previously, we performed geotechnical investigations for nearby projects at Block 26, Block 26a, Block 27, Block 28, and Block X4. Concurrent with this investigation, we performed the investigation for Block 32; the results of the Block 32 investigation were published under a separate cover in a report dated 26 July 2007.

Block 30 is bound by proposed Terry A Francois Boulevard to the east, Block 32 to the south, proposed Bridgeview Way to the west, and South Street to the north, as shown on Figure 2. The site is rectangular, with plan dimensions of approximately 290 by 310 feet. Currently, the site is a paved parking lot.

The proposed development will consist of a steel-framed, 6-story biotechnology/laboratory building. The proposed structure will occupy the majority of the site. Plans include concrete pavers surrounding the building.

Grading plans and proposed finished floor elevations were not available at the time this report was written. Current site grades range from approximately Elevation 99.6 to 101.3 feet.[1] Approximately 10 feet of soil was excavated in the recent past, but the site has since been backfilled. We observed and tested the placement and compaction of the backfill under separate contracts; part of our services were performed for Alexandria and part were performed for Catellus.

2.0 SCOPE OF SERVICES

The scope of services was outlined in our 13 April 2007 proposal. We reviewed existing subsurface data from the site and in the vicinity. To supplement existing information, we explored the subsurface conditions at the site by drilling five test borings and performing one cone penetration test (CPT).

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[1] Elevations estimated based on the topographic survey by Winzler and Kelly (June 2006).

Engineering studies were performed based on the soil and groundwater conditions defined by the borings and the results of laboratory tests. Using the results of our engineering studies, and the ongoing experience gained on similar sites in Mission Bay, we developed conclusions and recommendations regarding:

- soil, rock, and groundwater conditions at the site
- the most appropriate foundation type(s)
- design criteria for the most appropriate foundation type, including values for vertical and lateral pile capacities
- floor slab support
- estimated foundation and surrounding ground surface settlements
- seismic hazards, including ground rupture, liquefaction, and differential compaction
- mitigation measures to reduce the risk of seismic hazards, if needed
- subgrade preparation
- corrosion potential
- concrete flatwork and paver sections
- construction considerations.

3.0 FIELD INVESTIGATION

To supplement available subsurface information and gain further site specific data, we drilled five test borings and performed one CPT at the project site. The approximate location of the test borings and CPT are presented on Figure 2.

Prior to performing the field investigation, we:

- obtained a soil boring permit from the Monitoring Wells Section of the San Francisco, Department of Public Health (SFDPH)
- notified Underground Service Alert

- verified the boring location was clear of underground utilities using an independent utility locating contractor.

3.1 Test Borings

From 3 through 6 May 2007, the test borings, designated B30-1 through B30-5, were drilled using a truck-mounted, rotary-wash drill rig operated by Pitcher Drilling Company. The test borings were drilled to depths of approximately 79-1/2 to 129 feet below the existing ground surface. Our field engineer logged the borings and obtained samples of the material encountered for visual classification and laboratory testing. The borings were backfilled with cement grout under the observation of a San Francisco Department of Public Health inspector.

The boring logs are presented on Figures A-1 through A-5 in Appendix A. The soil and rock are classified in accordance with the charts shown on Figures A-6 and A-7, respectively.

Soil samples were obtained using three sampler types: two split-barrel samplers and a thin-walled sampler. The sampler types are as follows:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with brass tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners
- Shelby tube sampler with a 3.0-inch outside diameter and 2.875-inch inside diameter.

The sampler types were chosen on the basis of soil type and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soil. The Shelby tubes were used to obtain relatively undisturbed samples of the soft cohesive soil.

The S&H and SPT samplers were driven with a 140-pound safety hammer (rope and cathead system) falling about 30 inches. Where the S&H sampler was used, the blow counts required to drive the sampler the final 12 inches of an 18-inch drive were corrected to approximate SPT blow counts and are shown on the boring logs. Where the SPT sampler was used, the actual blow counts are shown on the boring logs. Hydraulic pressure was used to advance the 30-inch-long Shelby tubes into the soil and the pressure required is shown on the logs, measured in pounds per square inch (psi).

3.2 Cone Penetration Tests

On 4 May 2007, one CPT, designated C30-1, was advanced with a CPT rig provided by John Sarmiento and Associates. The approximate location of the CPT is shown on Figure 2. The CPT was advanced through the existing fill and into the underlying Bay Mud. C30-1 was terminated at a depth of approximately 35 feet below existing ground surface. The CPT was performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe into the ground. Electrical strain gauges within the cone continuously measured soil parameters during the entire depth of each probing. Soil data was recorded in the field on magnetic tape and transferred to a computer following the test. Accumulated data was then processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. The log of the CPT is presented on Figure A-8 in Appendix A. Soil types were determined using the classification chart shown on Figure A-9.

3.3 Laboratory Testing

All samples recovered from the field exploration program were examined for soil classification, and representative samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, plasticity, strength, percent fines, and compressibility. Results of the laboratory testing are included on the boring logs and in Appendix B.

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed to evaluate the corrosivity of the various soil types. The results of the corrosivity analyses are presented in Appendix C.

3.4 Other Investigations

We used borings by others from previous investigations and borings by Treadwell & Rollo from nearby sites, including Block 32, to evaluate subsurface conditions. The logs of the nearest test borings are presented in Appendix D.

4.0 SITE CONDITIONS

Our understanding of the site conditions are based on our earlier research of the entire Mission Bay development area, a review of published literature, subsurface exploration, and the knowledge gained from our ongoing involvement during construction of many projects in Mission Bay.

4.1 Site Conditions

Originally, the site was under water and a part of a shallow bay called Mission Bay. The area was reclaimed by placing fill starting after 1906. The Mission Bay Hazards Mitigation Program (Environmental Science Associates, 1990) reports that the majority of the fill at the site was placed by 1920. During that time, numerous areas of San Francisco were being developed. As a result, sand dunes were being removed along a stretch of current Market Street, and rock was being excavated from various hills throughout the City. Historic information indicates portions of these materials were dumped into Mission Bay for reclamation purposes.

Although we do not have specific information for Block 30, we understand nearby sites were occupied by an oil company and oil-related facilities, several above-ground storage tanks, offices, a lumber yard, a junk yard, and railroad tracks since 1902 (ESA, 1990).

As previously discussed, grading plans or proposed finished floor elevations were not available at the time this report was published. Current site grades range from approximately Elevation 99.6 to 101.3 feet. Approximately 10 feet of soil was excavated in the recent past, but the site has since been backfilled. Block 30 is currently a paved parking lot. The parking lot is relatively level and is graded to drain.

When a site nearby was excavated, wood piles were encountered. Although we have not observed any during our investigation, they may be present on Block 30. We understand the locations of the wood piles were scheduled to be surveyed; if this was done, the survey should be obtained and provided to the design team.

4.2 Subsurface Conditions

Three idealized subsurface profiles illustrating the general subsurface conditions at the site are presented on Idealized Subsurface Profiles A-A', B-B', and C-C' on Figures 3 through 5. The profiles depict the existence of fill, Bay Mud, sand and clay, Colma Formation, clay, and bedrock, as detailed below:

Fill: Where explored, the site is blanketed by approximately 9 to 30-1/2 feet of fill. The fill consists of gravel, sand, silt, and clay mixtures. It contains significant amounts of rock fragments, including serpentinite boulders, and building rubble. We observed and tested and where tested it was compacted. Corrosivity analyses indicate the fill material is "severely corrosive" to "non-corrosive."

Bay Mud: A weak and compressible marine clay deposit, referred to as Bay Mud, is present beneath the fill. This layer is 13 to 45 feet thick where explored within the project site and generally increases in thickness to the north and west. Laboratory test results from this and nearby investigations indicate it has a compression ratio of 0.26 to 0.35 and is normally to slightly overconsolidated,[2] with consolidation ratios ranging from 1.0 to 1.3. The clay has a coefficient of consolidation, c_v, of 22 to 54 feet squared per year (ft^2/yr) along the virgin compression curve. The coefficient of consolidation is a measure of the time rate of consolidation settlement; the higher the value, the faster the soil will consolidate.

The undrained shear strength of the Bay Mud is approximately 360 to 750 pounds per square feet (psf) where tested.

Sand and Clay: A dense clayey sand and stiff to hard clay was encountered below the Bay Mud in all borings. Where encountered the sand and clay layer is 6 to 14 feet thick. Where tested, the undrained shear strength of the clay is 2,030 to 3,450 psf.

Colma Formation: A medium dense to very dense sand, sand with clay, and clayey sand was encountered below the sand and clay. Where encountered and tested, the sand is approximately 5 to 30 feet thick with percent fines ranging from 5.6 to 22.9. The Colma Formation generally becomes thicker to the north and west.

Clay, Clay with Gravel, and Gravelly Clay: Very stiff to hard sandy clay and clay was encountered above the bedrock in borings B30-3, B30-4, and B30-5. Where encountered, the layer is 3 to 5-1/2 feet thick. An 8-foot-thick layer of stiff to very stiff Old Bay Clay was encountered in boring B30-1.

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2 An underconsolidated clay has not yet achieved equilibrium under the existing load; a normally consolidated clay has completed consolidation under the existing load; and an overconsolidated clay has experienced a pressure greater than its current load.

Bedrock: Bedrock was encountered at elevations ranging from 31.6 feet to -5.4 feet, approximately 69 to 106 feet below grade, respectively. Bedrock generally becomes deeper to the north and west. Bedrock encountered consists of serpentinite, shale, and sandstone of the Franciscan Complex. The rock is plastic to weak and with moderate to little weathering. Approximate contours of top of bedrock elevations are presented in Figure 6.

Groundwater: Groundwater was encountered during drilling. It was measured in all boreholes prior to switching from auger drilling to rotary wash, with the exception of B30-1. During our investigation, measured groundwater levels ranged from Elevation 90.5 feet to Elevation 92.4 feet. Elevations as high as Elevation 96 feet were measured at a nearby site.

5.0 GEOLOGY AND SEISMICITY

Our evaluation of the geology and seismicity of the area is based on our review of published reports and information in our files from other sites in the vicinity.

5.1 Regional Geology

The site is in the northeast portion of the San Francisco peninsula, which lies within the Coast Ranges geomorphic province. The northwesterly trend of ridges and valleys characteristic of the Coast Ranges is obscured in San Francisco, except for features such as Russian Hill, Telegraph Hill, Hunters Point, and Potrero Hill. San Francisco Bay and the northern portion of the peninsula lie within a down-dropped crustal block bound by the East Bay Hills and the Santa Cruz Mountains. The San Francisco Bay depression resulted from interaction between the major faults of the San Andreas fault zone, particularly the Hayward and San Andreas faults east and west of the bay, respectively (Atwater, 1979).

San Francisco's topography is characterized by relatively rugged hills formed by Jurassic- to Cretaceous-aged bedrock (Schlocker, 1974). The bedrock consists of highly deformed and fractured sedimentary rocks of the Franciscan Complex. The present topography resulted mainly from east-west compression of coastal California during the late Pliocene and Pleistocene epochs (Norris and Webb, 1990).

Serpentinite encompasses Potrero Hill immediately southwest of the site. The serpentinite bedrock is associated with ancient shear zones within and bounding portions of the Franciscan Complex bedrock units. The shear zones generally consist of a mixture of hard blocks of bedrock, from less than an inch to 25 feet or more in diameter, contained within a matrix of soft, intensely sheared shale. Serpentinite is the most common rock type, however, hydrothermally altered rocks such as calc-silicate compositions are common locally.

The fault separating the sandstone and shale units from the serpentinite is part of the Hunters Point- Fort Point Shear Zone. This shear zone extends across the width of the Peninsula and continues offshore both to the northwest and southeast. It is an ancient tectonic feature associated with a Cretaceous (approximately 100 million years ago) subduction zone and the emplacement of the Franciscan Complex units, and is not part of the more recent tectonic environment associated with the Pacific and North American plate interaction along the San Andreas Fault Zone.

The low-lying areas of the San Francisco peninsula are underlain by Quaternary sediments deposited on eroded Franciscan bedrock. Oscillating late-Quaternary sea levels that resulted from the advance and retreat of glaciers worldwide influenced sediment deposition within the pre-historic bay margin. The resulting sequence of alternating estuarine and terrestrial sediments corresponds to high and low sea-level stands, respectively. In contrast, Quaternary sediments in the plains landward of the bay are predominantly terrestrial.

By late Pleistocene time, the high sea level associated with the Sangamon interglacial (about 125,000 years ago) resulted in deposition of the Yerba Buena Mud (Sloan, 1992). Also known locally as "Old Bay Clay," the Yerba Buena Mud was deposited in an estuarine environment similar in character and extent to the present bay. Sea level lowering associated with the onset of Wisconsin glaciation exposed the bay floor and resulted in terrestrial sedimentation, such as the Colma formation, on the Yerba Buena Mud. Sea level rose again starting roughly 20,000 years ago, fed by the melting of Wisconsin-age glaciers. The sea re-entered the Golden Gate about 10,000 years ago (Atwater, 1979). Inundation of the present bay resulted in deposition of estuarine sediments, called Bay Mud, which continue to accumulate in the bay.

Historical development of the San Francisco Bay area resulted in placement of artificial fill material over substantial portions of modern estuaries, marshlands, tributaries, and creek beds in an effort to reclaim land (Nichols and Wright, 1971).

5.2 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 7. For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[3] [Working Group on California Earthquake Probabilities (WGCEP) (2003) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
San Andreas - 1906 Rupture	12.6	West	7.9
San Andreas - Peninsula	12.6	West	7.15
North Hayward	16	Northeast	6.49
Total Hayward	16	Northeast	6.91
Total Hayward-Rodgers Creek	16	Northeast	7.26
San Andreas- North Coast South	17	West	7.45
South Hayward	17	East	6.67
Northern San Gregorio	19	West	7.23
Total San Gregorio	19	West	7.44
Mt Diablo - MTD	33	East	6.65
Total Calaveras	34	East	6.93
Rodgers Creek	36	North	6.98
Concord/Green Valley	38	East	6.71
Monte Vista-Shannon	39	Southeast	6.8
Greenville	50	East	6.94

Figure 7 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the

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

San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 8) 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 93 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

6.0 GEOLOGIC HAZARDS

During a major earthquake, strong to violent ground shaking is expected to occur at the project site (Treadwell & Rollo, 2000). Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[4] lateral spreading,[5] cyclic densification,[6] landsliding, or can cause a tsunami. Each of these conditions has been evaluated based on our literature review, field investigation and analysis, and is discussed in this section.

6.1 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies during a major earthquake, it experiences a temporary loss of shear strength caused by a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. The site is within an area designated as potentially liquefiable (URS/Blume, 1974 and CDMG, 1997, adopted by CCSF April 1999). There was no documented observation of liquefaction at this site during the 1989 Loma Prieta Earthquake (Benuska, 1990).

The CPT and all of the borings, except B30-1 and B30-2, encountered very loose to medium dense sand or gravel with varying amounts of clay just above or below the water table, ranging in thickness from 1-1/2 to 15-1/2 feet. These layers could liquefy during a major earthquake. Using the Tokimatsu and Seed (1987) method for evaluating earthquake-induced liquefaction settlement and a peak ground acceleration (PGA) of 0.45g based on the site specific Probabilistic Seismic Hazard Assessment (PSHA) Model for 10% probability of being exceeded in 50 years, we estimate up to 6 inches of settlement may occur at locations across the site.

The liquefiable layer is not continuous; therefore, we judge the risk of lateral spreading is low.

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4 Liquefaction is a phenomenon in which saturated (submerged), cohesionless soil experiences a temporary loss of strength because of the buildup of excess pore water pressure, especially during cyclic loading such as those induced by earthquake. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded, fine-grained sand.

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

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

6.2 Cyclic Densification

Cyclic densification can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. Up to eight feet of loose to medium dense sand was encountered in several of the borings above the groundwater table. These layers may densify during an earthquake. Using the Pradel method for estimating cyclic densification of dry sand, we estimate settlement could be up to 1/4 inch.

6.3 Tsunami

According to published data (URS/Blume, 1974) the maximum run up (tsunami wave) at the Presidio occurred after the 1964 Alaskan earthquake. The wave measured 7.5 feet at the Golden Gate; no damage was reported along the San Francisco shoreline. The United States Geologic Survey (USGS) estimates the maximum probable tsunami wave run up at the Golden Gate will be 20 feet (Ritter and Dupre, 1972). If the maximum probable tsunami occurs, the site is within an area of potential tsunami inundation. In the China Basin Channel, the run up would be reduced to less than 10 feet (URS/Blume 1974).

6.4 Landslides, Erosion, and Seepages

The site is relatively level; therefore, the project site should not be subject to landslides or erosion. No springs or seepages were observed on site.

6.5 Probabilistic Seismic Hazard Analysis and Ground Response Analyses

We expect this site will experience strong ground shaking during a major earthquake on any of the nearby faults. To estimate the rock motion at the site, we performed a probabilistic seismic hazard analysis (PSHA). In accordance with our proposal, we developed design ground motions for a hazard level having 10 percent probability of exceedance in 50 years, which is consistent with the definition of Design Basis Earthquake (DBE) in the 2001 San Francisco Building Code (SFBC).

The probabilistic seismic hazard analysis (PSHA) was performed using the computer code EZFRISK 7.23 (Risk Engineering 2007). This approach is based on the probabilistic seismic hazard model developed by Cornell (1973) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources and earthquake activities were assigned to the faults based on historical and geologic data. The site-specific effects of the overburden soil were evaluated using the ground response program SHAKE91 (Idriss and Sun 1992) as part of a computational module in EZFRISK.

Details of our analyses are presented in Appendix E. The recommended horizontal spectrum is shown on Figure 9. Digitized values of the recommended DBE spectrum for a damping ratio of 5 percent are presented in Table 3.

TABLE 3
Recommended DBE Spectral Acceleration (g)
Damping Ratio of 5 percent

Period (seconds)	Recommended DBE Spectral Acceleration
0.00	0.450
0.10	0.536
0.20	0.751
0.30	0.924
0.40	1.056
0.50	1.149
0.60	1.204
0.70	1.225
0.80	1.216
0.90	1.182
1.00	1.126
1.10	1.055
1.20	0.971
1.30	0.881
1.40	0.788
1.50	0.695
1.60	0.606
1.70	0.524
1.80	0.451
1.90	0.388
2.00	0.350
2.50	0.233
3.00	0.174
4.00	0.111

7.0 DISCUSSION AND CONCLUSIONS

On the basis of our investigation and experience with similar sites, we conclude the project is feasible from a geotechnical standpoint. Geotechnical issues of concern include:

- adequate foundation support
- settlement behavior
- soil corrosivity
- groundwater
- construction considerations.

7.1 Settlement

The laboratory consolidation test results indicate that the Bay Mud is normally to slightly overconsolidated. This layer is 32-1/2 to 45 feet thick on the north side of the site and 13 to 31 feet thick on the south, where explored within the project site. The Bay Mud generally increases in thickness to the north and west. Depending on the amount of new fill placed, a new cycle of primary consolidation and secondary compression (strain-related movements) may begin, causing additional settlement to occur.

Grading plans or proposed finished floor elevations were not available at the time this report was written. Current site grades range from approximately Elevation 99.6 to 101.3 feet. Approximately 10 feet of soil was excavated in the recent past, but the site has since been backfilled.

Estimates of primary consolidation and secondary compression resulting from new fill loads over the next 50 years were estimated for a bottom of slab elevation equal to 100 feet, 101 feet, and 102 feet. Over the next 50 years, no settlement should occur if the bottom of slab elevation is 100 feet. We compute that settlement will be 2 to 2-1/2 inches for a bottom of slab elevation of 101 feet and 2-1/2 to 5 inches for Elevation 102 feet.

During a strong earthquake, the results of our analyses indicate that up to about 6-1/4 inches of additional cyclic densification and liquefaction-induced settlement may occur.

Because the building will be pile supported, as discussed in Section 7.2, settlement will be evident at building entrances and will affect utilities entering the building. Furthermore, differential settlement across the site will have an adverse effect on exterior improvements such as concrete flatwork, and asphalt pavements.

7.2 Foundations

The factors influencing the selection of a safe, economical foundation system with adequate capacities are:

- the presence of heterogeneous fill
- the presence of weak, compressible Bay Mud
- potential total and differential settlement if building loads are imposed on the fill and Bay Mud
- the variations in thicknesses, density, and depth of potential bearing layers.

The fill in its present condition is not capable of providing adequate bearing for a shallow foundation system; erratic and unpredictable settlement would occur. The Bay Mud beneath the site is weak, varies in thickness, and will consolidate under the weight of building loads. Even though the Bay Mud is normally to slightly overconsolidated, the heavy loads imposed by the new building will cause new excessive total and differential settlement that would damage the buildings.

Considering the poor bearing capacity of the existing fill and the anticipated differential settlement created by Bay Mud consolidation, we conclude a deep foundation system consisting of driven piles is the most appropriate and economical method for support of the building and floor slab. The piles should extend below the fill and Bay Mud and gain support from friction in the soil below the Bay Mud and end-bearing in the dense sand or bedrock.

A medium dense to very dense sand with clay, and clayey sand was encountered below the Bay Mud. Where encountered, the sand is approximately 5 to 30 feet thick with percent fines ranging from 5.6 to 22.9, where tested. The dense sand generally becomes thicker to the north and west. Driven piles, especially displacement type piles, typically encounter refusal in very dense, clean sand layers greater than 10 feet thick. If a significant amount of fines (greater than about 10 percent of either clay or silt) are present, the pile will generally not achieve refusal in the layer. Furthermore, if silt or clay layers are present below a thin layer of sand, the pile may punch through the sand. Consequently, pile lengths may vary across the site. Where piles do not meet refusal in dense sand, they should be driven to refusal in bedrock. The depth of pile embedment into the sand depends on its density and percent fines; for budgeting purposes, we estimate piles should encounter refusal after penetrating 15 feet into the dense sand layer of the Colma Formation (approximate pile lengths should vary from 65 to 85 feet as measured from existing grade). If refusal is not encountered, the piles will gain their capacity primarily in friction; in this case, piles driven to a length of 90 feet below the pile cap should be adequate to support the loads presented in Tables 4 and 5. If, however, the blowcounts are low (less than about 12 blows per foot), the friction is likely lower than calculated and we may recommend that piles be driven deeper. This will be evaluated on a case-by-case basis, if needed. On the basis of our past experience at sites in the vicinity, we believe 14-inch-square, prestressed precast concrete piles would be the most economical driven pile type, although their design will need to account for the variability expected. Steel piles would also be appropriate.

Because of the settlement caused by placement of new fill, the piles may experience downdrag loads depending on the amount of fill that will be placed and the thickness of the Bay Mud layer. Downdrag due to negative friction is an additional load transferred to the pile as the Bay Mud surrounding the pile consolidates under the fill load. The downward movement of the compressible soil layer and the soil above it imposes negative frictional stresses on the pile. Consequently, if downdrag loads are predicted, the piles should be designed to support downdrag loads in addition to the building loads to prevent excessive movement.

During an earthquake, the fill will liquefy and lose strength which in turn will reduce the lateral capacity of the pile. Pile caps, grade beams, and skirt walls may be used for lateral resistance in the non-liquefiable fill above the water table. If there is insufficient lateral capacity, additional piles may be installed or the fill can be improved to mitigate the potential for liquefaction. Soil improvement methods include rapid impact compaction (RIC), stone columns, compaction grouting, deep dynamic compaction (DDC), and jet grouting. Several of these methods have been used at sites within Mission Bay to improve the soil density and reduce liquefaction induced settlements. If noise and vibrations are not desirable at the site or its surroundings, then only stone columns, jet grouting, or compaction grouting should be considered. Soil improvement methods are discussed in Section 8.2.

The ability of a pile to resist lateral loads is directly related to the stiffness of the pile, the stress-strain characteristics of the upper 10 to 20 feet of soil below the pile cap, and the allowable pile deflection.

Additional lateral load resistance can be obtained by passive resistance acting against the face of below-grade elements, such as pile caps, grade beams, interior key walls, and perimeter skirt walls. The amount of passive resistance will depend on the depth of the pile caps, grade beams, and walls. The design of below-grade elements will need to take into account the estimated settlements. Negative frictional stresses will be induced on key and skirt walls by the downward movement of fill caused by settlement; the foundation design should account for these additional downdrag loads.

7.2.1 Foundation Settlement

Where the piles will transfer building loads to relatively incompressible bedrock some elastic settlement will occur. Shorter piles bearing within the dense sand will experience settlement as a result of the settlement of the underlying clay created by foundation stresses. We estimate the total settlement of piles meeting refusal in the sand or rock should range between approximately 1/2 and 1 inch, depending on the length of the pile and the consistency of the supporting soil. Most of the settlement of piles bearing in bedrock is anticipated to occur during construction. Differential settlement could be up to approximately 1/2 inch between adjacent columns supported on new piles.

7.3 Soil Corrosivity

A corrosion study for Blocks 30 and 32 was performed by JDH Corrosion Consultants and the results were presented in a report dated 27 June 2007, see Appendix C. The following discussion omits details; therefore, the corrosion report should be read in its entirety. The report states that based on the results of in-situ testing the top 2.5 to 15 feet of soil at the site is classified as "corrosive" with respect to corrosion of buried cast/ductile iron and steel structures. The results of the chemical analyses indicate the soils are "severely corrosive" to "mildly corrosive" with respect to steel and ductile iron based upon resistivity measurements. The chloride levels indicate "severely corrosive" to "non-corrosive" conditions to steel and ductile iron. The sulfate levels indicate "non-corrosive" conditions for concrete structures placed into these soils with regard to sulfate attack. The pH of the soil indicates "non-corrosive" conditions to buried steel and concrete and the Redox potentials indicate aerobic conditions which are classified as "non-corrosive" to buried steel structures.

Steel elements placed below grade will corrode; protection of foundations, utilities, and other structural elements, which extend into these layers, will be required. The report indicates piles should be designed using Type II cement with a maximum water-to-cement ratio of 0.35 and minimum depth of cover of two inches over the prestressing wires. Also, a mineral admixture should be added to the concrete mix.

The corrosion report recommends reinforced concrete slabs or footings be constructed using a Type II cement mix with a maximum water-to-cement ratio of 0.40 and minimum depth of cover for the reinforcing steel of three inches, as discussed in Appendix C. If a mineral admixture is added in accordance with the recommendations in Appendix C, slab foundations/footings should be designed for a maximum water-to-cement ratio of 0.45.

Corrosion will adversely affect utilities and foundation elements. Corrosion control measures will be required, as discussed in Appendix C.

7.4 Groundwater

During our investigation, measured groundwater levels ranged from Elevation 90.5 feet to Elevation 92.4 feet. We conclude a design high groundwater elevation of +96 feet (SFCD + 100 feet) is appropriate; however, during construction, it is likely the water level will be deeper. This design groundwater has been measured at a nearby site during the last winter rainy season.

7.5 Construction Considerations

The fill is easily remolded and loses strength when wet. Therefore, site preparation and grading may be difficult if performed during the rainy season.

Serpentinite was encountered in the fill. Serpentinite often contains naturally occurring asbestos, and it is difficult and costly to dispose of, whether it contains asbestos or not. Also, because of health risks associated with breathing asbestos fibers, special handling and/or disposal procedures may be required if this material is encountered during construction.

The presence of boulders in the fill may make it difficult to make excavations for utility trenches and elevator pits, or to predrill pile locations. Brick, concrete, and other building rubble may also be present in the fill. Their presence may add to the difficulty of excavating and predrilling.

Depending on the time of year the work is performed, groundwater may be relatively shallow throughout the site. Excavations should be dewatered as needed to install utilities and compact soil. Because gravel and loose rubble have been found in the fill, there is a potential for significant water inflow into any excavation. In these areas, water impermeable shoring walls, such as sheet piles, may be required. Any excavation below the water table will require a site-specific dewatering plan.

The driving of displacement piles will cause the ground to heave. It is difficult to estimate the amount of heave; however, it could be on the order of several inches. Even with predrilling, heave may occur and adversely affect adjacent improvements. A pre-construction survey and monitoring during pile driving should be undertaken to monitor these effects.

At nearby sites, the piles driven into bedrock experienced erratic driving behavior. The Franciscan mélange contains significant clay matrix with blocks of rock. Where the piles are driven in the matrix, refusal is unlikely and the piles will provide support by friction. These conditions should be expected at random locations during driving and the contractor should be prepared to splice added sections.

8.0 RECOMMENDATIONS

From a geotechnical standpoint, the site can be developed as planned, provided the recommendations presented in this section of the report are incorporated into the design and contract documents. Criteria for foundation design, together with recommendations for site preparation, floor slabs, fill placement and seismic design are presented in this section of the report.

8.1 Pile Foundations

The building should be supported on a pile foundation gaining its capacity below the Bay Mud in the dense Colma sand or bedrock. All piles should be driven to refusal, or if refusal is not encountered, the piles will gain their capacity primarily in friction; in this case, piles driven to a length of 90 feet below the pile cap should be adequate to support the loads presented in Tables 4 and 5. If, however, the blowcounts are low (less than about 12 blows per foot), the friction is likely lower than calculated and we may recommend that piles be driven deeper. This will be evaluated on a case-by-case basis, if needed. All foundations should be designed for the corrosive conditions, as recommended in Appendix C.

8.1.1 Axial Capacity

The allowable axial capacity of the piles depends on the downdrag load. If no fill is added to the site, downdrag forces will not be present. Where fill is added, downdrag forces will vary throughout the site due to variable thickness of fill and Bay Mud; two downdrag zones, designated Zone A and Zone B, were created as shown on Figure 10. Our recommended pile capacities for concrete piles if no fill is added are presented in Table 4.

TABLE 4

Recommended Pile Capacities
Precast, Prestressed Concrete Piles
No New Fill

Pile Zone	Assumed Bottom of Slab Elevation[1] (feet)	$Q_{ultimate}$ Axial Capacity (kips)	Q_{actual} Downdrag (kips)	$Qnet_{allowable}$ Dead Plus Live Loads (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift (kips)
A & B	≤ 100.5	630	0	250	330	50

1 Elevation based on San Francisco City Datum plus 100 feet.
2 Factors of Safety of 2 and 3 were used for friction and end bearing, respectively.

If more than 6 inches of fill is added to the site, a downdrag load will be present. Our recommended pile capacities for concrete piles if greater than six inches of fill is added is presented in Table 5.

TABLE 5

Recommended Pile Capacities
Precast, Prestressed Concrete Piles with New Fill

Pile Zone	Bottom of Slab Elevation[1] (feet)	$Q_{ultimate}$ Axial Capacity (kips)	Q_{actual} Downdrag (kips)	$Qnet_{allowable}$ Dead Plus Live Loads (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift (kips)
A	>100.5	630	100	200	270	50
B	>100.5	630	45	225	305	50

1 Elevation based on San Francisco City Datum plus 100 feet.
2 Factors of Safety of 2 and 3 were used for friction and end bearing, respectively.

The ultimate capacities shown represent the strength of the soil and the interaction between the soil and the pile. The dead and live loads plus the downdrag load should not exceed the structural capacity of the pile. The structural engineer should check that the structural capacity of the pile is not exceeded.

To avoid axial compression capacity reduction created by group effects, piles should be spaced at least three pile widths apart, measured center to center.

8.1.2 Lateral Load Resistance

The piles should develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on:

- the pile stiffness
- the strength of the surrounding soil
- axial load on the pile
- the allowable deflection at the pile top and the ground surface
- the allowable moment capacity of the pile.

The lateral capacity of piles will be significantly increased if the potential for liquefaction is mitigated, as stated in Section 8.2. We have calculated the lateral capacity based on 1/2-inch lateral deflection at the top of pile for 14-inch-square prestressed, precast concrete piles with fixed and free head for two conditions: "With Liquefaction" and "No Liquefaction." The "With Liquefaction" case applies if the site is not improved, whereas the "No Liquefaction" case is for piles in areas of the site that have been improved. The lateral load for each case and the moment verses depth profiles for 1/2 inch of lateral deflection and vertical load of 250 kips are presented on Figures 11 and 12. Our analysis assumes pile caps will extend approximately five feet below the floor slab.

The lateral capacities are for a single pile only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown on Figure 13. The reduction factors are based on a minimum pile spacing of three widths. The moment profile for a single pile with an unfactored load should be used to check the design of individual piles in a group.

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. A passive resistance of 130 psf (rectangular distribution) may be used to compute passive resistance above the water table for the "With Liquefaction" case. For the "No Liquefaction" case 250 pcf (triangular distribution) up to a maximum of 2000 psf may be used to compute passive resistance above the water table. These values include a factor of safety of 1.5. To account for lack of confinement and settlement, the lateral resistance should be ignored in the upper 1-1/2 feet of the pile cap.

8.1.3 Construction Considerations

We recommend an indicator pile program be performed to provide data for choosing production pile lengths. Indicator piles may be installed at column locations and can be used for support of the building. We recommend at least 27 indicator piles be driven. Indicator piles should be installed at production pile locations selected by us and approved by the structural engineer. They should be installed with the same equipment that will be used to install the production piles.

It is difficult to accurately predict the depth of embedment into the dense sand before the piles will achieve refusal. Also, it is difficult to determine which piles will punch through the sand, but will encounter refusal in bedrock. The dense sand layer has variable thickness, percent fines, and density. Therefore, to accommodate the variations in the sand and attempt to limit the required cutoff, the pile capacities are based on the strength of the sand. During indicator pile driving, however, we will attempt to penetrate the dense sand layer with some piles by driving the piles hard, and drive piles to refusal in bedrock. We will use this data to further define the presence and thickness of the dense sand layer, as well as to evaluate depth to and hardness of rock. Therefore, we recommend the lengths of 17 indicator piles be chosen to extend at least 10 feet into bedrock, with tip elevations of about 57 to -35 feet (approximately 60 to 115 feet long). Bedrock elevation contours are shown on Figure 6. Cutoff lengths up to 60 feet should be anticipated. The remaining 10 indicator piles should be cast in lengths sufficient to extend to the bottom of the very dense sand. In general, the 17 longer piles should be driven first, followed by the shorter piles.

Determining the driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length, and soil conditions. All piles should be driven continuously to refusal using a hammer that can deliver sufficient energy to the tip of the piles to drive them efficiently without damage. To reduce pile damage, the hammer should be throttled down or otherwise prevented from striking with full energy while driving through the Bay Mud layer.

Refusal blow count criteria should be determined in the field after indicator pile driving. On a preliminary basis, we judge the refusal blow count using a hammer with a maximum rated energy of 85,000-foot-pounds on fuel setting 4 would be approximately 35 blows per foot in the dense sand layer. To maintain vertical alignment and provide better control during driving, fixed leads should be used and the plumbness of piles should be checked often during driving.

We recommend the contractor perform a Wave Equation Analysis of Pile (WEAP) for the proposed pile-hammer combination prior to the indicator pile installation to evaluate the potential pile driving situation including the use of a follower, as appropriate. We should review the results prior to driving indicator piles. We also recommend attaching pile driving analyzer (PDA) transducers to twelve indicator piles selected by us before driving. The pile integrity and dynamic capacity of these piles should be monitored with the PDA during initial driving and retap. A Case Pile Wave Analysis Program (CAPWAP) should be performed on one representative restrike blow on these twelve indicator piles. The restrikes should be performed at least 72 hours after initial drive, although one week is preferred.

There are existing foundations, rubble and boulders in the fill; pile locations should be predrilled to the bottom of the fill. Predrilling will reduce the potential for damage and will help the contractor maintain pile alignment. If a predrill auger larger than the least pile dimension is used, the annular space between the pile and the auger hole should be backfilled with pea gravel or lean concrete. If the void is not properly backfilled the lateral capacity of the piles will be reduced. To reduce the amount of spoils, the predrilling should not extend more than a few feet into the Bay Mud.

8.2 Mitigation of Liquefaction Hazards

If the available lateral pile capacity for the "With Liquefaction" case is not sufficient, site improvement should be performed to increase the lateral capacity and decrease the potential for liquefaction-induced settlement. Liquefaction potential can be mitigated by densifying the soil using appropriate soil improvement methods and/or by providing drainage. Soil improvement methods, including rapid impaction compaction (RIC) and compaction grouting, densify the liquefiable soil, thereby decreasing the liquefaction potential. Stone columns improve the soil and provide rapid drainage.

8.2.1 Compaction Grouting

Compaction grouting consists of pumping a low slump (less than two inches) grout mix under high pressure through steel grout pipes. The low slump grout displaces the loose sand, which pushes more sand into less volume, thereby increasing its density. The grout columns also act to reinforce the soil as vertical members. The compaction grouting improvement technique may be problematic because the layer that should be densified is shallow. Pumping grout under pressure where low overburden pressures exist may push the grout to the surface, or cause heave, as opposed to compacting the soil laterally. Field verification of the level of improvement is necessary to check that the improved conditions meet the desired results, as discussed in section 8.2.4.

8.2.2 Stone Columns

Installation of stone columns is a ground improvement technique that results in in-situ densification and provides rapid drainage of granular soil. Stone column installation is accomplished using large, powerful, vibrating probes that are inserted to the desired depth of improvement and withdrawn. The voids created through densification are backfilled with gravel or crushed rock and compacted while withdrawing the probe, leaving a dense stone column typically 3 to 4 feet in diameter surrounded by densified soil. Stone columns also serve as drains to allow rapid dissipation of pore pressures which may develop in adjacent soil during an earthquake. The vibratory probe method of installing stone columns is effective in sandy soil with less than about 25 percent fines; the sand fill at this site generally contains 15 percent fines or less.

Field verification of the level of improvement is necessary to check that the improved conditions meet the desired results, as discussed in section 8.2.4. Settlements of six or more inches should be anticipated during the installation of the stone columns. The placement of fill to bring improved areas to final grade will cause consolidation of the Bay Mud; therefore, additional settlement of the ground surface will occur.

8.2.3 Rapid Impact Compaction

Rapid Impact Compaction (RIC) is a method of soil improvement using a track-mounted machine that imparts energy by dropping an approximately 7.5-ton weight from a controlled height, about one meter, onto a patented foot. The energy is delivered at a rate of 40 to 60 blows per minute. Drop height, number of blows, and penetration per blow are monitored and/or controlled by an on-board data acquisition system. Compaction points are performed on a geometric grid, the spacing of which is determined based on the properties of the soil to be densified. Craters will be created if RIC is performed and import soil will be required to raise the subgrade to the initial elevation.

On the basis of recent experience at a site in Mission Bay, we recommend that production RIC treatment consist of 13 compaction points per 20-foot by 20-foot area. The RIC should be performed in intermittent zones measuring 40 feet by 40 feet in plan to avoid large areas of pore pressure increase that may potentially result in loss of soil strength. Every other zone should be skipped during the initial treatment.

Where craters deeper than 18 inches are formed by RIC, the area should be retreated with an additional 13 compaction points. The retreatment should be performed no sooner than 24 hours after the initial treatment to allow pore pressures to dissipate. The bid should provide a unit price (on a square-foot basis) to retreat areas; however, the base bid should assume no recompaction is required. The requirement for recompaction will be based on crater depth only, and will be based on correlation with crater depth and the confirmation CPT from the test section.

8.2.4 Soil Improvement Verification

Any liquefaction mitigation method should be performed under our observation. Field verification of the level of improvement is necessary to check that the improved conditions meet the desired results, as discussed in the following section.

Regardless of the soil improvement method used, we recommend a test section be performed. The test section should be on the order of 30 feet by 30 feet in plan dimension. The test, including the liquefaction testing, should be performed prior to driving the production piles. The improved fill (where classified as sand, clayey sand, or silty sand) should have minimum SPT blowcounts [$(N_1)_{60\text{-}CS}$],[7] over three continuous SPTs, of at least 20 blows per foot (bpf) and average SPT blow counts of 25 bpf. If CPTs are used for confirmation, minimum and average tip resistances [$(q_{c1N})_{CS}$],[8] over an interval of three feet, should be at least 80 to 100 tons per square foot (tsf), respectively. The above criteria may need to be reevaluated depending on the soil type encountered.

8.3 Floor Slabs

Because consolidation settlement will occur and liquefaction-induced settlement is predicted during an earthquake, we recommend the ground floor slab of the building be designed to span between pile caps and/or grade beams, and thus not rely on the fill for support. Entrances to the building should be

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[7] The $(N_1)_{60\text{-}CS}$ is N-value that has been normalized to an overburden pressure of one tsf and corrected to account for the effects of fines content.

[8] The $(q_{c1N})_{CS}$ is tip resistance that has been normalized to an overburden pressure of one tsf and corrected to account for the effects of fines content.

designed to transition from areas of structural support to areas of no support where between 0 and 5 inches of static settlement, depending on the bottom of slab elevation (per section 7.1) and an additional 6-1/4 inches of seismically-induced settlement could occur. Alternatively, building entrances may be designed to accommodate static settlement only. When an earthquake and subsequent liquefaction-induced settlement occurs, the entrances slabs may be replaced or releveled as needed.

Initially, the slab will be in contact with the ground. Moisture is likely to condense on the underside of the ground floor slabs, even though it will be above the design groundwater table. Consequently, a moisture barrier should be considered if movement of water vapor through the slab would be detrimental to its intended use. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 6.

TABLE 6

Gradation Requirements for Capillary Moisture Break

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

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

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, we judge it would be prudent for the floor slab concrete mix to be designed with a low w/c ratio, less than 0.50. If concrete slabs or footings are more than a few feet deep and are in contact with the soil, they should be designed for the water-to-cement ratio recommended in the corrosion study located in Appendix C. If approved by the project structural engineer, the sand can typically be eliminated and the concrete placed directly over the vapor retarder, provided water is not added in the field. If necessary, workability may be increased by adding plasticizers.

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

8.4 Seismic Design

The site is in Seismic Zone 4. In its current condition, the fill is liquefiable; therefore, the site is classified as a Soil Profile Type S_F, in accordance with SFBC. A site-specific response spectrum is required for S_F sites. As discussed in Section 6.5, we performed a PSHA and we recommend the structure be designed in accordance with the horizontal response spectra presented on Figure 9. Details of our analyses are presented in Appendix E.

8.5 Excavation

Where space permits, the sides of excavations can be sloped. Temporary excavation slopes should be no steeper than 1-1/2:1 (horizontal to vertical) in the fill above the water table. Where space does not permit a sloped excavation and where excavations extend below five feet, shoring will be required. Excavations in Bay Mud should be shored.

If water seepage is encountered during excavation, dewatering measures, such as placing pumps in sumps in the bottom of the excavation, should be employed. There is currently a fee imposed by the City and County of San Francisco Public Utilities Commission for discharge of construction-generated water into the combined system.

8.6 Site Preparation

All concrete and asphalt pavements and other existing improvements within the areas to be developed should be removed during site demolition. All topsoil and organics should be removed from the footprint of structural fill or improvements, and may be stockpiled for use in landscaped areas, if approved by the architect.

Existing foundations may adversely affect the construction and performance of new improvements. Existing piles and pile caps will create "hard spots" which will cause differential settlement of the ground surface. They should be removed to a sufficient depth to reduce adverse effects. The removal depth depends on the site conditions, new foundation types, and desired performance.

We recommend existing foundations be removed to the bottom of new pile caps, structural slabs, and utilities within the building footprint. All pile caps and footings should be completely removed beneath new slabs-on-grade, pavements, sidewalks, and landscaped areas. In general, single piles beneath these elements should be removed to a depth of four feet below final soil subgrade. Piles in groups should be removed to a depth of eight feet beneath final soil subgrade.

Existing utilities to be abandoned which are greater than six inches in diameter should be removed within the depth of new pile caps within building footprints. Existing utilities greater than six inches and deeper than pile caps with the building footprint should be capped at each end to prevent water accumulation. Utilities less than six inches may be left in place within pile-supported building footprints.

Existing utilities within three feet below soil subgrade outside of building footprints should be removed. Existing utilities greater than six inches and deeper than three feet should be capped at each end. Utilities less than six inches and deeper than three feet may be left in place.

8.7 Earthwork

The surface exposed by stripping and/or excavation should be:

- scarified to a minimum depth of six inches
- moisture conditioned to near optimum
- compacted to at least 90 percent relative compaction.[9]

If soft areas are encountered, the soft material should be removed and replaced with either lean concrete or engineered fill. Excavations made to remove existing foundation elements and utilities should be filled with lean concrete or properly compacted fill. Where the bottom of these overexcavations are near or below the water table, it should be covered with a geotextile overlain by at 1/2- to 3/4-inch crushed rock to a minimum of six inches above the groundwater to provide a more stable base for backfill. Fill can then be backfilled and recompacted according to our recommendations.

All fill should be placed in horizontal layers not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction.

To reduce future maintenance of private streets, plazas, and sidewalks, we recommend street and sidewalks sections be underlain by at least two feet of engineered fill. In cut areas, or where less than two feet of new fill will be placed, existing grade should be overexcavated at least 18 inches below sidewalks or street subgrade or to two feet above the groundwater, whichever is less. The excavation surface should be scarified to a depth of at least six inches, moisture-conditioned, and recompacted to at least 90 percent relative compaction. New fill should be placed in eight-inch-thick loose lifts and compacted to at least 90 percent relative compaction. The final six inches of subgrade and all of the aggregate base beneath exterior slabs and pavements should be rolled to expose a firm non-yielding surface and compacted to at least 95 percent relative compaction.

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[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.

From a geotechnical standpoint, most on-site soil free of organic matter and rocks or lumps larger than four inches in greatest dimension should be suitable for use as fill or backfill provided it is properly moisture conditioned. Bay Mud is not a suitable material for use as fill.

Imported fill material should also be free of organic debris and rocks or lumps larger than four inches in greatest dimension. All material to be used as fill should have a low expansion potential, defined by a liquid limit less than 40 and a plasticity index (PI) lower than 12. Samples of all imported fill should be submitted to the geotechnical engineer for testing at least 72 hours before delivery to the site.

8.8 Below-Grade and Retaining Walls

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

TABLE 6

Lateral Earth Pressures

Site Condition	Soil Type	Unrestrained Walls	Restrained Walls	Seismic Conditions[1]	Passive Resistance[2]
Site Not Improved - Fill Liquefiable[3]	Fill above water table[4]	40 pcf	60 pcf	40 pcf + 11H psf	130 psf
	Fill below water table	85 pcf	100 pcf	100 pcf + 11H psf	130 psf
Site Improved - Fill Not Liquefiable	Fill above water table	40 pcf	60 pcf	40 pcf + 15H psf	250 pcf
	Fill below water table	85 pcf	95 pcf	85 pcf + 15H psf	125 pcf
Both Conditions	Bay Mud	90 pcf	90 pcf	75 pcf + 11H psf	500 psf

1 Wall should be designed for the more critical loading condition of restrained or seismic conditions.
2 Passive resistance includes a factor of safety of 1.5.
3 Wall should be designed for liquefiable case unless site improved.
4 Design groundwater is Elevation 96 feet.

The pressures summarized in the table assume the ground surface behind the wall is horizontal. 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 walls 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 (Elevation 96 feet). We should check the manufacturer's specifications regarding the proposed prefabrication drainage panel material to verify it is appropriate for its intended use.

An acceptable alternative is to backdrain the wall with drain rock, at least one foot wide, extending down to the design groundwater elevation. Filter fabric should be placed between the gravel drain and the soil. This system is usually not used where shoring is the backside form for the walls.

Below-grade walls that are not drained should be designed for the pressures given for "Fill below the water table" within the entire depth of fill. To prevent against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints.

Landscape site amenity walls that are not pile supported should be supported on continuous footings at least 16 inches wide or isolated spread footings at least 24 inches wide. Footings should be founded at least 18 inches below the lowest adjacent soil subgrade. Footings may be designed using an allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads. The allowable bearing pressure may be increased by one-third for total loads, including wind or seismic forces. Depending on the location of the walls, settlement may occur. We should check the settlement based on the location and actual bearing pressures.

The excavations for the wall footings should be free of standing water, debris, and disturbed materials prior to placing concrete. The footing subgrade should be rolled to a dense, non-yielding surface before placement of the reinforcing steel. The bottoms and sides of excavations should be maintained in a moist condition until concrete is placed. We should check foundation excavations prior to placement of reinforcing steel to confirm suitable bearing material is present.

8.9 Utilities

Utilities should be designed to accommodate the predicted settlement. Hangers and flexible connections may be used. The hangers should be corrosion resistant. Where utilities are hung, they should be backfilled with pea gravel, to allow the ground to settle without loading the utilities. However, because of the flowable nature of pea gravel, it cannot be relied upon to provide lateral load resistance against pile caps or grade beams; therefore, where passive resistance against an adjacent pile cap or other structural element is being relied upon, all trenches within five feet of pile caps should be backfilled with properly compacted soil and the hanger spacing design should account for the soil loading. Flexible connections, which allow for approximately 11-1/4 inches of differential movement (where utilities enter the building), should be used as needed. If it is desired to only plan for static settlement, flexible connections allowing for 5 inches of differential movement may be used.

The existing fill is corrosive. Corrosion control measures, such as coatings, and/or polyethylene encasement, supplemented with cathodic protection, should be used to protect direct buried metallic pressure piping. All underground pipelines should also be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to minimize potential galvanic corrosion problems. For more detail, see the recommendations by JDH Corrosion Consultants in Appendix C. A corrosion consultant should be retained during utility design.

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, and compacted to at least 90 percent relative compaction. Where sheet piling is used as shoring and is to be removed after backfilling, it should be placed a minimum of two feet away from the pipes or conduits to prevent disturbance to them as the sheet piles are extracted. It may be difficult to drive sheet piles through rubble in the fill. Where trenches extend below the groundwater level, it will be necessary to dewater them to keep the trench base from softening and to allow for placement of the pipe utilities and backfill.

Backfill for utility trenches should be compacted according to the recommendations presented for general site fill. Jetting of trench backfill is not permitted. The soil excavated from the trenches can be reused to backfill the trenches, provided the material can be compacted to the required compaction. If sand or gravel with less than 10 percent fines (particles passing the No. 200 sieve) is used, it should be compacted to at least 95 percent relative compaction. Drain rock and rod mill should be mechanically tamped in 12-inch lifts where placed beneath pavements; however, within the footprint of the pile-supported building, the backfill for utilities suspended from the slab should consist of uncompacted pea gravel or rod mill. 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.

8.10 Concrete Pavement, Exterior Slabs and Pavers

For all concrete flatwork, exterior slabs, and pavers, the subgrade should be proof rolled to provide a firm and non-yielding surface. Concrete flatwork may be placed directly on prepared subgrade; for better performance, however, four inches of aggregate base compacted to 95 percent relative compaction should be placed beneath the concrete.

Where rigid pavement is required, for loading and service areas, we recommend six inches of concrete for medium traffic and eight inches of concrete for heavy traffic. Loading and service areas should be underlain by six inches of Class 2 aggregate base compacted to 95 percent relative compaction. Aggregate base material should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications.

Pedestrian paver sections should consist of 60 to 80 millimeter pavers set on a one-inch thick sand bed on four inches of aggregate base, compacted to 95 percent relative compaction.

Paver sections for vehicular traffic are presented in Table 7.

TABLE 7

Vehicular Paver Section

TI	Paver Thickness (mm)	Class 2 Aggregate Base (inches)
5.0	80	9
6.0	80	12

Vehicular pavers should be set on a one-inch laying course of sand. Aggregate base should conform to Section 26-1.02A of the current Caltrans Standard Specifications. The thickness of aggregate base is based on an assumed R-value of 30 for the existing fill. During construction, this thickness may be revised if soil with a lower R-value is encountered.

For better performance beneath non-pile-supported flat work or pavers, we recommend the subgrade be prepared to provide at least 24 inches of engineered fill.

8.11 Asphalt Pavement

To evaluate pavement thicknesses, we relied upon the results of R-value testing from nearby sites. R-values of the typical Mission Bay fill, consisting of clayey sand with gravel to sandy gravel with clay, range from 19 to 65. We used an R-value of 30 in our design. If the subgrade soil is not similar to the typical fill, samples should be collected and tested, and if appropriate, the pavement section design should be revised. Table 8 presents our recommendations for vehicular and pedestrian asphalt concrete pavement.

TABLE 8

Pavement Section Design (Subgrade R-value of 30)

Pavement Type	Traffic Index	Asphalt Concrete Thickness (inches)	Class 2 Aggregate Base (inches)
Vehicular	5.0	2.5	6.5
Vehicular	6.0	3.0	8.5

Subgrade should be compacted in accordance to the recommendations in section 8.6. Class 2 aggregate base should be compacted to 95 percent relative compaction and proof-rolled to verify the material is firm and non-yielding. Aggregate base material should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications.

8.12 Site Drainage

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly adjacent to structures, or on roadways or pavements. Surface runoff should be directed away from foundations to properly designed and installed drop inlets.

9.0 ADDITIONAL GEOTECHNICAL SERVICES

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

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the site at the time of this investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Treadwell & Rollo, Inc. should be notified to make supplemental recommendations, as necessary.

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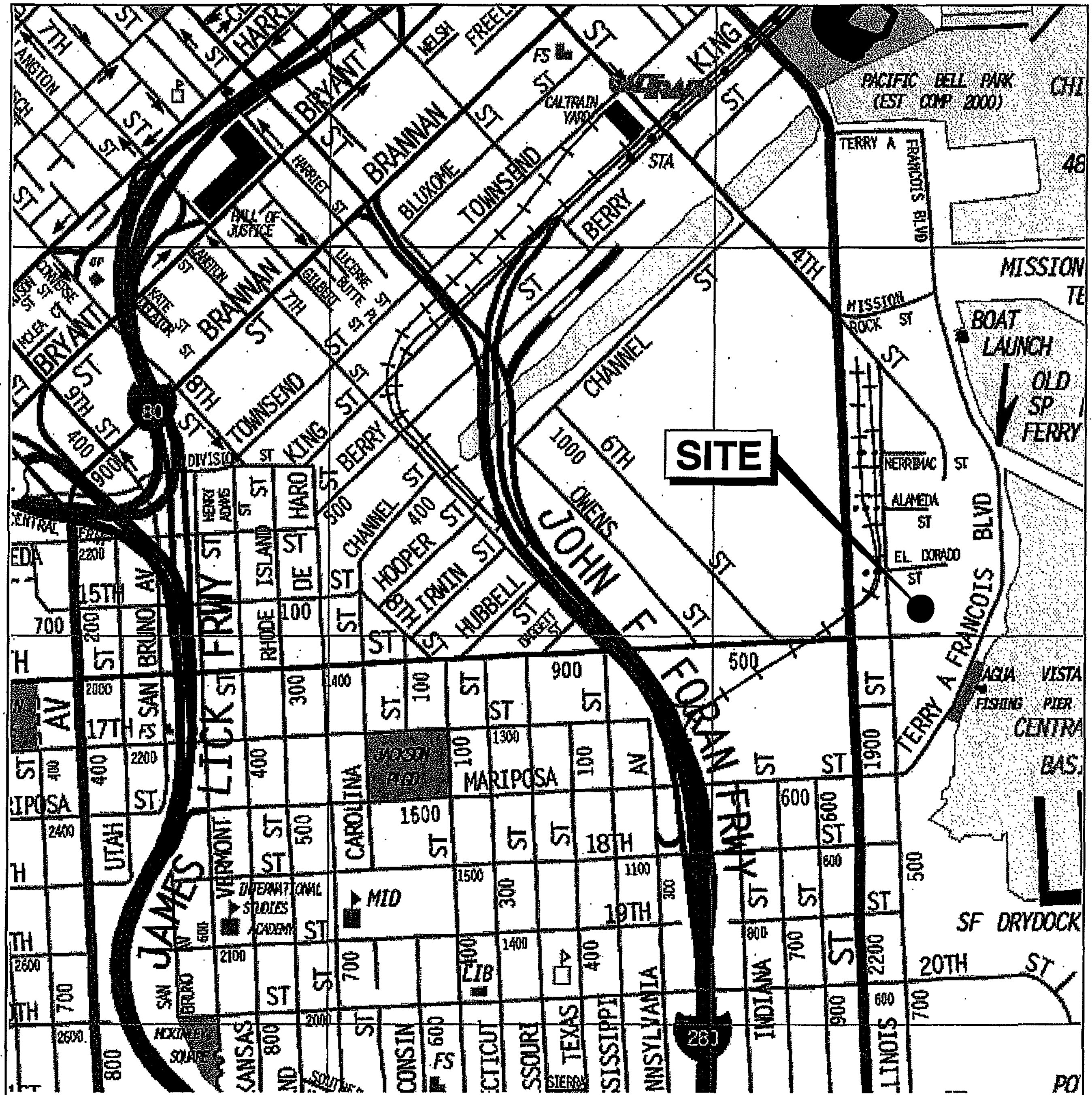
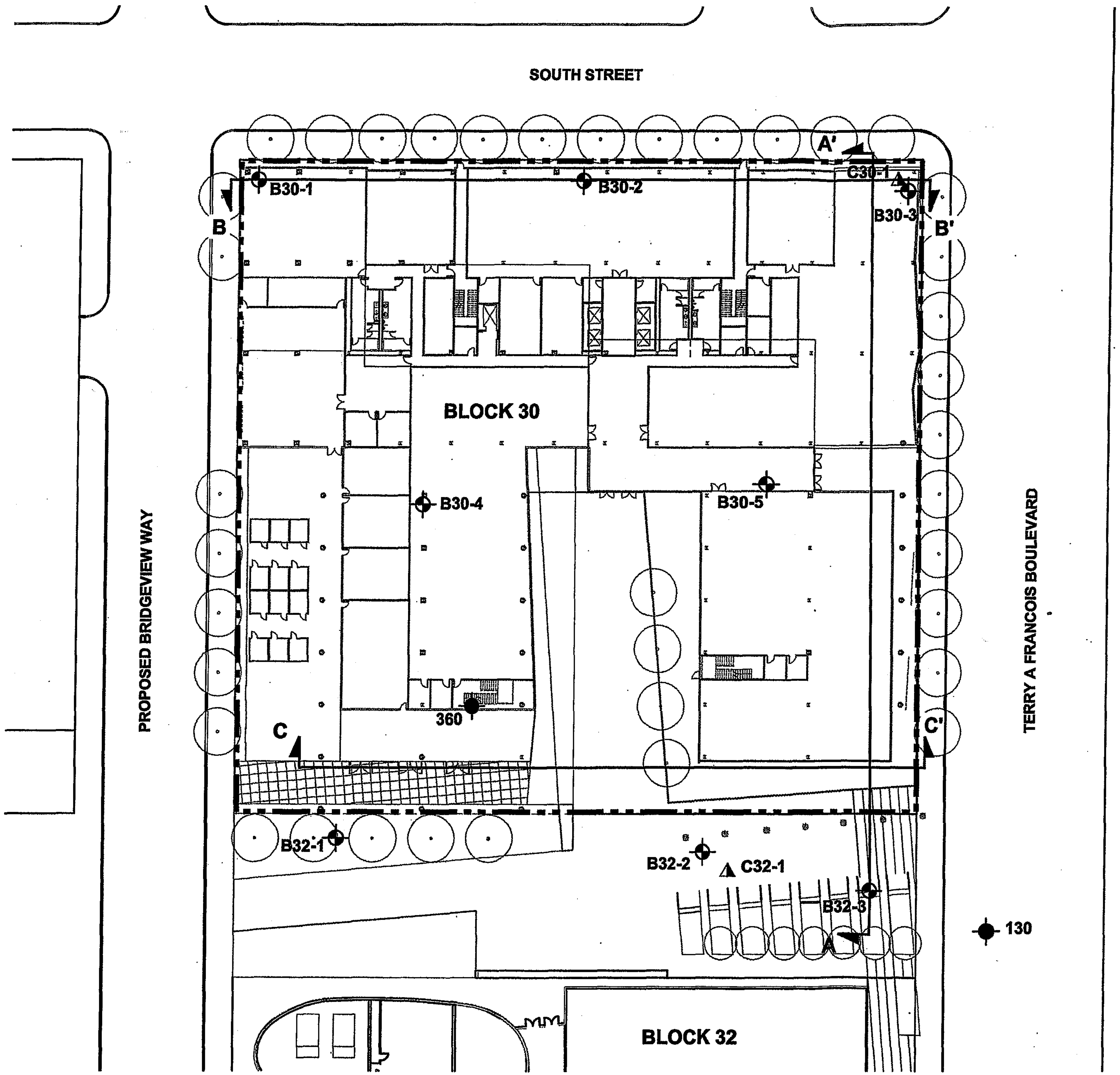
FIGURES



Base map: The Thomas Guide
San Francisco County
1999

No scale

BLOCK 30 MISSION BAY San Francisco, California	SITE LOCATION MAP		
Treadwell&Rollo	Date 09/25/07	Project No. 4086.16	Figure 1



EXPLANATION

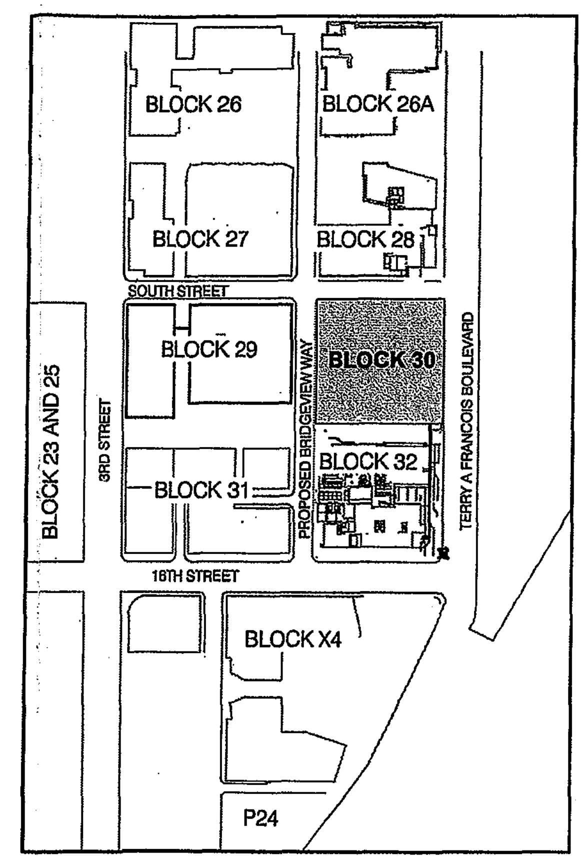
B30-1 Approximate location of boring by Treadwell & Rollo, Inc.

C30-1 Approximate location of cone penetration test by Treadwell & Rollo, Inc.

360 Approximate location of boring by Treadwell & Rollo, Inc. or others for previous investigations

A A' Idealized cross section location

Property line



VICINITY MAP

Not to scale

0 50 Feet

Approximate scale

BLOCK 30
MISSION BAY
San Francisco, California

SITE PLAN

Date 10/12/07	Project No. 4086.16	Figure 2

Treadwell&Rollo

Reference: PDF drawing "A2.01 Ground Floor Plan" e-mailed by SMWM 6/20/07.

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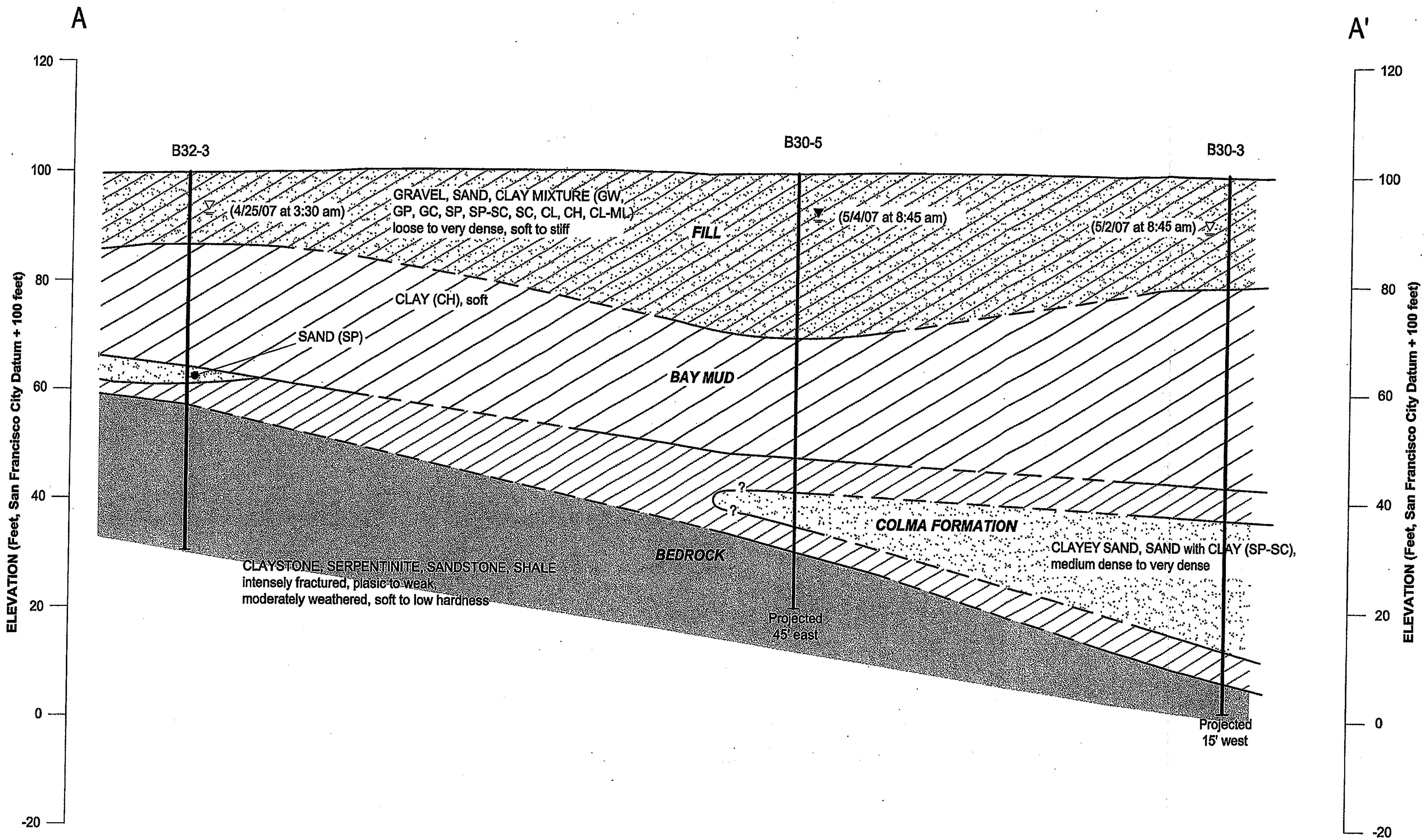

Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

BLOCK 30
MISSION BAY
San Francisco, California

IDEALIZED SUBSURFACE PROFILE A-A'

Date 09/25/07	Project No. 4086.16	Figure 3

Treadwell&Rollo

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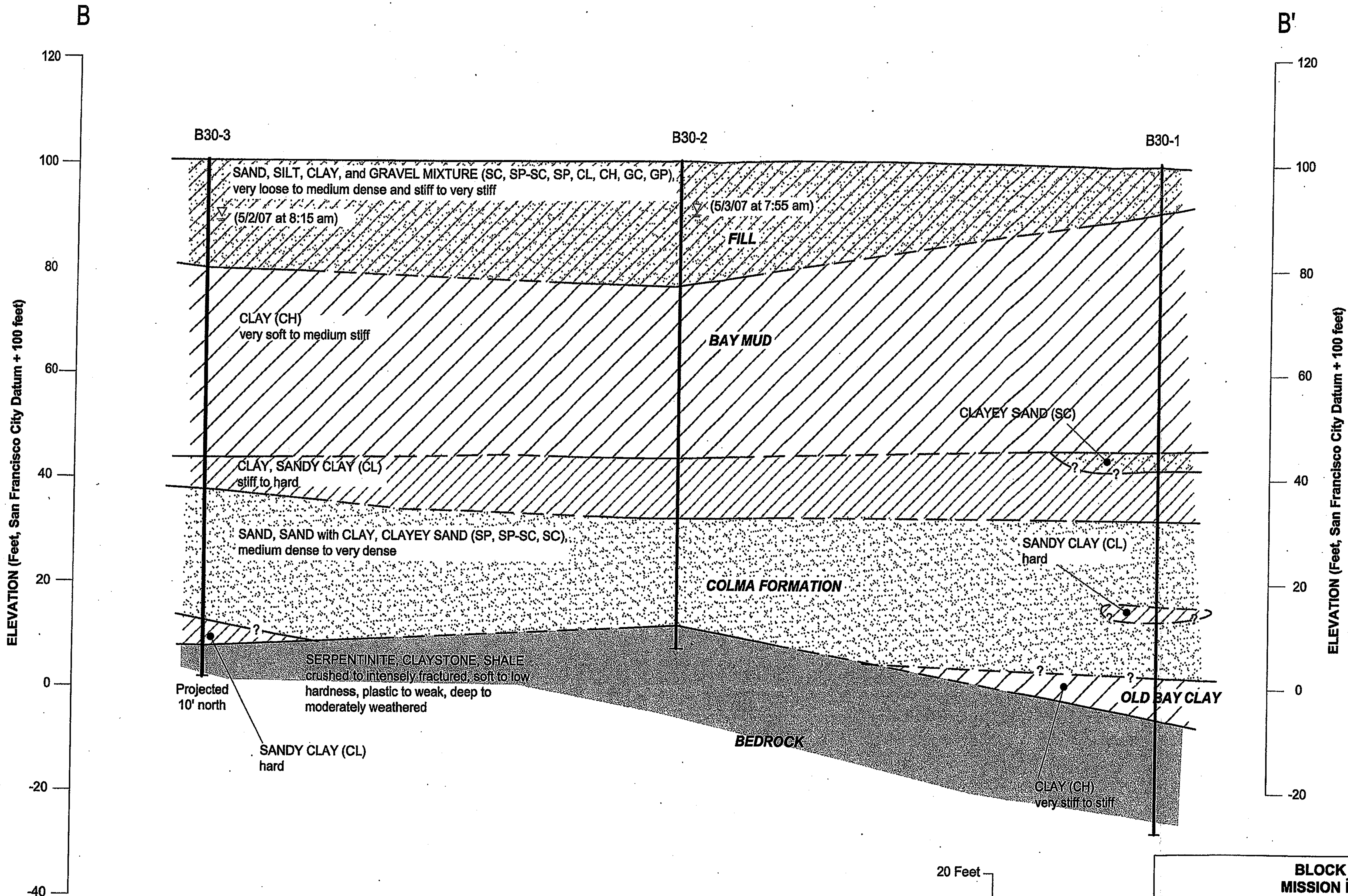
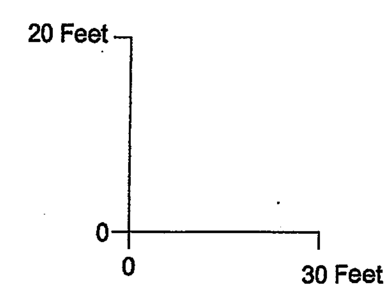


Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.



Approximate Scale

BLOCK 30
MISSION BAY
San Francisco, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 09/11/07	Project No. 4086.16	Figure 4

Treadwell&Rollo

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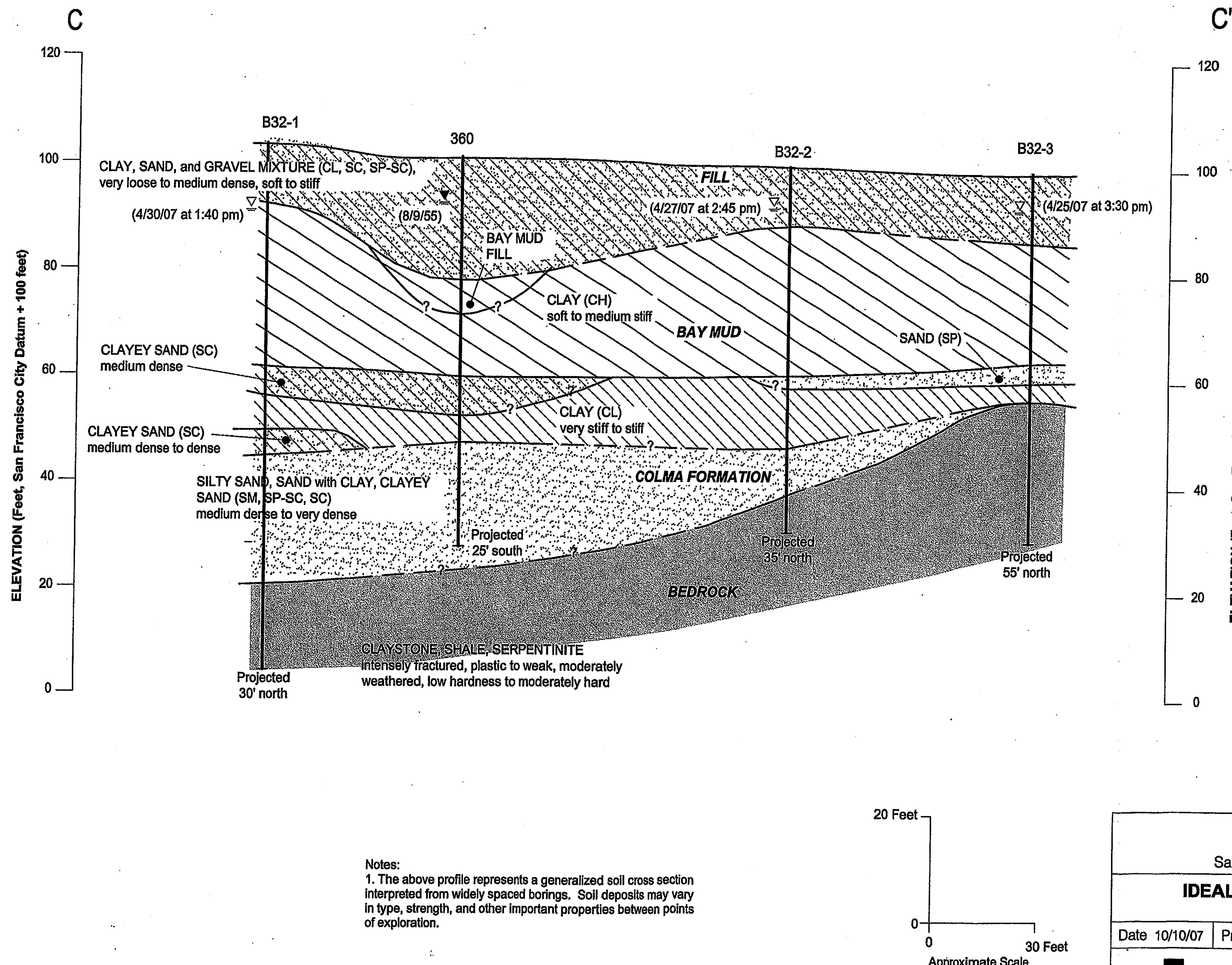

Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

BLOCK 30 MISSION BAY San Francisco, California		
IDEALIZED SUBSURFACE PROFILE C-C'		
Date 10/10/07	Project No. 4086.16	Figure 5

Treadwell&Rollo

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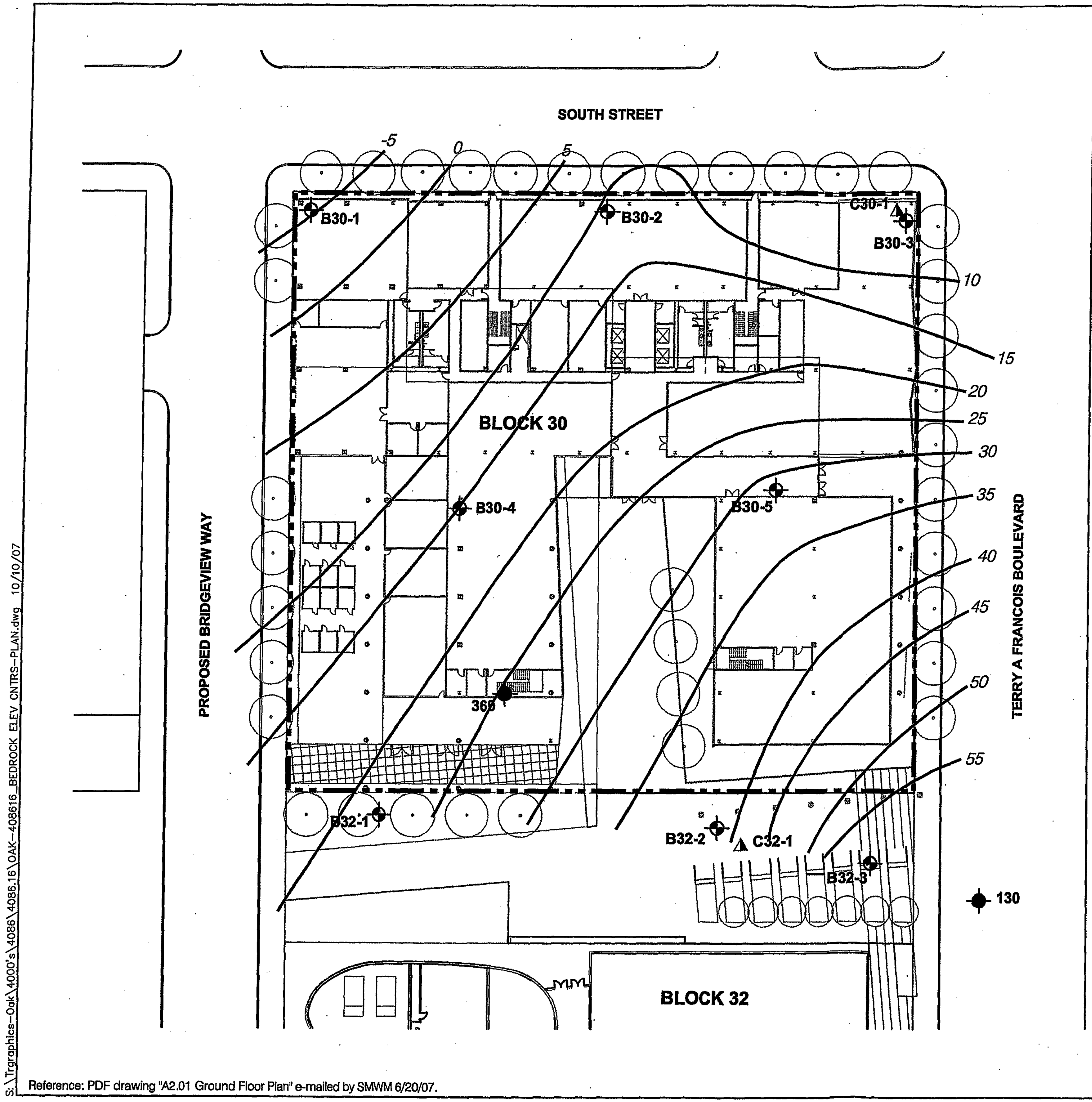
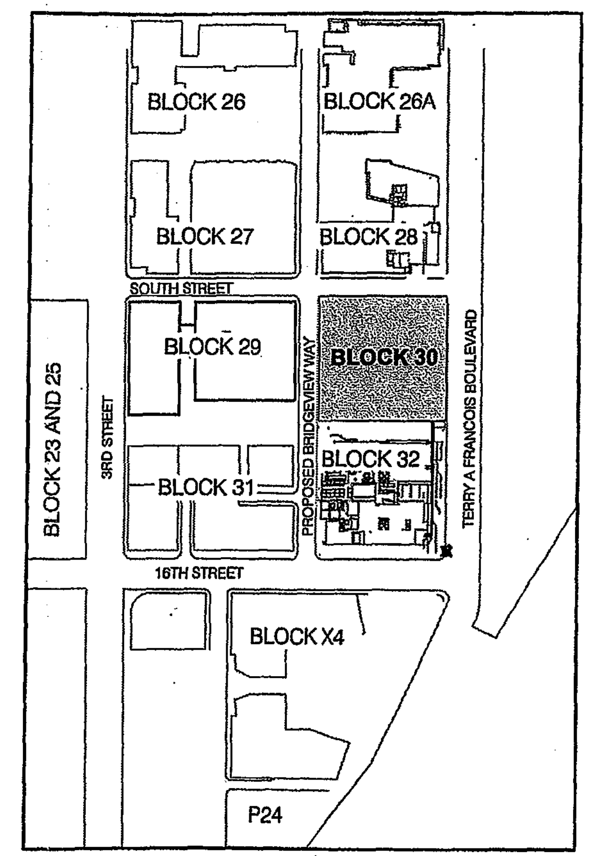


Reference: PDF drawing "A2.01 Ground Floor Plan" e-mailed by SMWM 6/20/07.

EXPLANATION

10	Estimated top of bedrock elevation contour, San Francisco City Datum plus 100 feet
B30-1	Approximate location of boring by Treadwell & Rollo, Inc.
C30-1	Approximate location of cone penetration test by Treadwell & Rollo, Inc.
360	Approximate location of boring by Treadwell & Rollo, Inc. or others for previous investigations
	Property line

Note: see text regarding driving behavior in serpentinite rock



VICINITY MAP

Not to scale

0 50 Feet

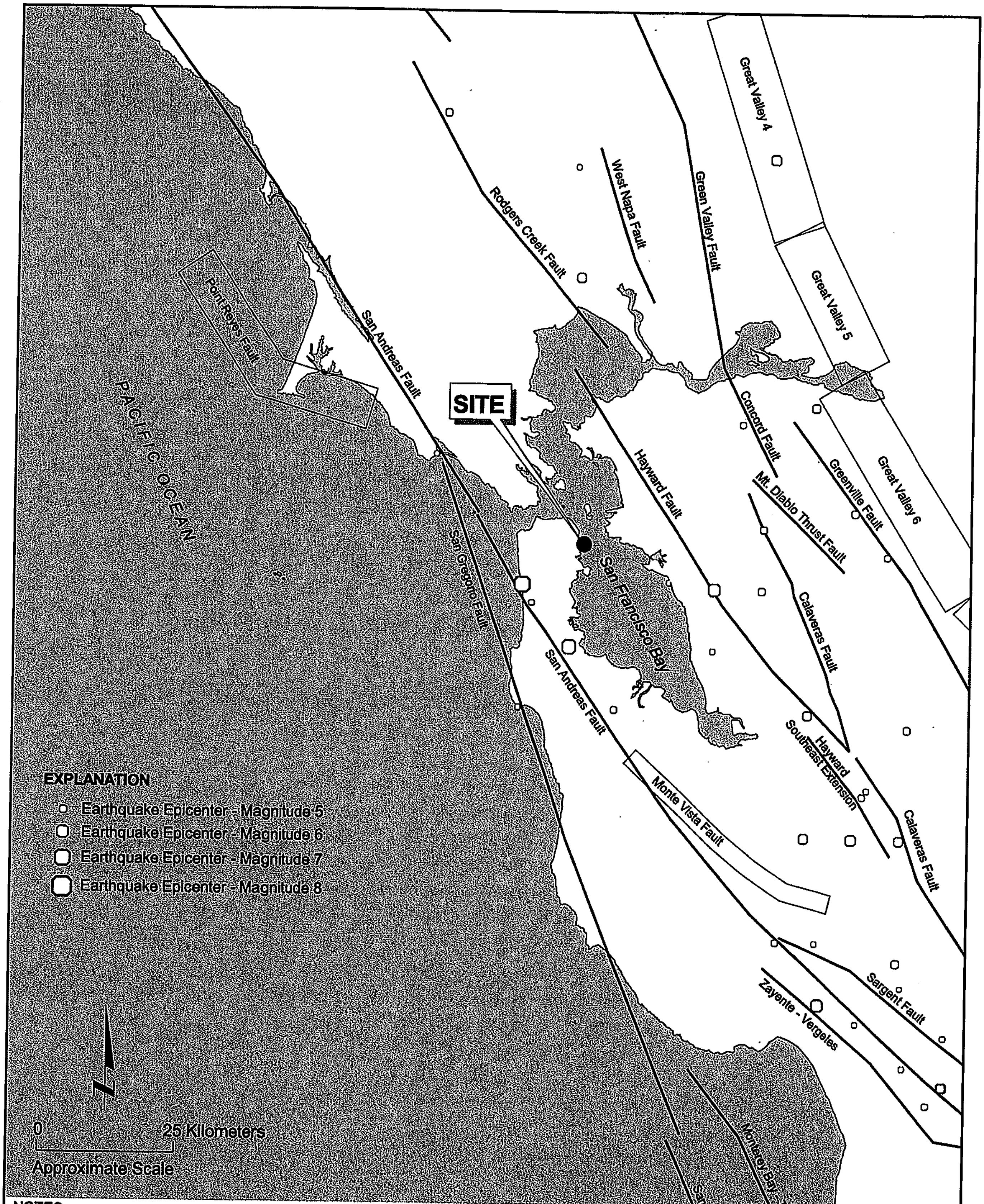
Approximate scale

BLOCK 30
MISSION BAY
San Francisco, California

BEDROCK ELEVATION CONTOURS

Date 10/10/07 | Project No. 4086.16 | Figure 6

Treadwell&Rollo

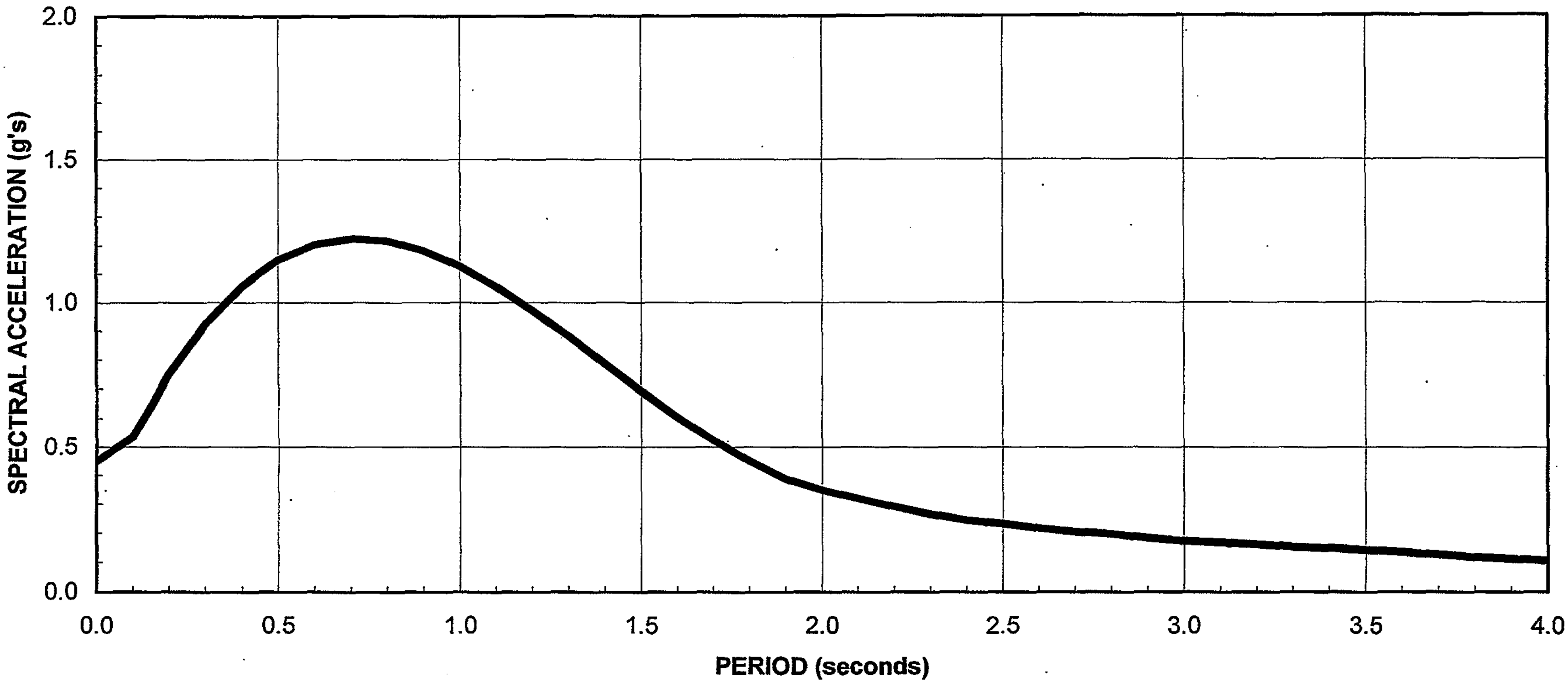


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

BLOCK 30 **MISSION BAY** San Francisco, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
Treadwell&Rollo	Date: 09/25/07 | Project No. 4086.16 | Figure: 7

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.

BLOCK 30 **MISSION BAY** San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**
Treadwell&Rollo	Date: 09/25/07 | Project No. 4086.16 | Figure: 8



Damping Ratio = 5%

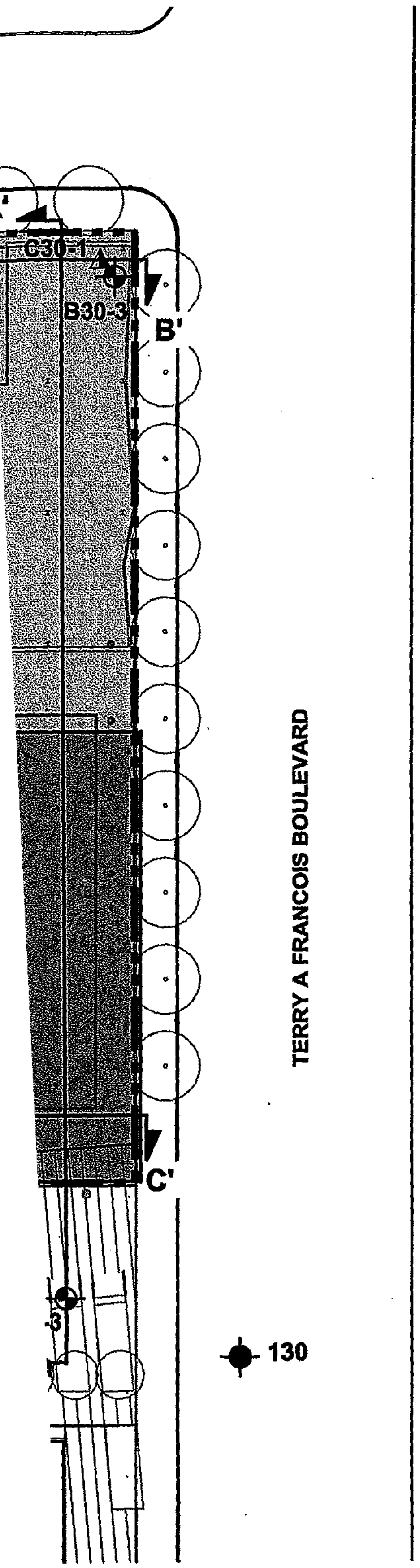
Note: DBE has a 10% probability of exceedance in 50 years.

BLOCK 30
MISSION BAY
San Francisco, California

RECOMMENDED DBE SPECTRUM

Date 07/25/07	Project No. 4086.16	Figure 9

Treadwell&Rollo



EXPLANATION

B30-1 — Approximate location of boring by Treadwell & Rollo, Inc.

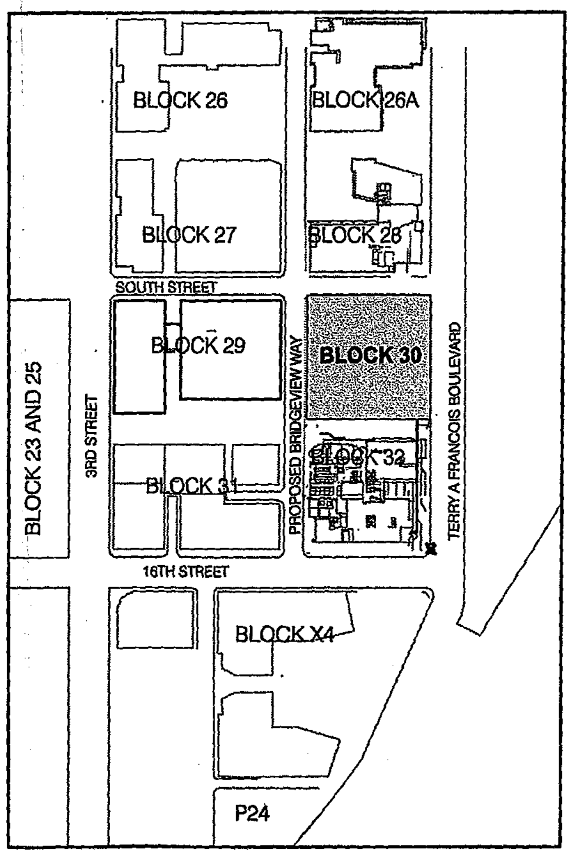
C30-1 — Approximate location of cone penetration test by Treadwell & Rollo, Inc.

360 — Approximate location of boring by Treadwell & Rollo, Inc. or others for previous investigations

A — A' — Idealized cross section location

Property line

Downdrag Zone - see Tables 4 and 5 in report



VICINITY MAP — Not to scale

0 — 50 Feet

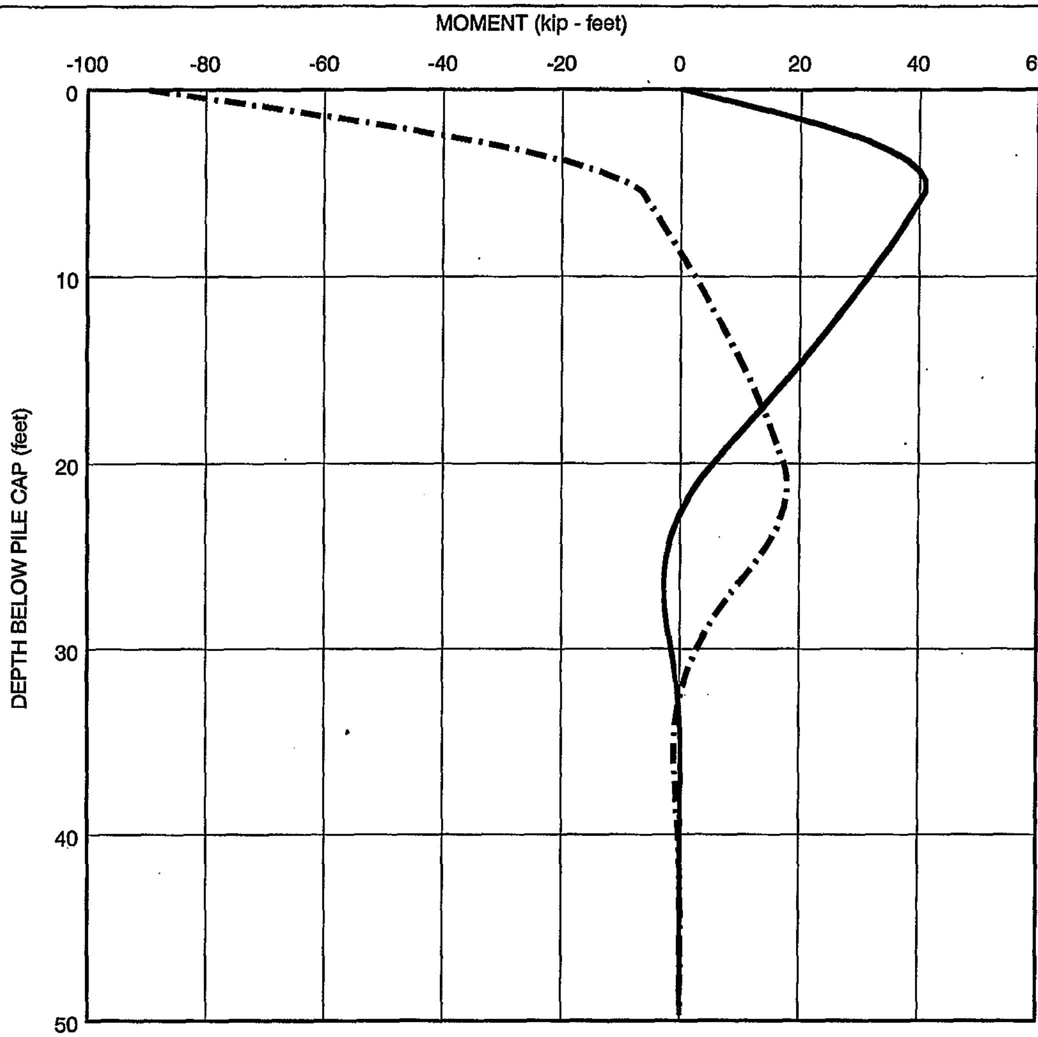
Approximate scale

BLOCK 30
MISSION BAY
San Francisco, California

DOWNDRAG ZONES

Date 09/25/07	Project No. 4086.16	Figure 10

Treadwell&Rollo

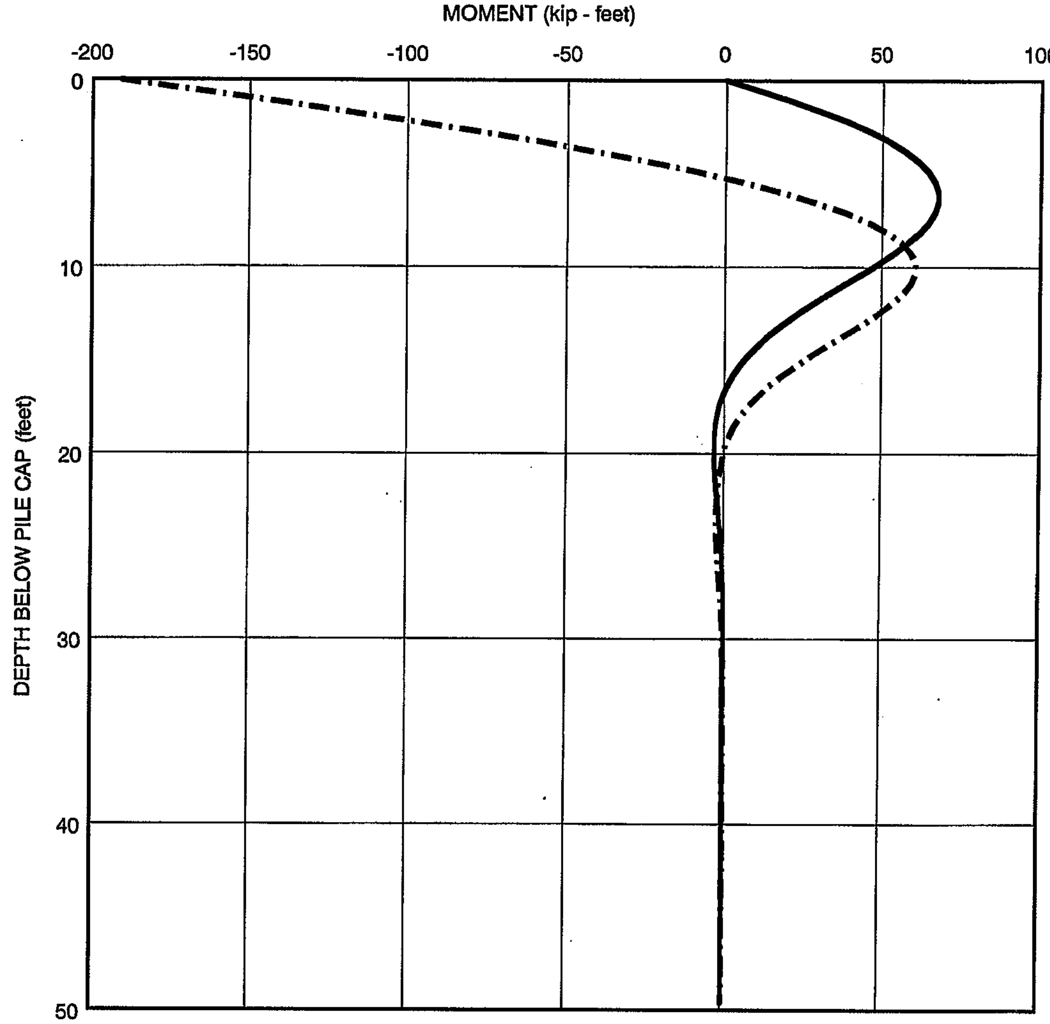


Symbol	Pile Head Connection	Pile Size (inches)	Lateral Load (kips)	Depth to First Point of Zero Deflection (feet)
———	Free	14	11	14
— · — · —	Fixed	14	21	26

Notes for Figure:

1. The profiles shown are for a single, square reinforced concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 250 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown on Figure 13a and 13b. However, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no applied moment at the pile head.
4. Assumes site has not been improved to mitigate against liquefaction.
4. Assumes pile cap depth of five feet below bottom of floor slab.
5. Passive resistance of pile caps has not been included.

BLOCK 30 MISSION BAY San Francisco, California	MOMENT PROFILE PRECAST PRESTRESSED CONCRETE PILE WITH LIQUEFACTION
Treadwell&Rollo	Date 09/26/07 · Project No. 4086.16 · Figure 11



Symbol	Pile Head Connection	Pile Size (inches)	Lateral Load (kips)	Depth to First Point of Zero Deflection (feet)
———	Free	14	17	11
▬ ▪ ▬ ▪ ▬ ▪	Fixed	14	42	14

Notes for Figure:

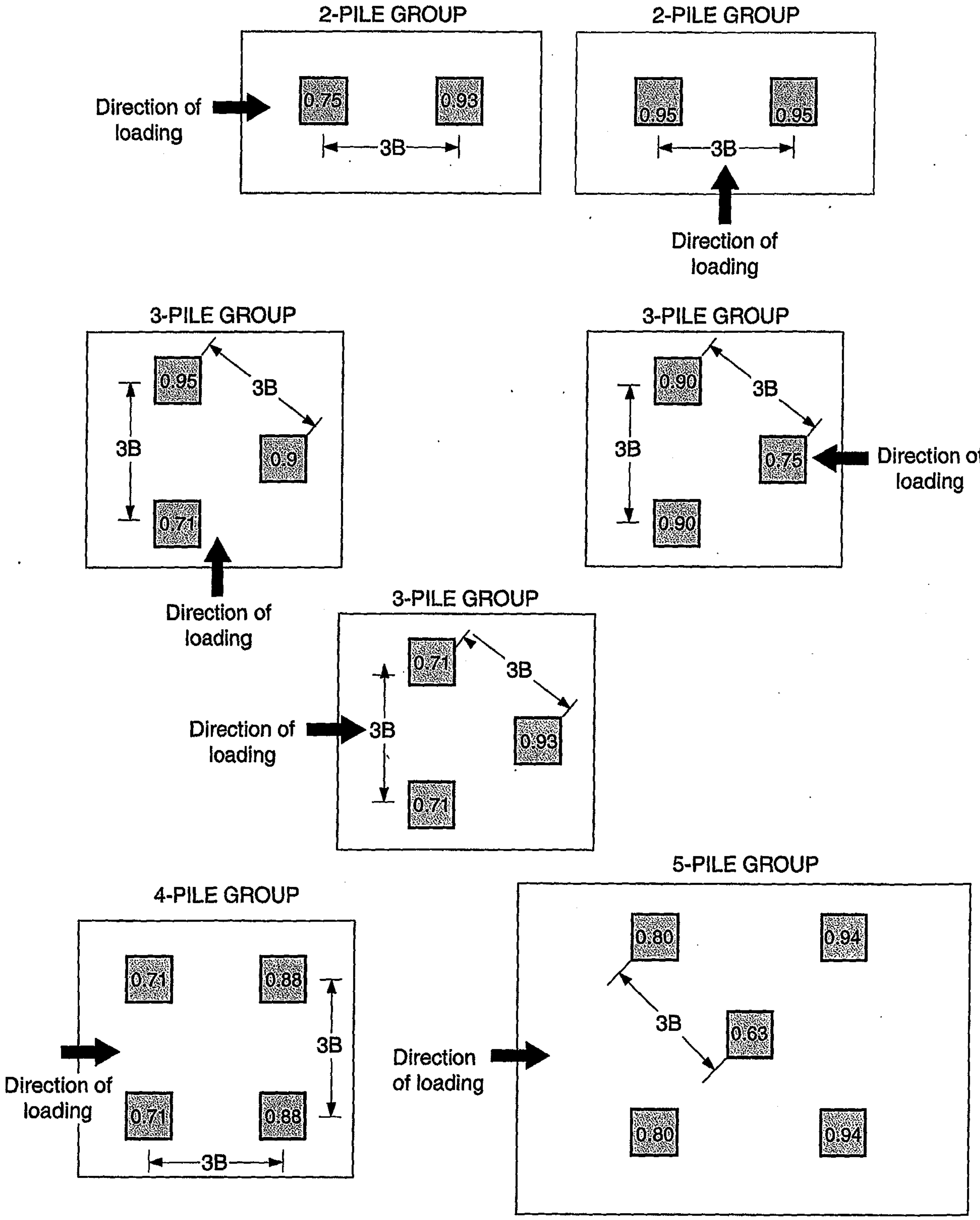
1. The profiles shown are for a single, square reinforded concrete steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 250 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown on Figure 13a and 13b. However, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no applied moment at the pile head.
4. Assumes site has been improved to mitigate against liquefaction.
4. Assumes pile cap depth of five feet below bottom of floor slab.
5. Passive resistance of pile caps has not been included.

BLOCK 30
MISSION BAY
San Francisco, California

Treadwell&Rollo

MOMENT PROFILE
PRESTRESSED PRECAST CONCRETE PILE
NO LIQUEFACTION

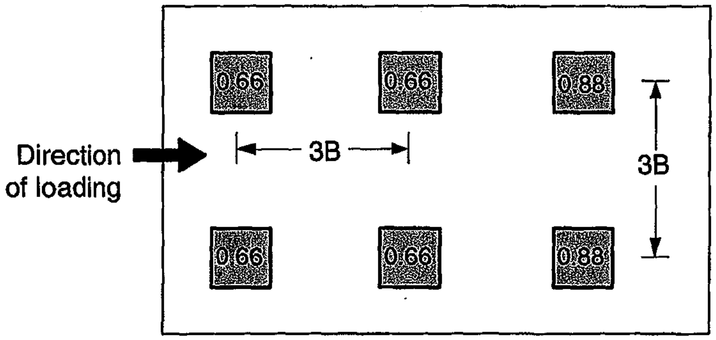
Date 09/25/07	Project No. 4086.16	Figure 12



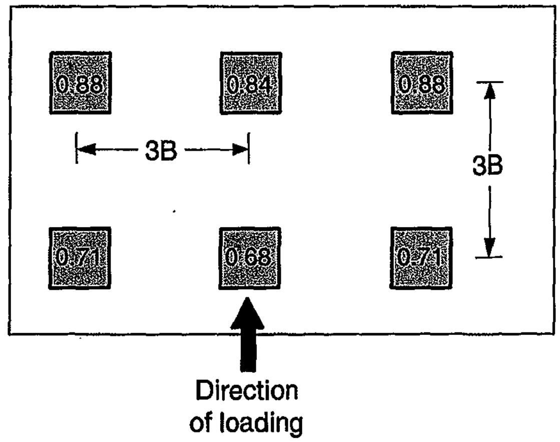
Note: 1. To account for group effects, the lateral load capacity of the pile group should be multiplied by the average of the factors shown; however, the moment profile (Figures 11 and 12) used to check individual piles in a group should be for the unfactored load.
2. The factor for caps with greater than 9 piles, is the average of the factors shown for the 9-pile group.

BLOCK 30 **MISSION BAY** San Francisco, California	**LATERAL LOAD** **GROUP REDUCTION FACTORS**
Treadwell&Rollo	Date 09/25/07 | Project No. 4086.16 | Figure 13a

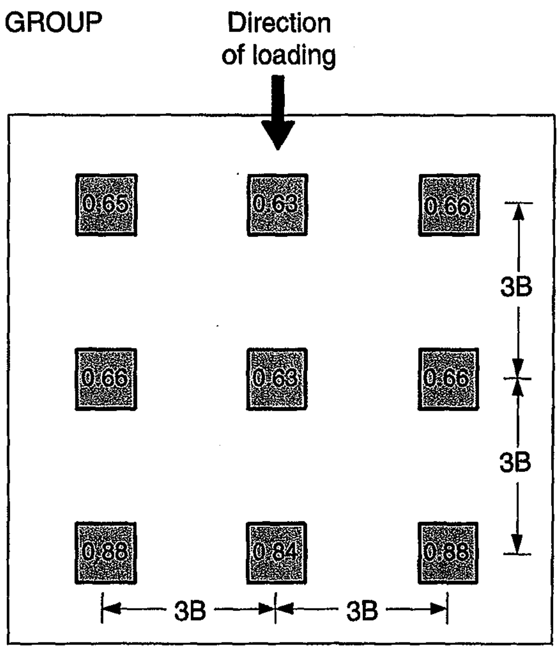
6-PILE GROUP



6-PILE GROUP



9-PILE GROUP



Note: 1. To account for group effects, the lateral load capacity of the pile group should be multiplied by the average of the factors shown; however, the moment profile (Figures 11 and 12) used to check individual piles in a group should be for the unfactored load.
2. The factor for caps with greater than 9 piles, is the average of the factors shown for the 9-pile group.

BLOCK 30 **MISSION BAY** San Francisco, California	**LATERAL LOAD** **GROUP REDUCTION FACTORS**
Treadwell&Rollo	Date 09/25/07 | Project No. 4086.16 | Figure 13b

APPENDIX A
Logs of Test Borings and CPTs

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-1 PAGE 1 OF 5

Boring location: See Site Plan, Figure 2

Date started: 5/6/07 | Date finished: 5/6/07

Drilling method: Rotary Wash

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

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

Logged by: L. Splitter

LABORATORY TEST DATA

Ground Surface Elevation: 100.6 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				2 inches concrete over 6 inches aggregate base							
1–2.7			SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, with brick fragments	FILL						
3	S&H	19	CL-ML	SANDY SILTY CLAY with GRAVEL (CL-ML) olive-gray, very stiff, moist, with brick fragments LL = 26, PI = 5	FILL						
6	SPT	17	SP	SAND (SP) olive, medium dense, moist, with glass and gravel	FILL						
8	SPT	4	SP	gray-brown, very loose, with brick, rock in shoe, blow count low because pushed into clay	FILL						
9–10			CH	CLAY (CH) gray, very soft, wet	BAY MUD						
11	S&H	1	CH		BAY MUD						
13–14	ST	0 to 75 psi	CH	gray, trace sand	BAY MUD						
26			CH	shells at 26 feet	BAY MUD						
28–29	ST	0 to 100 psi	CH	blue-gray, soft Consolidation Test, see Figure B-1	BAY MUD	TxUU	1,200	360		58.6	63

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-1a

TEST GEOTECH LOG 408616.GPJ TR.GDT 9/18/07

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-1 PAGE 2 OF 5

DEPTH (feet)	Sampler Type	Sample	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
30	ST			CH	CLAY (CH) (continued)	BAY MUD						
43–45	ST		0 to 75 psi	CH	gray, soft	BAY MUD						
53				CH	sandy at 54 feet	BAY MUD						
54–55	S&H		35	SC	CLAYEY SAND (SC) mottled olive-gray and olive, dense, wet, fine-grained sand yellow-brown at 54.75 feet							
58				CL	CLAY (CL) olive, stiff to very stiff, wet,							
59–60	S&H		8	CL								

Depth scale: 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60

Treadwell&Rollo

Project No.:	Figure:
4086.16	A-1b

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-1 PAGE 3 OF 5

DEPTH (feet)	Sampler Type	Sample	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
60–61	S&H		8	CL	CLAY (CL) (continued) with gray and yellow-brown mottling at 60.5						
62–67	S&H (64–65.5)		35		SANDY CLAY (CL) yellow-brown with gray mottling, hard, wet, trace fine gravel						
67–77	SPT (69–70.5)		20	SP-SC	SAND with CLAY (SP-SC) orange-brown, medium dense, wet						
	SPT (74–75.5)		52		mottled olive and red-brown, very dense						
77–84.5	SPT (79–80.5)		51	SP	SAND (SP) olive-brown, very dense, wet						
84.5–86.5	SPT (84–85.5)		31	CL	SANDY CLAY (CL) olive, hard, wet						
86.5–90	SPT (89–90)		86/11"	SP-SC	SAND with CLAY (SP-SC) olive-brown, very dense, wet				5.6		

COLMA (67–90)

Treadwell&Rollo

Project No.: 4086.16

Figure: A-1c

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/6/07

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-1 PAGE 4 OF 5

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91–92				SP-SC	SAND with CLAY (SP-SC) (continued)	COLMA						
93–96	SPT (94–95)		50/ 5"	SP	SAND (SP) olive-brown, very dense, wet	COLMA						
97–105	SPT (99–100)		15	CH	CLAY (CH) gray, stiff to very stiff, wet rock fragments in cuttings at 106 feet	OLD BAY CLAY						
106–110	SPT (109)		50/ 5"		SERPENTINITE intensely fractured, low hardness, weak, moderately weathered	BEDROCK						
111–120	SPT (114–115)		71		CLAYSTONE intensely fractured, low hardness, plastic, deeply weathered	BEDROCK						

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/12/07

Treadwell&Rollo

Project No.: 4086.16 Figure: A-1d

PROJECT: **BLOCK 30**
MISSION BAY EAST
San Francisco, California

Log of Boring B30-1

DEPTH (feet)	Sampler Type	Sample	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
121					CLAYSTONE (continued)	BEDROCK						
122					SERPENTINITE intensely fractured, low hardness, weak, little weathered							
125	SPT		56									
127					SHALE/SERPENTINITE crushed, soft, plastic							
129	SPT		50/ 2"									

Boring terminated at a depth of 129.2 feet.
Boring backfilled with cement grout.
Groundwater not measured at time of drilling.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-1e

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/12/07

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-2

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 5/3/07 | Date finished: 5/3/07

Drilling method: Rotary Wash

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

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

LABORATORY TEST DATA

Ground Surface Elevation: 100.4 feet [2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				2 inches asphalt concret over 12 inches aggregate base							
1–6			SP	SAND with GRAVEL (SP) olive-brown, medium dense, moist, with angular to subangular gravel, traces of brick and Serpentinite fragments	FILL						
3	S&H	17									
5				higher brick content, trace fines							
6	SPT	12									
6–9			CH	CLAY with SAND and GRAVEL (CH) dark gray, stiff, moist							
8	SPT	9		olive clay was observed from cuttings at 88 feet ▽ (5/3/07 at 7:55 am)							
9–14			SC	CLAYEY SAND with GRAVEL (SC) green-gray, loose, wet, serpentinite fragments LL = 32, PI = 13					17.6	13.0	
11	S&H	7									
12	SPT	48		gray, dense							
14–25			CH	SANDY CLAY with GRAVEL (CH) dark gray, stiff, wet, with angular to subangular gravel, and Shale fragments							
17	SPT	13									
20	SPT	14									
25–30			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
30	ST	100 psi									

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-2a

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-2

DEPTH (feet)	Sampler Type	Sample	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
31	ST		100 psi	CH	CLAY (CH) (continued)					66.9	59
40	ST		100 psi	CH	BAY MUD						
50–51	ST		100 to 250 psi	CH	sand lense at 51.5 feet						
57–58				CL	CLAY (CL) olive with orange-brown mottling, very stiff, wet						
59–60	S&H		24	CL		TxUU	2,200	2,030		25.5	100

Treadwell&Rollo

Project No.: 4086.16

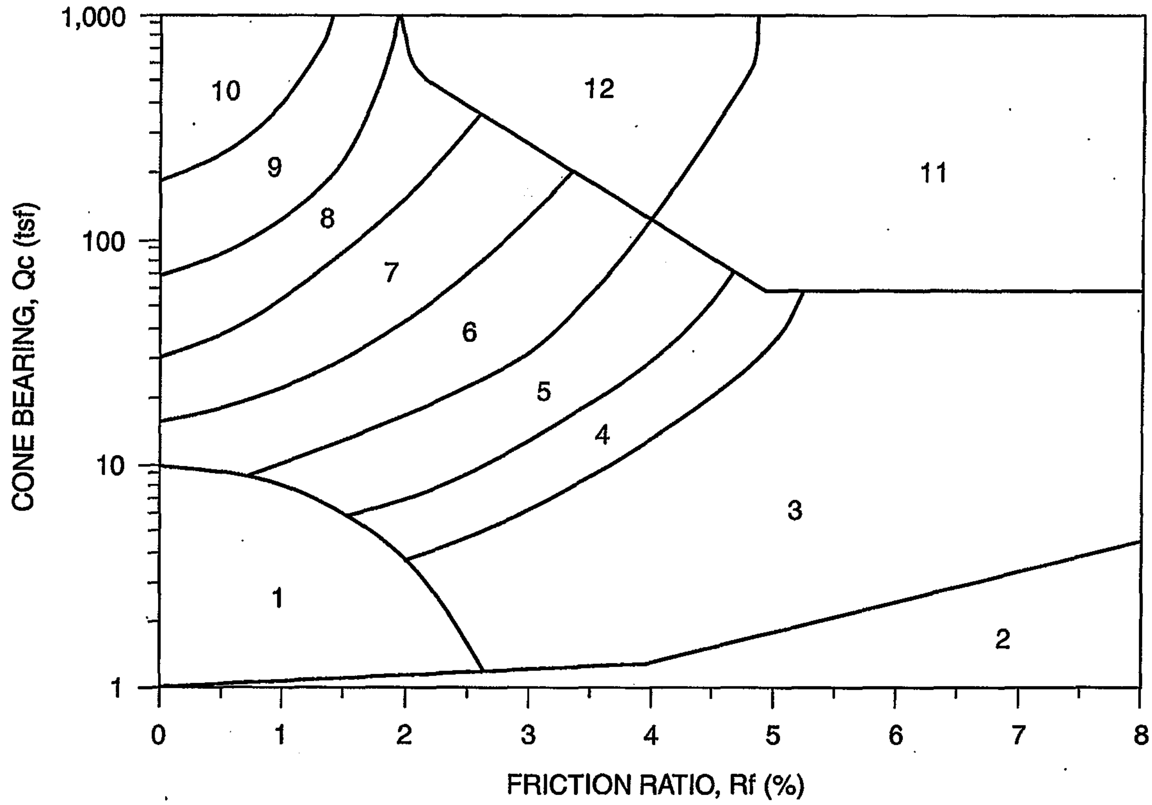
Figure: A-2b

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/12/07

DEPTH (feet)	Sampler Type	Sample	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
60	S&H		24	CL	CLAY (CL) (continued)						
62				CL	SANDY CLAY (CL) yellow-brown with olive mottling, hard, wet						
64.5	SPT		38								
67				SP-SC	SAND with CLAY (SP-SC) orange-brown, dense, wet						
69.5	SPT		34								
74.5	SPT		85/ 11"		very dense						
77				SP	SAND (SP) olive, very dense, wet						
79.5	SPT		87/ 11.5"								
84.5	SPT		69								
89	SPT		50/ 3"		SERPENTINITE						

COLMA

BEDROCK



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).

BLOCK 30
MSSION BAY
San Francisco, California

SARIMIENTO

Treadwell&Rollo

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 08/03/07	Project No. 4086.16	Figure A-9

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-2 PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample	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
91					SERPENTINITE intensely fractured, weak, moderately weathered, low hardness (BEDROCK)						
92											
93											
94	SPT		50/ 1"								
95											
96											
97											
98											
99											
100											
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 94.1 feet.
Boring backfilled with cement grout.
Groundwater encountered at 9 feet at 7:55 am on 5/3/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

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Project No.: 4086.16

Figure: A-2d

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/12/07

PROJECT: **BLOCK 30 MISSION BAY EAST** San Francisco, California

Log of Boring B30-3

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 5/2/07 | Date finished: 5/2/07

Drilling method: Rotary Wash

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

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

Ground Surface Elevation: +100.3 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				2 inches asphalt concrete over 12 inches aggregate base							
1–6.5			SC	CLAYEY SAND with GRAVEL (SC) olive-brown, medium dense, moist, with angular to subangular gravel	FILL						
3	S&H	26									
5.5	SPT	17		olive-gray, with serpentinite fragments							
6.5–9			CL	SANDY CLAY with GRAVEL (CL) olive-gray, stiff, moist							
8	SPT	9									
9–14.5			SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray, medium dense, wet ∇ (5/2/07 at 8:15 am)							
11	S&H	18							6.0	11.0	
12	SPT	14									
14.5–18.5			GC	CLAYEY GRAVEL with SAND (GC) olive-gray, medium dense, wet							
17	SPT	10							13.6	22.3	
18.5–20			GP	GRAVEL (GP) dark gray, medium dense, wet							
19.5	SPT	19									
20–30			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
25	ST	75 psi		Consolidation Test, see Figure B-2						72.0	57

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-3a

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-3 PAGE 2 OF 4

DEPTH (feet)	Sampler Type	Sample	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
30				CH	CLAY (CH) (continued)	BAY MUD						
34–36	ST		75 to 100 psi	CH		BAY MUD						
44–46	ST		75 to 100 psi	CH	Consolidation Test, see Figure B-3	BAY MUD					63.4	62
54–56	ST		150 to 250 psi	CH / CL	CLAY (CL) yellow-brown with olive mottling, hard, wet							
59–60	SPT		37	CL								

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-3b

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-3

DEPTH (feet)	Sampler Type	Sample	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
60	SPT		37	CL	CLAY (CL) (continued)						
62				SC	CLAYEY SAND (SC) orange-brown, medium dense, wet						
65	S&H		18								
70	SPT		46		dense, lower fines content						
72				SP-SC	SAND with CLAY (SP-SC) orange-brown, very dense, wet						
75	SPT		69						7.7	25.0	
78				SC	CLAYEY SAND (SC) olive with orange-brown mottling, dense, wet						
80	SPT		34								
88				CL	SANDY CLAY (CL) olive and yellow-brown with dark brown mottling, hard, wet						
90	SPT		33								

COLMA (62 to 88 feet)

Treadwell&Rollo

Project No.: 4086.16

Figure: A-3c

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/12/07

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	**Log of Boring B30-3** PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample	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
	SPT		33	CL	SANDY CLAY (CL) (continued)						
91											
92											
93					SERPENTINITE intensely fractured, weak, moderately weathered, low hardness (BEDROCK)						
94	SPT		50/ 4"								
95											
96											
97											
98											
99	SPT		50/ 0"								
100											
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 99 feet.
Boring backfilled with cement grout.
Groundwater encountered at 9.8 feet at 8:15 am on 5/2/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-3d

TEST GEOTECH LOG 408616.GPJ TR.GDT 6/12/07

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	**Log of Boring B30-4** PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 5/5/07 | Date finished: 5/5/07

Drilling method: Rotary Wash

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

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

LABORATORY TEST DATA

Ground Surface Elevation: 100.4 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				3 inches asphalt concret over 12 inches aggregate base							
1–3			SC	CLAYEY SAND (SC) olive-brown, medium dense, moist	FILL						
3	S&H	15									
3–5.5			SP	SAND (SP) yellow-brown, medium dense, moist, fine-grained sand							
6	SPT	13									
5.5–10			CH	CLAY with GRAVEL (CH) gray, stiff, moist							
8	SPT	6		∇ (5/5/07 at 8:40 am)							
				green with dark green mottling, medium stiff, wet, with angular Serpentinite gravel							
10–11.5	S&H	4	GC	CLAYEY GRAVEL (GC) green-gray, loose, wet, with Serpentinite							
12	SPT	12									
11.5–15			SC	CLAYEY SAND with GRAVEL (SC) olive, medium dense, wet							
15–25			SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray, medium dense, wet							
17	SPT	13									
20	SPT	4		very loose to loose					6.7	19.9	
25–30			CH	CLAY (CH) gray, medium stiff, wet, with shell fragments	BAY MUD						
29.5	ST	75 psi				PP		750			

Treadwell&Rollo

Project No.: 4086.16

Figure: A-4a

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-4

DEPTH (feet)	Sampler Type	Sample	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
31	ST		75 psi	CH	CLAY (CH) (continued)						
32–39					BAY MUD						
40	ST		75 to 100 psi	CH	Consolidation Test, see Figure B-4	TxUU	1,500	725		74.4	56
41–49											
50	ST		100 to 250 psi	SP	SAND (SP) gray, wet						
52–53				CL	CLAY with SAND (CL) olive with orange-brown mottling, very stiff, wet						
55	S&H		18	CL		TxUU	1,700	3,450		22.3	105
56–58											
59	SPT		26		olive with red-brown mottling, very stiff, wet						
60											

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Project No.: 4086.16 | Figure: A-4b

DEPTH (feet)	Sampler Type	Sample	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
60	SPT		26	CL	CLAY with SAND (CL) (continued)							
62				SC	CLAYEY SAND (SC) orange-brown, medium dense, wet, fine-grained sand	COLMA						
64.5	SPT		18									
69.5	SPT		58		very dense, lower fines content					12.4	23.3	
74.5	SPT		56		olive, higher fines content							
77				SP-SC	SAND with CLAY (SP-SC) orange-brown, very dense, wet							
79.5	SPT		61									
81				CL	SANDY CLAY (CL) olive, hard, wet							
84.5	SPT		36									
85					SERPENTINITE intensely fractured, moderately hard, weak, moderately weathered	BEDROCK						
87					SHALE intensely fractured, moderately hard, weak, moderately weathered							
89.5	SPT		50/ 4.5"									
90												

LABORATORY TEST DATA

PROJECT: **BLOCK 30**
MISSION BAY EAST
San Francisco, California

Log of Boring B30-4

DEPTH (feet)	Sampler Type	Sample	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
					SHALE (continued)	BEDROCK						
91												
92					SERPENTINITE intensely fractured, moderately hard, weak, moderately weathered							
93												
94	SPT		50/ 5.5"									
95												
96												
97												
98												
99												
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 95 feet.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet at 8:40 am.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-4d

PROJECT: **BLOCK 30**
MISSION BAY EAST
San Francisco, California

Log of Boring B30-5

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 5/4/07 | Date finished: 5/4/07

Drilling method: Rotary Wash

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

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

MATERIAL DESCRIPTION

Ground Surface Elevation: 100.3 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1.6				3 inches asphalt concret over 12 inches aggregate base and 4 inches concrete						
1.6			SC	CLAYEY SAND with GRAVEL (SC) olive-gray, medium dense, moist						
3	S&H	16								
5–6	SPT	10		loose to medium dense, with brick fragments						
7.8			CH	CLAY with SAND (CH) gray, medium stiff to stiff, wet, with brick fragments and Serpentinite (5/4/07 at 8:45 am)						
8	SPT	8								
10				stiff, no brick						
11	S&H	11								
11			CL-ML	SANDY SILTY CLAY (CL-ML) gray, stiff, wet LL = 23, PI = 7						
12	SPT	11								
14.5			SP-SC	SAND with CLAY and GRAVEL (SP-SC) green-gray, medium dense, wet						
17	SPT	11						10.8	16.1	
19–20	SPT	6		loose						
24–25	SPT	6		green with orange-brown mottling				11.9	24.1	
29–30	SPT	8								

FILL (from 1.6 to 30 feet)

Treadwell&Rollo

Project No.: 4086.16 | Figure: A-5a

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	**Log of Boring B30-5** PAGE 2 OF 3

DEPTH (feet)	Sampler Type	Sample	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
30	SPT		8		SAND with CLAY and GRAVEL (SP-SC) (continued)	FILL						
31–33				CH	CLAY (CH) gray, soft, wet	BAY MUD						
34–36	ST		75 to 300 psi		sand lens at 35.5 to 37 feet							
39–40	S&H		2		with shell fragments							
44–46	ST		75 to 150 psi									
52–53				CL	CLAY (CL) yellow-brown with orange-brown mottling, hard, wet, with trace fine-grained sand							
54–55	SPT		35									
58				SC	CLAYEY SAND (SC) orange-brown, dense, wet	COLMA						
59–60	SPT		36							29.2	18.9	

Treadwell&Rollo

Project No.: 4086.16

Figure: A-5b

PROJECT:	BLOCK 30 MISSION BAY EAST San Francisco, California	Log of Boring B30-5 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	Sample	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
	SPT		36	SC	CLAYEY SAND (SC) (continued)	COLMA						
61												
62												
63				CL	CLAY (CL) olive, very stiff, wet							
64												
65	SPT		22									
66												
67												
68												
69	SPT		50/ 4.5"		SANDSTONE intensely fractured, friable, low hardness	BEDROCK						
70												
71												
72												
73					SERPENTINITE intensely fractured, friable, low hardness							
74												
75	SPT		85/ 10"									
76												
77												
78												
79	SPT		50/ 5.5"									
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 79.5 feet.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet at 8:55 am on 5/4/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

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Project No.: 4086.16 | Figure: A-5c

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with split-barrel sampler other than Standard Penetration Test sampler. Darkened area indicates soil recovered
- Classification sample taken with Standard Penetration Test sampler
- Undisturbed sample taken with thin-walled tube
- Disturbed sample
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample
- Sample taken with Direct Push sampler

SAMPLER TYPE

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

BLOCK 30
MISSION BAY
San Francisco, California

Treadwell&Rollo

CLASSIFICATION CHART

Date 05/16/07 | Project No. 4086.16 | Figure A-6

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

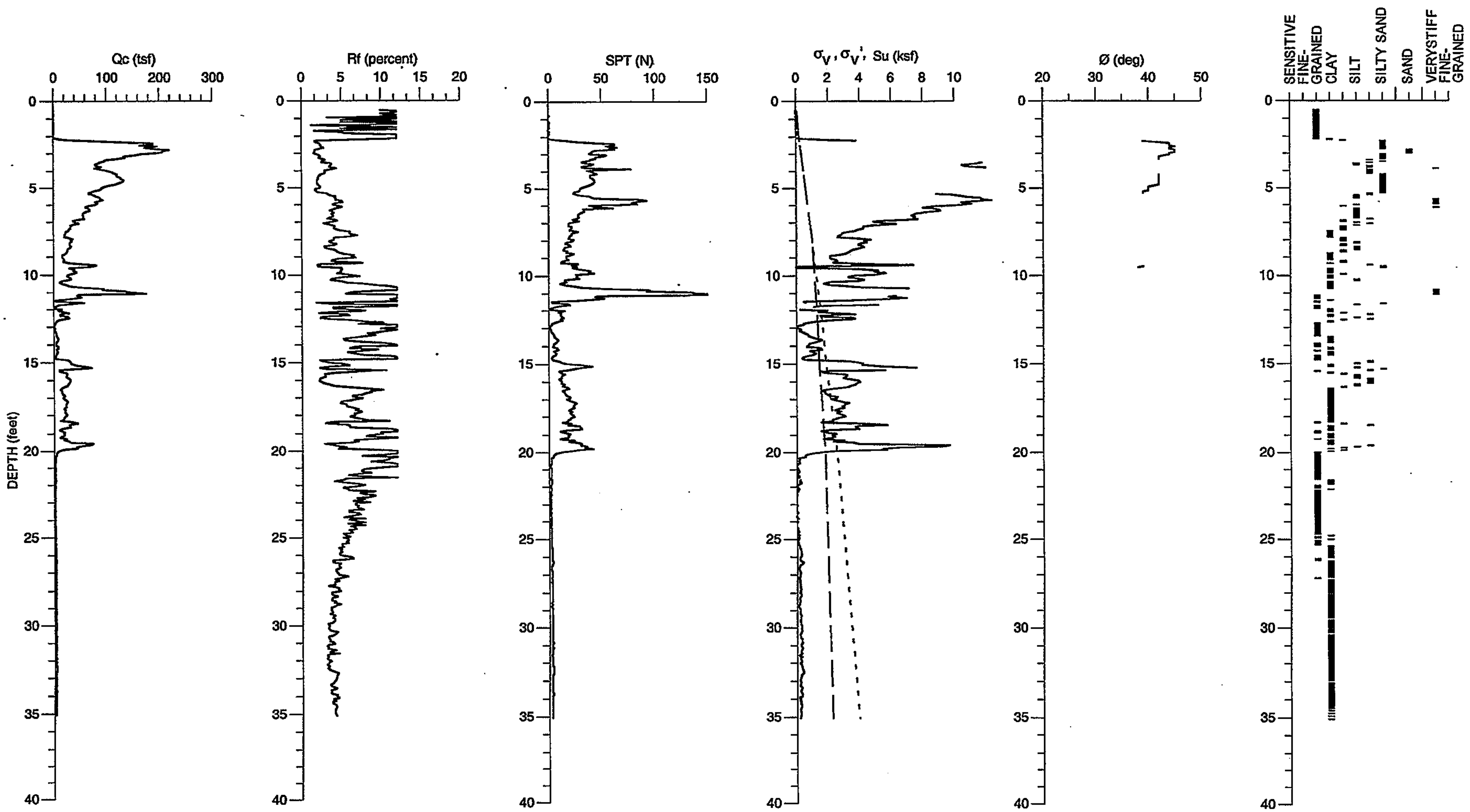
ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated



Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, s_u

Terminated at 35.0 feet.
Groundwater assumed to be at a depth of 8.0 feet bgs.
Date performed: 05/04/07.
Elevation: 100.3 feet, Datum: San Francisco City Datum plus 100 feet.

BLOCK 30
MISSION BAY
San Francisco, California

CONE PENETRATION TEST RESULTS
C30-1

Date 09/20/07	Project No. 4086.16	Figure A-8

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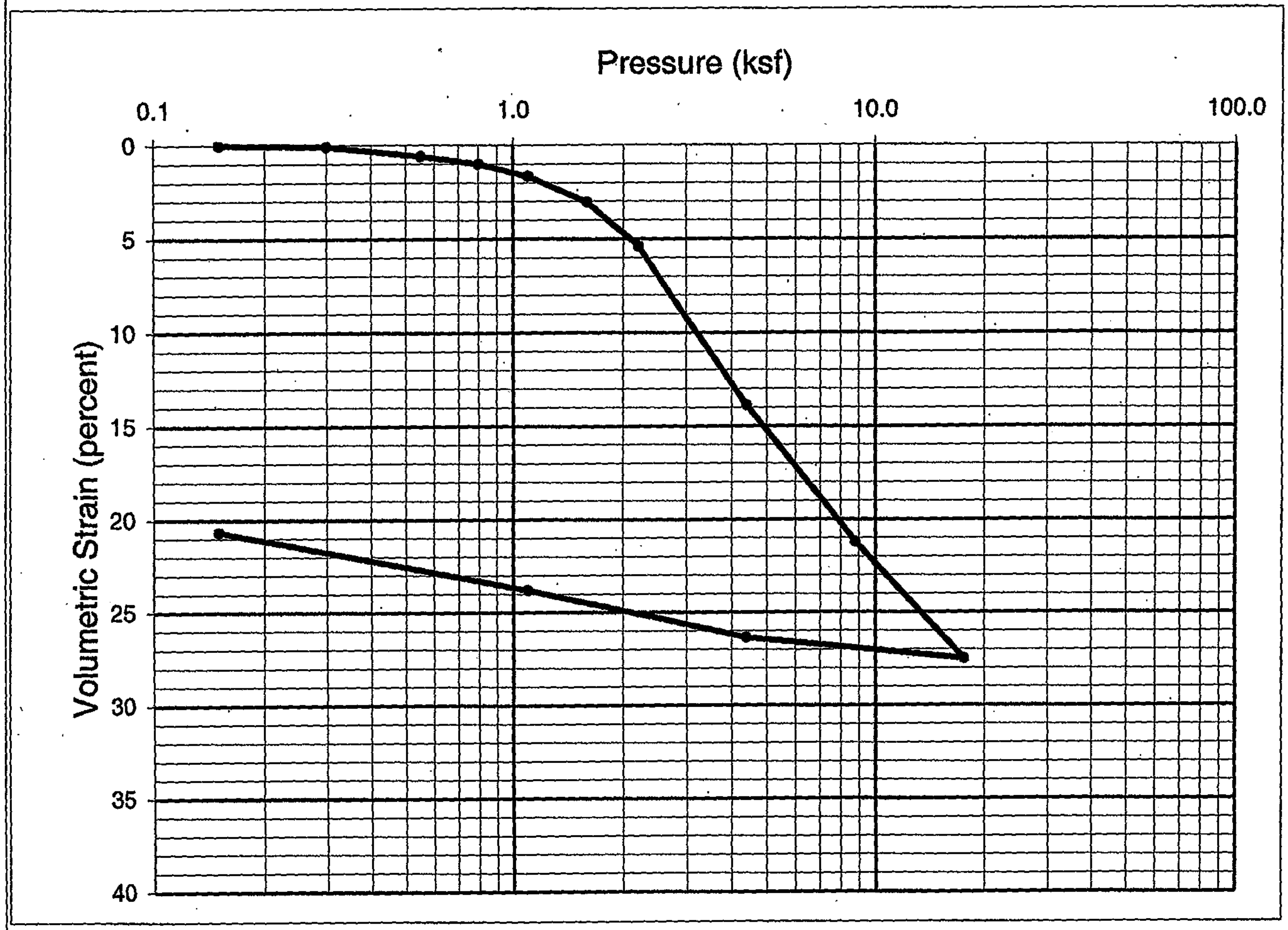
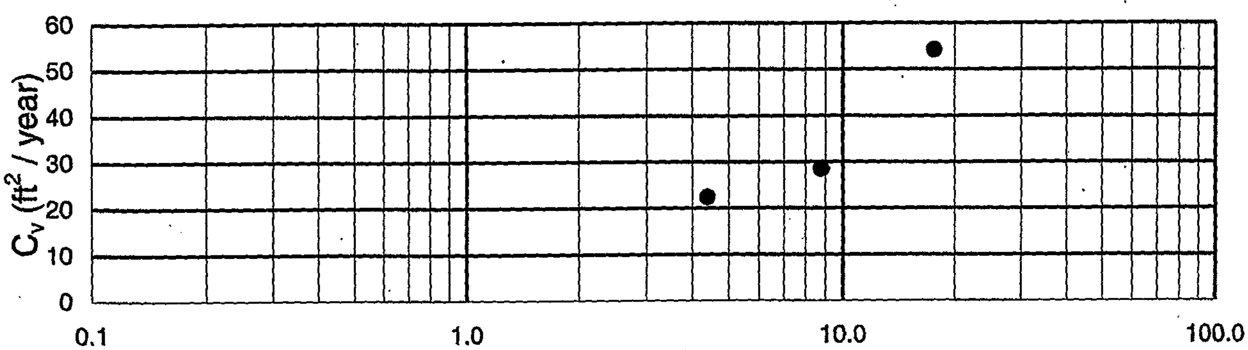
APPENDIX B
Laboratory Test Results





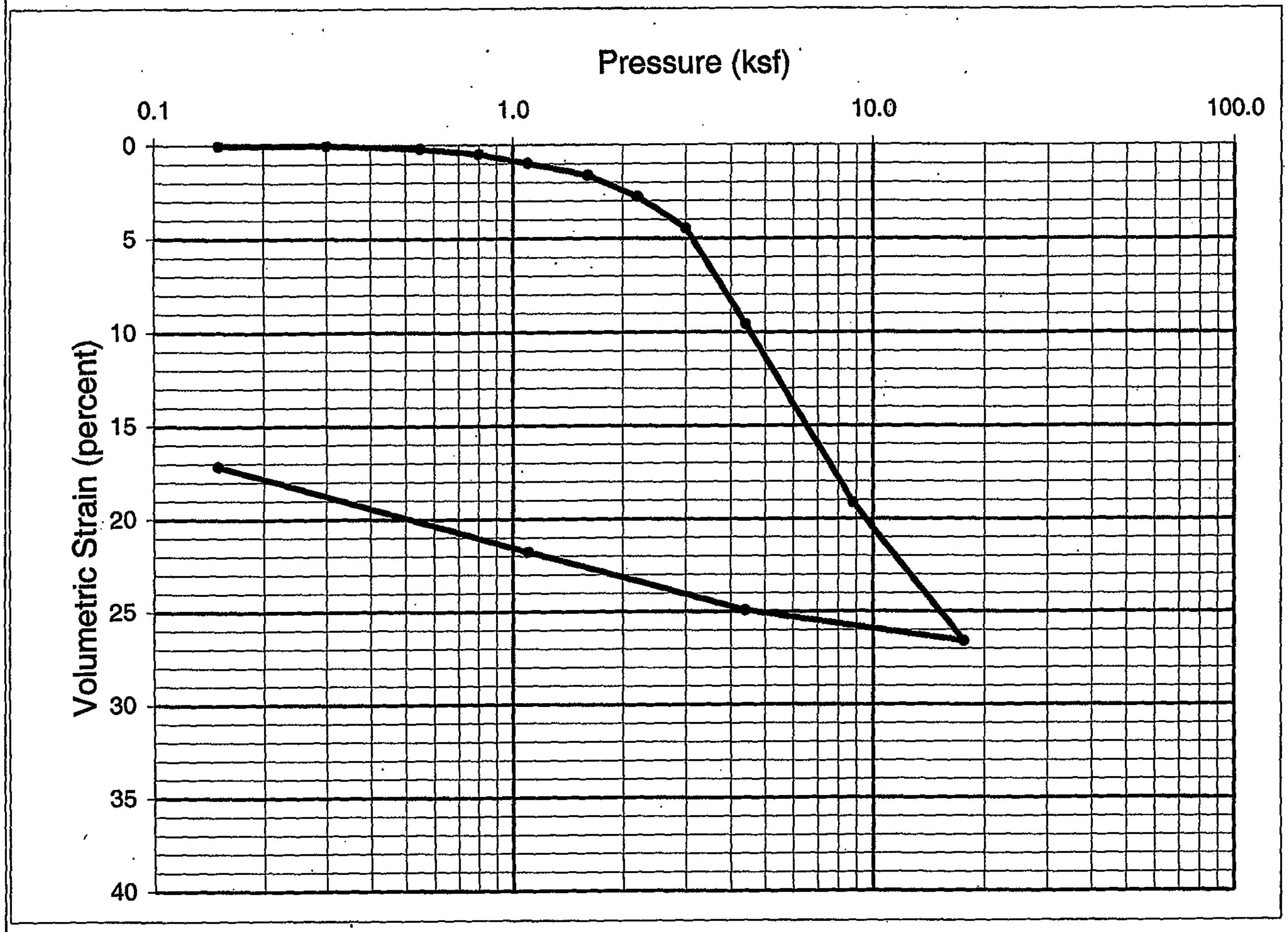
Sampler Type: Shelby Tube				Condition	Before Test			After Test	
Diameter (in)	2.41	Height (in)	1.00	Water Content	w_o	58.6 %	w_f	42.3	%
Overburden Pressure, p_o	1,700	psf		Void Ratio	e_o	1.66	e_f	1.14	
Preconsol. Pressure, p_c	1,900	psf		Saturation	S_o	95 %	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.26			Dry Density	γ_d	63 pcf	γ_d	79	pcf
Compression Ratio, $C_{\varepsilon r}$	0.04						G_s	2.70	(assumed)
Classification CLAY (CH), gray					Source	B30-1 @ 28'			

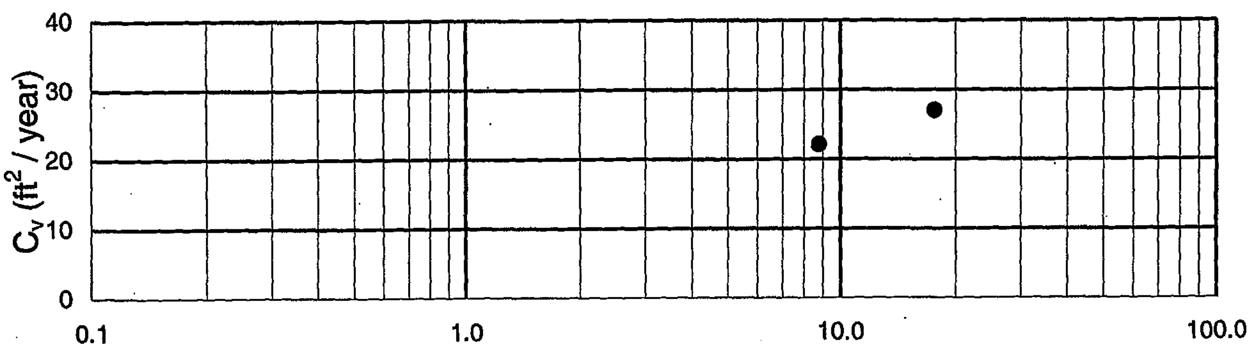
BLOCK 30 - MISSION BAY
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 09/26/07 | Project No. 4086.16 | Figure B-1





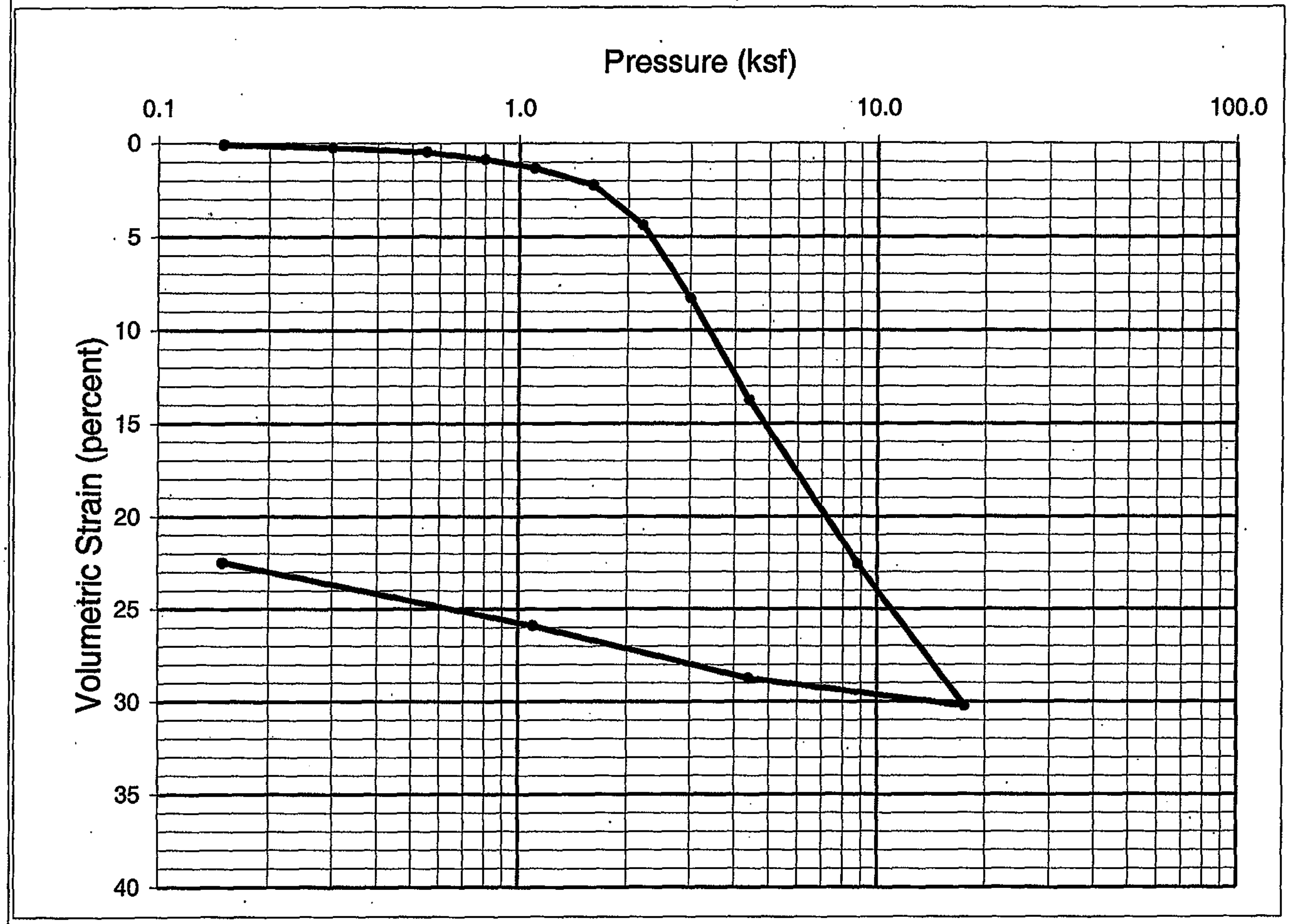
Sampler Type: Shelby Tube				Condition	Before Test			After Test	
Diameter (in) 2.41	Height (in) 1.01			Water Content	w_o	63.4 %		w_f	47.7 %
Overburden Pressure, p_o	1,800	psf		Void Ratio	e_o	1.73		e_f	1.29
Preconsol. Pressure, p_c	2,100	psf		Saturation	S_o	99 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.31			Dry Density	γ_d	62 pcf		γ_d	74 pcf
Compression Ratio, $C_{\varepsilon r}$	0.05							G_s 2.70	(assumed)
Classification CLAY (CH), gray				Source	B30-3 @ 24'				

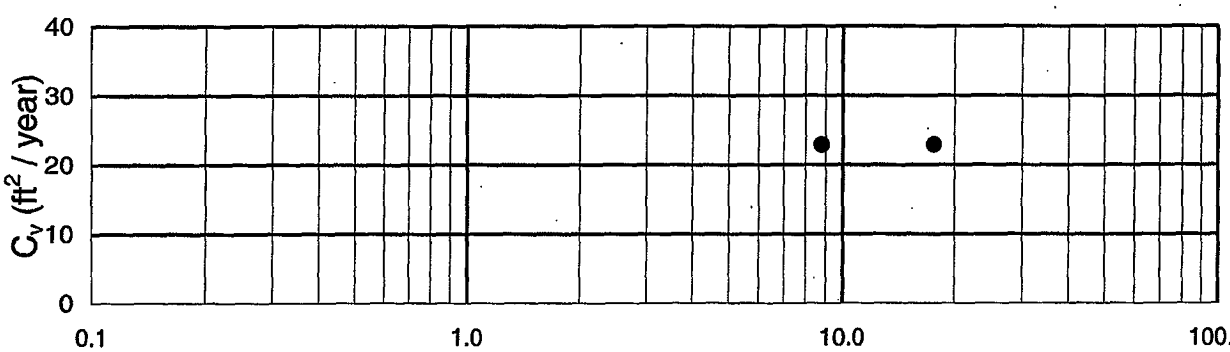
BLOCK 30 - MISSION BAY
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 09/26/07 | Project No. 4086.16 | Figure B-2





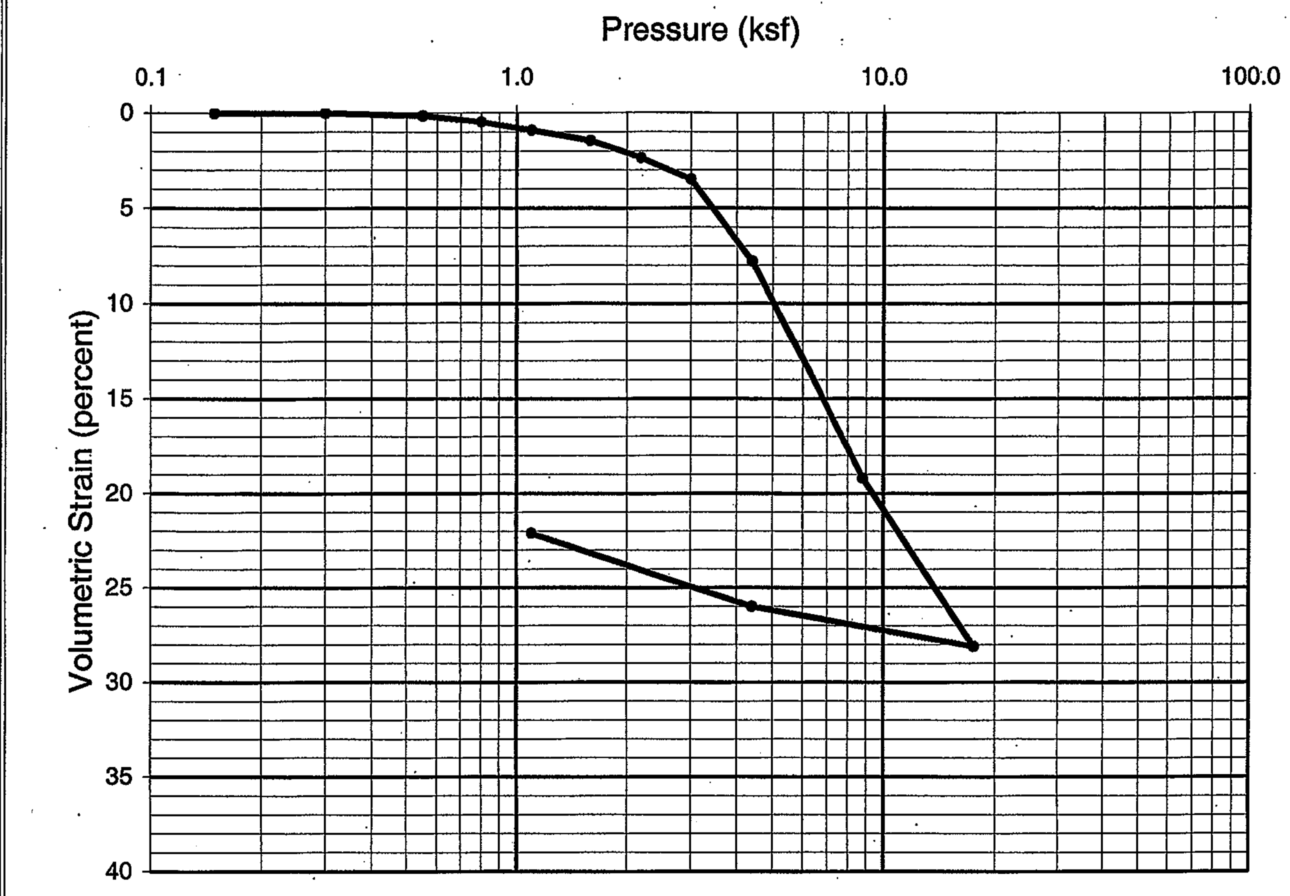
Sampler Type: Shelby Tube				Condition		Before Test			After Test	
Diameter (in)	2.41	Height (in)	1.01	Water Content	w_o	72.0	%	w_f	48.3	%
Overburden Pressure, p_o	2,550	psf		Void Ratio	e_o	1.96		e_f	1.30	
Preconsol. Pressure, p_c	2,600	psf		Saturation	S_o	99	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.29			Dry Density	γ_d	57	pcf	γ_d	73	pcf
Compression Ratio, $C_{\varepsilon r}$	0.05							G_s	2.70	(assumed)
Classification	CLAY (CH), gray			Source		B30-3 @ 44'				

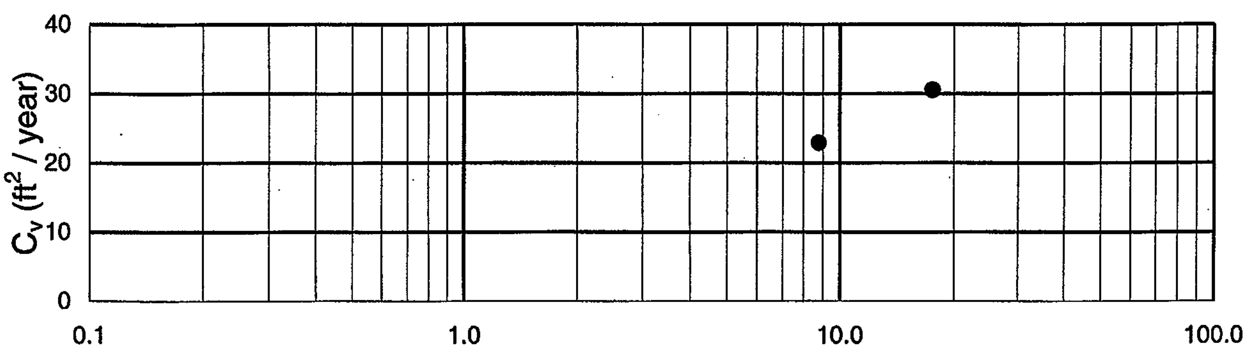
BLOCK 30 - MISSION BAY
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date	09/26/07	Project No.	4086.16	Figure	B-3





Sampler Type: Shelby Tube				Condition		Before Test		After Test	
Diameter (in) 2.41	Height (in) 1.01			Water Content	w_o	74.4 %	w_f	56.3 %	
Overburden Pressure, p_o	2,450	psf		Void Ratio	e_o	2.02	e_f	1.52	
Preconsol. Pressure, p_c	3,300	psf		Saturation	S_o	100 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.35			Dry Density	γ_d	56 pcf	γ_d	67 pcf	
Compression Ratio, $C_{\varepsilon r}$	0.06						G_s	2.70	(assumed)
Classification CLAY (CH), gray				Source		B30-4 @ 39'			

BLOCK 30 - MISSION BAY
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 09/26/07 | Project No. 4086.16 | Figure B-4

SIEVE ANALYSIS

Sample Information

Sample Identification: B30-3 at 16.5 feet
Soil Description: Clayey Gravel with Sand (GC), dark gray/green/brown
Date of Test: 5/27/2007
Test Performed by: EG

Fines Content Analysis (Wash Sieve)

Weight of Sieve (gm)	108.0
Dry Wt. Soil + Sieve (gm) (before washing)	475.1
Dry Wt. Soil + Sieve (gm) (after washing)	425.2
Dry Wt. Soil (gm)	317.2
% Passing No. 200 Sieve	13.6

Sieve Analysis Test Results

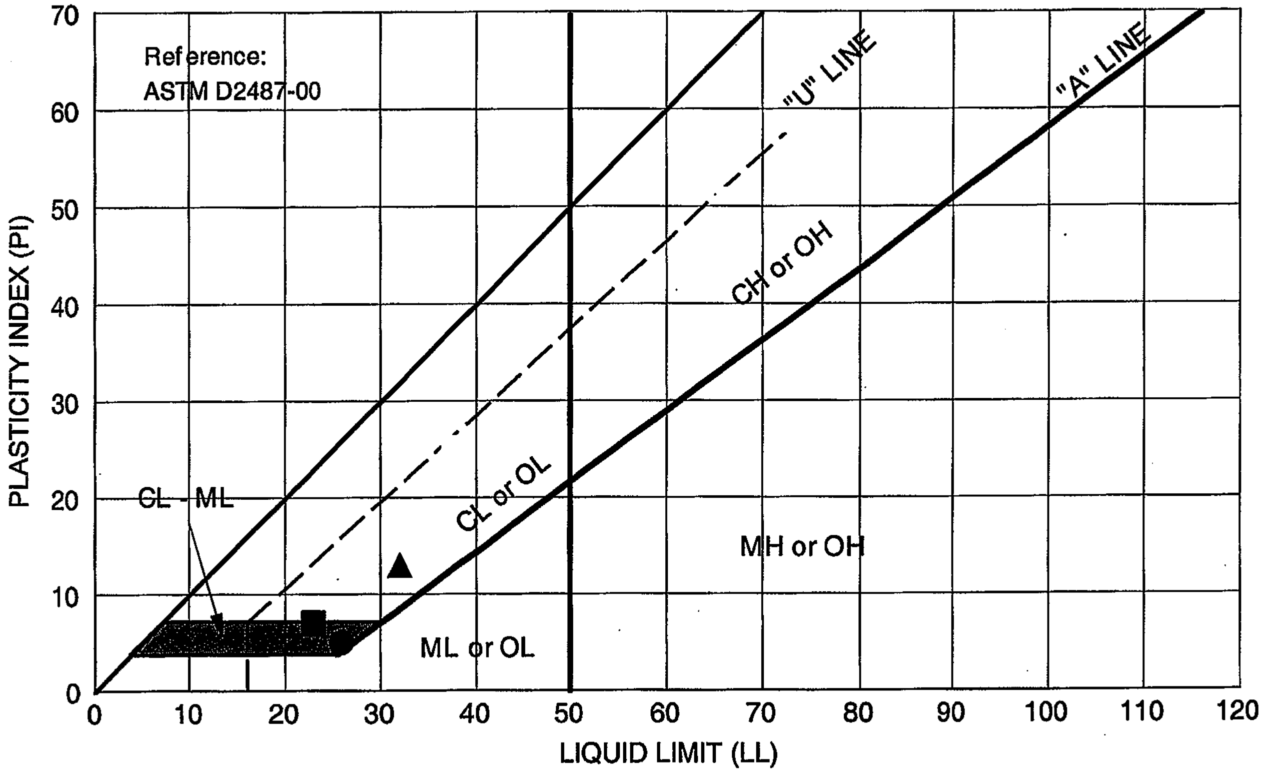
Sieve Opening (mm)	Sieve No.	Weight of Sieve (gm)	Weight of Soil + Sieve (gm)	Weight of Soil Retained (gm)	Percent Retained	Cumulative Percent Retained	Percent Passing
38.1	1-1/2	0.0	0.0	0.0	0.0%	0.0%	100.0%
19.05	3/4	926.1	999.9	73.8	20.1%	20.1%	79.9%
9.525	3/8	899.8	966.3	66.5	18.1%	38.3%	61.7%
4.76	4	873.3	926.0	52.7	14.4%	52.6%	47.4%
2.36	8	1043.2	1076.0	32.8	8.9%	61.6%	38.4%
1.18	16	961.2	987.0	25.8	7.0%	68.6%	31.4%
0.6	30	945.2	965.9	20.7	5.6%	74.2%	25.8%
0.3	50	927.7	945.9	18.2	5.0%	79.2%	20.8%
0.149	100	713.5	729.0	15.5	4.2%	83.4%	16.6%
0.074	200	719.2	729.8	10.6	2.9%	86.3%	13.7%
Fines	Pan	376.8	377.1	0.3	13.7%	100.0%	0.0%

Total Weight of Sample on Sieves (gm) 316.9

Total Weight of Sample (including washed soil) 366.8

Client: TREADWELL & ROLLO
Project Name: Block 30
Project Number: 4086.16

GEO ENGINEERING SERVICES
11 Driftwood Court, Pacifica California 94044
tel 650.359.4260 fax 650.359.2911



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B30-1 at 3 feet	SANDY SILT CLAY with GRAVEL (CL-ML), olive-gray	--	26	5	--
▲	B30-2 at 10 feet	CLAYEY SAND with GRAVEL (SC), green-gray	--	32	13	--
■	B-30-5 at 11.5 feet	SANDY SILTY CLAY (CL-ML), gray	--	23	7	--

BLOCK 30
MISSION BAY
San Francisco, California

Treadwell&Rollo

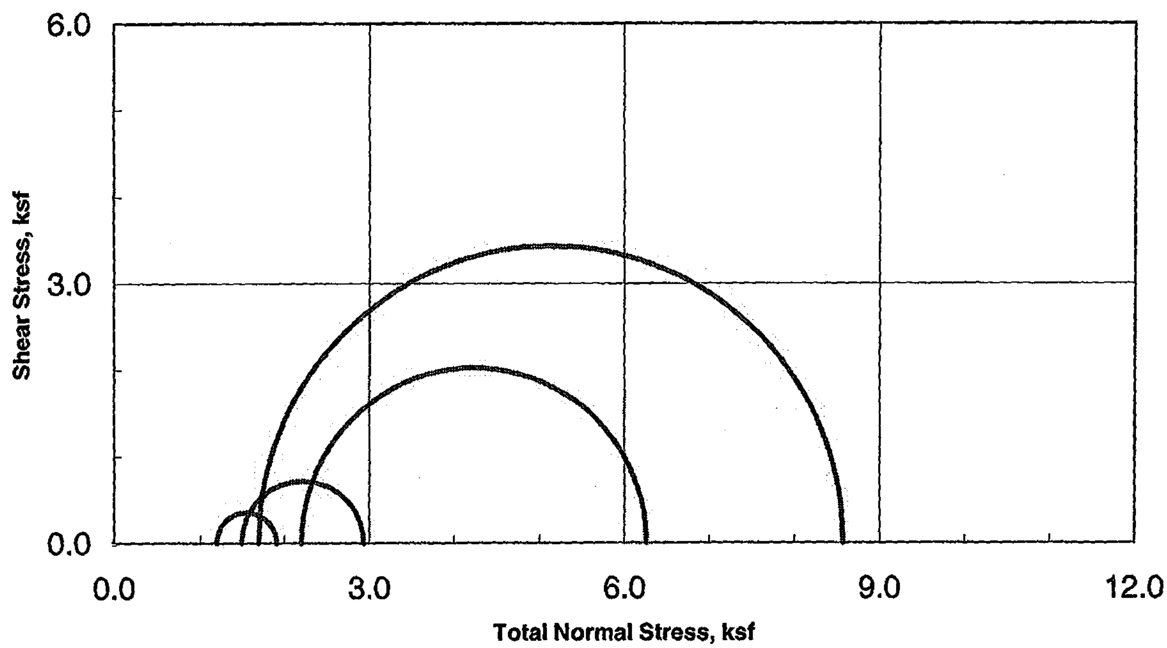
PLASTICITY CHART

Date 09/25/07 | Project No. 4086.16 | Figure B-6

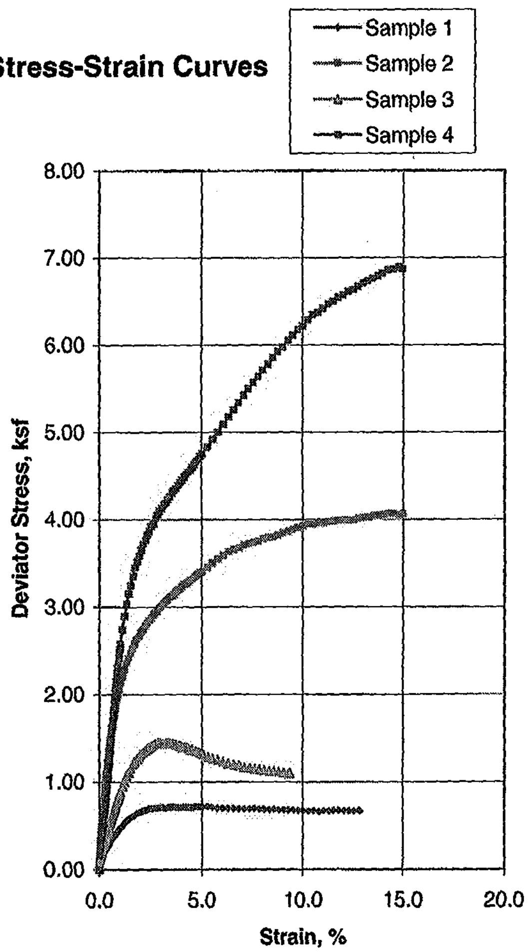
Treadwell&Rollo

APPENDIX C
Soil Corrosivity Analysis and Recommendations

Unconsolidated-Undrained Triaxial Test
ASTM D-2850



Stress-Strain Curves



Sample Data	1	2	3	4
Moisture %	58.1	25.5	63.8	22.3
Dry Den,pcf	65.5	99.7	61.8	105.1
Void Ratio	1.572	0.691	1.725	0.604
Saturation %	99.7	99.6	99.8	99.7
Height in	6.02	5.00	6.01	5.01
Diameter in	2.86	2.43	2.86	2.40
Cell psi	8.3	15.3	10.4	11.8
Strain %	3.40	14.10	2.80	14.70
Deviator, ksf	0.720	4.061	1.450	6.896
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.060	0.050	0.060	0.050
Job No.:	010-1041a			
Client:	Treadwell & Rollo			
Project:	Block 30/Block 32 - 4086.16/.17			
Boring:	B30-1	B30-2	B30-4	B30-4
Sample:				
Depth ft:	28	59.5	39	54.0

Visual Soil Description

Sample #	
1	Gray CLAY
2	Olive Brown CLAY w/ Sand
3	Gray CLAY
4	Olive Brown CLAY w/ Sand

Remarks:

REVISED SOIL CORROSIVITY EVALUATION & RECOMMENDATIONS FOR CORROSION CONTROL

MISSION BAY BLOCK 30/32
SAN FRANCISCO, CA

for

Treadwell & Rollo, Inc.
Oakland, California

July 6, 2007

Prepared by:

JDH Corrosion Consultants
Incorporated

424 N. Wiget Lane
Walnut Creek, CA 94598
Tel. No. 925.927.6630
Fax No. 925.927.6634

JDH Corrosion Consultants
Incorporated

July 6, 2007

Treadwell & Rollo, Inc
501 14th Street, Third Floor
Oakland, California 94612

Attention: **Lisa Splitter**

Subject: **Revised Soil Corrosivity Evaluation & Recommendations for Corrosion Control**
Mission Bay Block 30/32
San Francisco, CA

Dear Lisa,

Pursuant to your request, ***JDH Corrosion Consultants, Inc.***, has conducted a site corrosivity evaluation for the above referenced project site and we have provided herein recommendations for long-term corrosion control for the underground utilities at this site.

PROJECT BACKGROUND

This project involves the construction of commercial buidlings, located in San Francisco, CA at the intersection of South Street and Terry Francois Boulevard. We have assumed that the proposed structures will be supported on reinforced concrete or steel pile foundations and there will be buried utilities associated with the development.

PURPOSE

The purpose for this evaluation is to determine the corrosion potential, resulting from the soils at the subject site and to provide recommendations for long-term corrosion control for the concrete foundations and buried metallic utilities.

SOIL TESTING AND ANALYSIS

Ten (10) soil samples were collected from the site by ***Treadwell & Rollo, Inc.*** field personnel and transported to a state certified testing laboratory, **CERCO Analytical, Inc.** (certificate no. 2153) located in Pleasanton, CA for chemical analysis. Each sample was analyzed for pH, chlorides, resistivity, sulfates and Redox potential using ASTM test methods as detailed in the table below. The preparation of the soil samples for chemical analysis was in accordance with the applicable specifications.

Soil Analysis Test Methods

Chemical Analysis	ASTM Method
Chlorides	D512C
pH	D2976/D4972/G51
Resistivity	G 57
Sulfate	D516A(SM 4500)
Redox Potential	D1498

The results of the chemical analysis reported in the attached CERCO reports dated June 8, 2007 are as follows:

CERCO Analytical, Inc.
Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	N.D. – 1,900 mg/kg	Severely Corrosive to Non-corrosive*
pH	7.9 - 8.7	Non-Corrosive*
Resistivity (@100% saturation)	370 – 9,200 ohms-cm	Severely Corrosive to Mildly Corrosive *
Sulfate	18 - 180 mg/kg	Non-corrosive **
Redox Potential	410 - 480 mV	Non-corrosive*

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Chemical Testing Analysis

The chemical analysis provided by **CERCO Analytical, Inc.** indicates that the soils are "severely corrosive" to "mildly corrosive" with respect to steel and ductile iron based upon the resistivity measurements. The chloride levels indicate "severely corrosive" to "non-corrosive" conditions to steel and ductile iron. The sulfate levels indicate "non-corrosive" conditions for concrete structures placed into these soils with regard to sulfate attack. The pH of the soil indicates "non-corrosive" conditions to buried steel and concrete and the Redox potentials indicate aerobic conditions which are classified as "non-corrosive" to buried steel structures.

In-Situ Soil Resistivity Measurements

The in-situ resistivity of the soil was measured at two (2) locations at the project site by ***JDH Corrosion Consultants, Inc.*** field personnel. Resistance measurements were conducted with probe spacing of 2.5, 5, 7.5, 10 and 15-feet at each location.

For analysis purposes we have calculated the resistivity of soil layers 0-2.5, 2.5-5, 5-7.5, 7.5-10 and 10-15' using the Barnes Method as follows:

$\rho_{b\text{-}a} = KR_{(b\text{-}a)}$

Where;

$\rho_{b\text{-}a}$	=	soil resistivity of layer depth b-a (ohm-cm)
a	=	soil depth to top layer (ft)
b	=	soil depth to bottom layer (ft)
R_a	=	soil resistance read at depth a (ohms)
R_b	=	soil resistance read at depth b (ohms)
$R_{b\text{-}a}$	=	resistance of soil layer from a to b (ft)
K	=	layer constant = $60.96\pi(b\text{-}a)$ (cm)

and $$\frac{1}{R_{b\text{-}a}} = \frac{1}{R_a} - \frac{1}{R_b}$$

In-Situ Soil Resistivity Analysis

Corrosion of a metal is an electro-chemical process and is accompanied by the flow of electric current. Resistivity is a measure of the ability of a soil to conduct an electric current and is, therefore, an important parameter in consideration of corrosion data. Soil resistivity is primarily dependent upon the chemical content and moisture content of the soil mass.

The greater the amount of chemical constituents present in the soil, the lower the resistivity will be. As moisture content increases, resistivity decreases until maximum solubility of dissolved chemicals is attained. Beyond this point, an increase in moisture content results in dilution of the chemical concentration and resistivity increases.

The corrosion rate of steel in soil normally increases as resistivity decreases. Therefore, in any particular group of soils, maximum corrosion will generally occur in the lowest resistivity areas. The following classification of soil corrosivity, developed by William J. Ellis[1], is used for the analysis of the soil data for the project site.

Resistivity (Ohm-cm)	Corrosivity Classification
0 – 500	Very Corrosive
501 – 2,000	Corrosive
2,001 – 8,000	Moderately Corrosive
8,001 – 32,000	Mildly Corrosive
> 32,000	Progressively Less Corrosive

The above classifications are appropriate for the project site and the results are presented in the tables attached to the end of this report. In general, the soils are classified as "corrosive" with respect to corrosion of buried cast/ductile iron and steel structures throughout the top 2.5 -15 feet of the site.

The attached graph of the in-situ soil resistivity data for the soil layers 2.5' to 15' indicates that 13% of the soils are classified as "severely corrosive", 63% as "corrosive", and 25% as "moderately corrosive".

DISCUSSION

Reinforced Concrete Slab Foundations

The presence of water-soluble sulfate ions in the soils tested in the upper 10 ft of the soil at the site was at a low level. As such, Type II cement can be utilized for the concrete foundations. However the soils are corrosive and the chloride levels are high. In order to slow the ingress of aggressive ions, it is recommended that the water/cement ratio should not exceed 0.40 in order to achieve a dense concrete, with a minimum depth of cover of 3" over the reinforcing bars, especially in the areas where the foundation is more than a few feet deep.

Piles

Pre-stressed Pre-cast Reinforced Concrete Piles

The pre-stressed, pre-cast concrete piles will pass through the aggressive Bay mud. It is therefore recommended that Type II cement should be utilized. The water/cement ratio should not exceed 0.35 in order to achieve a dense concrete, with a minimum depth of cover of 2" over the pre-stressing wires. Also, a mineral admixture shall be added to the concrete mix.

Bare Steel Piles

Due to the corrosive soils being encountered, the piles are expected to experience corrosion, especially from ground level to 10 feet below the top of the Bay Mud. It is therefore recommended to use a corrosion allowance on all exposed surfaces of the piles. In addition use of coatings and cathodic protection may be required, depending upon the specific design of the structure.

Underground Metallic Pipelines

The soils at the project site are considered to be "corrosive" to ductile/cast iron, steel and dielectric coated steel. Therefore, we recommend the use of coatings, and/or polyethylene encasement, supplemented with cathodic protection for direct buried metallic pressure piping such as domestic and fire water pipelines. All underground pipelines should also be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to minimize potential galvanic corrosion problems.

RECOMMENDATIONS

Reinforced Concrete Slab Foundations

For application in foundation footings, we recommend using a Type II modified cement mix with a maximum water-to-cement ratio of 0.45 and a minimum depth of cover for the reinforcing steel of 3-inches. Also, a mineral admixture shall be added to the concrete mix. The amount of mineral admixture shall be 25% of the total amount of the cementitious material used in the concrete mix conforming to ASTM Designation: C618 type F or N (fly ash).

Piles

Pre-stressed Pre-cast Reinforced Concrete Piles

It is recommended that Type II cement should be utilized. The water/cement ratio should not exceed 0.35 in order to achieve a dense concrete, with a minimum depth of cover of 2" over the pre-stressing wires. Also, a mineral admixture shall be added to the concrete mix. The amount of mineral admixture shall be 25% of the total amount of the cementitious material used in the concrete mix and shall be comprised of 80% by mass mineral admixture conforming to ASTM Designation: C618 type F or N and 20% by mass mineral admixture meeting ASTM Designation: C 1240.

Bare Steel Piles

It is recommended to use a corrosion allowance on all exposed surfaces of the piles from ground level to 10 feet below the top of the Bay Mud. The exact length of the pile requiring the corrosion allowance will vary depending upon the design of the structure and the specific soils conditions for the subject piles. The amount of corrosion allowance (i.e. thickness) to be added to the piles is dependent upon the type of pile being used and the desired design life for the subject piles as provided in the following table:

Total Added Thickness for Corrosion Allowance

Pile Type	50-yr Design Life	75-yr Design Life	100-yr Design Life
Pipe Type Pile	(1/16") .0625-in.	(3/32") .09375-in.	(1/8") .125-in.
H-piles	(1/8") .125-in.	(3/16") .1875-in.	(1/4") .25-in.

A dielectric barrier such as a 10-mil thick polyethylene sheet, should also be installed between the pile cap or reinforced concrete foundation and the soil underneath to minimize the effects of the galvanic cell between steel in soil and steel in concrete. In addition the possible use of coatings and cathodic protection should be considered, depending upon the specific design of the steel supports.

Ductile Iron Pipe (Pressure Piping such as Domestic Water and Fire)

1. Direct buried ductile iron pipe should be encased in 8-mil polyethylene as specified in AWWA specification C-105. Epoxy coatings are also an acceptable alternative type of coating system for the pipe and/or fittings such as valves.

2. All rubber gasket joints, fusion epoxy coated flanges and flexible couplings on ductile iron pipelines should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

3. Insulating flanges and/or couplings should be installed to electrically isolate the buried portion of pipeline from other metallic pipelines, reinforced concrete structures and above grade buildings or structures.

4. Test stations shall be installed on all ductile iron pipelines at a spacing of 800 to 1,000 feet. Bonding and test stations shall comply with all applicable City Standards.

5. A sacrificial type of cathodic protection utilizing ***H-1 alloy magnesium*** anodes should be installed to protect the entire length of buried metallic pipelines. Cathodic protection should be designed in accordance with NACE Standard RP1069-02 and applicable City standards and included with the contract documents to permit installation along with the pipeline.

6. As an alternate, non-metallic piping may be used in lieu of ductile iron piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures. However, all metallic valves, fittings and appurtenances on non-metallic piping will require protection as specified below.

Ductile Iron Fittings & Metallic Valves (On Plastic Piping)

1. All direct buried ductile iron fittings installed on non-metallic piping shall be provided with a bituminous coating from the factory and encased in an 8-mil polyethylene bag in the field in accordance with AWWA Specification C-105. All bolts, restraining rods, etc. shall be coated with bitumastic prior to encasement in the polyethylene bag.

2. All metallic valves shall be coated from the factory (i.e. using powered epoxy or equivalent type of coating system) and all bolts shall be coated with bitumastic in the field and the entire valve shall be encased in an 8-mil polyethylene bag in accordance with AWWA Specification C-105.

3. A sacrificial type of cathodic protection utilizing ***H-1 alloy magnesium*** anodes should be installed to protect the valves and fittings. Cathodic protection should be designed in accordance with NACE Standard RP1069-02 and applicable City standards and included with the contract documents to permit installation along with the pipeline.

Steel Pipelines (Natural Gas Pipelines & Risers)

1. A fusion-bonded epoxy coating system or a suitable tape coating should be applied to all buried steel pipelines in accordance with ANSI/AWWA C214-95, "AWWA Standard for Tape Coating Systems for the Exterior of Steel Water Pipelines." Also, a tape coating per AWWA Standard C209-95 is recommended for special sections, connections and fittings.

2. Insulating flanges and/or couplings should be installed to electrically isolate the buried portions of steel pipelines from other metallic pipelines, reinforced concrete structures and above grade structures.

3. All rubber gasket joints, fusion epoxy coated flanges and flexible couplings should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

4. A sacrificial type of cathodic protection using ***H-1 alloy magnesium*** anodes should be installed to protect the buried portions of steel pipelines used for the natural gas piping systems. Cathodic protection should be designed in accordance with NACE Standard RP0169-02 and applicable City standards and included with the contract documents to permit installation along with the subject pipeline.

5. As an alternate, non-metallic piping may be used in lieu of steel piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures.

Sewer and Storm Drain Lines

1. Sewer and storm drain lines that will be routed underneath a concrete foundation should be encased in 8-mil polyethylene as specified in AWWA specification C-105.

Copper Water Pipelines (Service Lines)

1. Direct buried copper water services should be encased in 6-mil minimum polyethylene as specified in AWWA specification C-105.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees expressed or implied are provided.

We appreciate the opportunity to be of service to **Treadwell & Rollo,** on this project and trust that you find the analysis and recommendations contained herein satisfactory.

If you have any questions concerning the contents of this report or if we can be of any additional assistance, please do not hesitate to contact us at (925) 927-6630.

Respectfully submitted,

J. Darby Howard, Jr.

REGISTERED PROFESSIONAL ENGINEER
J. DARBY HOWARD, JR.
No. CR 1055
Exp. June 30, 2007
CORROSION
STATE OF CALIFORNIA

J. Darby Howard, Jr., P.E.
JDH Corrosion Consultants, Inc.
Principal

Sean Yost

Sean Yost
JDH Corrosion Consultants, Inc.
Project Engineer

cc: File 27085

REFERENCES

1. *Ellis, William J., Corrosion of Concrete Pipelines, Western States Corrosion Seminar, 1978*

2. *AWWA Manual of Water Supply Practices - M27, First Edition, External Corrosion - Introduction to Chemistry and Control (Denver, CO: 1987)*

3. *3. National association of Corrosion Engineers, Standard Recommended Practice, RP 01-69-02, Control of External Corrosion on underground or Submerged Pipeline*

Client: *Treadwell and Rollo*
Project: *Mission Bay B30/B32*
Location: *San Francisco, CA*
Date: *5/31/2007*
Subject: *In-Situ Soil Resistivity Data*

Severely Corrosive
Corrosive
Moderately Corrosive
Mildly Corrosive
Progressively Less Corrosive

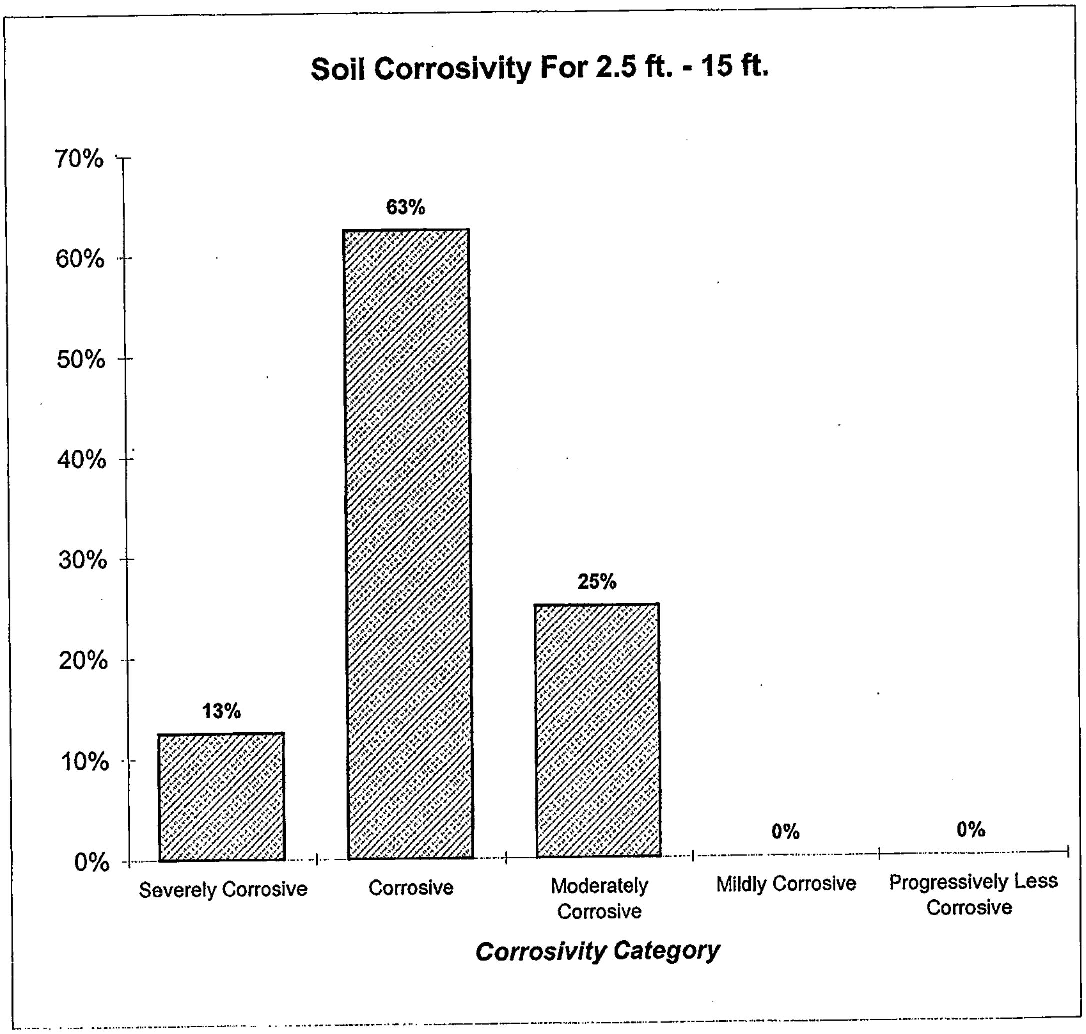
*Test	Location	Resistance Data From AEMC Meter						Soil Resistivities (ohm-cm)						Barnes Layer Analysis (ohm-cm)					
#	Description	2.5	5	7.5	10	15	20	2.5	5	7.5	10	15	20	0-2.5'	2.5-5'	5-7.5'	7.5-10"	10-15'	15-20'
1	Site 1	17.43	6.96	2.28	1.206	0.643	0	8345	6664	3275	2309	1847	NA	8345	5547	1623	1226	1319	NA
2	Site 2	19.23	4.67	1.217	0.631	0.14	0	9206	4472	1748	1208	402	NA	9206	2953	788	627	172	NA

Treadwell and Rollo
Mission Bay B30/B32
San Francisco, CA

In-Situ Soil Resistivity Data for soil layers 2.5' to 15'

Corrosivity Category	Resistivity (Ohm-Cm)			No. In Category	Total %	Cumulative %
Severely Corrosive	0	to	500	1	13%	13%
Corrosive	501	to	2000	5	63%	75%
Moderately Corrosive	2001	to	8000	2	25%	100%
Mildly Corrosive	8001	to	32000	0	0%	100%
Progressively Less Corrosive		Above	32001	0	0%	100%

Total Number of Tests = 8



APPROXIMATE IN-SITU RESISTIVITY & SOIL SAMPLE LOCATIONS.

WENNER 4-PIN TEST METHOD USED FOR IN-SITU SOIL RESISTIVITY MEASUREMENTS FOR LOCATIONS 1 AND 2.

0" ENG 1" ARCH 2"
SCALE: NTS

REV	DATE	ENG	CHK	DESCRIPTION

C A T H O D I C   P R O T E C T I O N

Treadwell & Rollo
San Francisco, CA
Mission Bay B30/B32

IN-SITU SOIL RESISTIVITY LOCATIONS

ENG: DH CAD: BY CHK: DH DATE: 5/31/2007 JOB: 27085 FILE: 27085

JDH Corrosion Consultants Incorporated

FIG-1

SHEET 1 OF 1

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

Client: JDH Corrosion Consultants, Inc.
Client's Project No.: 27085
Client's Project Name: Block 30/32
Date Sampled: Not Indicated
Date Received: 1-Jun-07
Matrix: Soil
Authorization: Transmittal dated 05/31/07

Date of Report: 8-Jun-2007

	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0706001-001	B30-1. 1M @ 2.5	470	7.9	-	2,900	-	33	60
0706001-002	B30-2,1 @ 3	470	8.1	-	9,200	-	N.D.	18
0706001-003	B30-3, 5 @ 10	480	8.4	-	380	-	1,900 (1)	45
0706001-004	B30-4, 4 @10	480	8.4	-	370	-	1,100	59
0706001-005	B30-5, 1 @ 3	470	8.1	-	3,300	-	N.D.	92
0706001-006	B32-1,1 @ 3	460	8.7	-	3,400	-	30	120
0706001-007	B32-2, 2 @ 3.5	460	8.6	-	6,800	-	N.D.	25
0706001-008	B32-3, 1 @ 2.5	460	8.3	-	8,000	-	N.D.	48
0706001-009	B32-4, 1 @ 3	450	8.4	-	2,600	-	67	180
0706001-010	B32-5, 5 @ 10.5	410	8.4	-	4,300	-	N.D.	140

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	4-Jun-2007	4-Jun-2007	-	4-Jun-2007	-	04-Jun-2007 & 5-Jun-2007	4-Jun-2007

* Results Reported on "As Received" Basis

N.D. - None Detected

(1) Detection limit is elevated to 75 mg/kg due to dilution

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

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APPENDIX D
Logs of Test Borings from Other Investigations

PROJECT: **BLOCK 32**
MISSION BAY
San Francisco, California

Log of Boring B32-1

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 4/30/07 | Date finished: 5/1/07

Drilling method: Rotary Wash

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

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

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: +105 feet[2])	Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1			SC	CLAYEY SAND with GRAVEL (SC) olive-brown, medium dense, moist trace brick and subangular gravel	FILL						
3	S&H	13		LL = 20, PI = NP							
6	SPT	3		very loose							
8	SPT	7		loose, with serpentinite fragments					25.9	13.7	
11	S&H	5									
11			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
12	SPT	5		∇ (4/30/07 at 1:40 pm)							
17	ST	50 to 75 psi		Consolidation Test, see Figure B-1		TxUU	1,050	275		66.8	60
25	ST	50 to 75 psi		Consolidation Test, see Figure B-2						57.6	66
30											

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Project No.: 4086.17 | Figure: A-1a

PROJECT:	BLOCK 32 MISSION BAY San Francisco, California	Log of Boring B32-1 PAGE 2 OF 4

DEPTH (feet)	Sampler Type	Sample	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
				CH	CLAY (CH) (continued)						
31											
32											
33											
34											
35	ST		0 to 75 psi		BAY MUD						
36											
37–41											
42				SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet						
43–44											
45	SPT		26								
46–47				CL	CLAY (CL) olive, stiff to very stiff, wet, with trace silt						
48–49											
50	SPT		16								
51–53											
54				SC	CLAYEY SAND (SC) yellow-brown, medium dense to dense, wet						
55	S&H		30								
56–58											
59					SILTY SAND (SM) orange-brown, medium dense, wet COLMA						
60	SPT		28						14.9	24.7	

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Project No.: 4086.17 | Figure: A-1b

PROJECT: **BLOCK 32 MISSION BAY** San Francisco, California

Log of Boring B32-1

DEPTH (feet)	Sampler Type	Sample	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
60	SPT		28	SM	SILTY SAND (SM) (continued)	COLMA						
62				SP-SC	SAND with CLAY (SP-SC) orange-brown, very dense, wet, trace fines, medium grained sand							
65	SPT		46/ 5.5"							11.7	22.8	
70	SPT		56							8.8	22.6	
75	SPT		59									
80	SPT		56		olive, fine-grained sand							
83					SERPENTINITE intensely fractured, weak, moderately weathered, moderately hard	BEDROCK						
85	SPT		88/ 3"									
90	SPT		56		plastic, soft							

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Project No.: 4086.17

Figure: A-1c

PROJECT: **BLOCK 32**
MISSION BAY
San Francisco, California

Log of Boring B32-1

DEPTH (feet)	Sampler Type	Sample	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
	SPT		56		SERPENTINITE (continued)						
91											
92											
93											
94	SPT		50/3"		friable, low hardness						
95					BEDROCK						
96											
97											
98											
99	SPT		50/3"		weak						
100											
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 99.25 feet.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 12.5 feet at 1:40 pm on 4/30/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

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Project No.: 4086.17 | Figure: A-1d

PROJECT: **BLOCK 32 MISSION BAY** San Francisco, California

Log of Boring B32-2

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 4/27/07 Date finished: 4/30/07

Drilling method: Rotary Wash

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

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

Ground Surface Elevation: +101 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–4			SC	SAND with CLAY (SP-SC) gray-brown, medium dense, moist, with traces of brick and angular gravel (FILL)						
3	S&H	12								
4.5–11			SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, with fragments of bricks (FILL)						
6	SPT	14								
8	SPT	4		∇ (4/27/07 at 2:45 pm) olive-brown, very loose to loose, wet						
10.5	SPT	2		very loose						
11–30			CH	CLAY (CH) gray, soft, wet, with shell fragments (BAY MUD)						
13	ST	0 to 75 psi								
19–20.5	ST	50 to 150 psi			TxUU	850	345		59.1	65
29.5	ST									

TEST GEOTECH LOG 408617.GPJ TR.GDT 8/3/07

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Project No.: 4086.17 Figure: A-2a

PROJECT:	BLOCK 32 MISSION BAY San Francisco, California	Log of Boring B32-2 PAGE 2 OF 3

DEPTH (feet)	Sampler Type	Sample	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
31	ST		0 to 150 psi	CH	CLAY (CH) (continued) sandy at 30.5 feet	BAY MUD						
32–34												
35	S&H		4		soft to medium stiff							
36–39												
40	ST		100 to 300 psi	SP	SAND (SP) gray, wet							
41												
42				CL	CLAY (CL) olive, very stiff, wet							
43–44												
45	S&H		16				TxUU	1,650	1,540		29.6	94
46–49												
50	S&H		13		stiff							
51–52												
53				SC	CLAYEY SAND (SC) yellow-brown, dense, wet	COLMA						
55	SPT		34							18.7	22.0	
56–59												
60	SPT		31									

TEST GEOTECH LOG 408617.GPJ TR.GDT 8/3/07

Treadwell&Rollo

Project No.: 4086.17 Figure: A-2b

PROJECT:	BLOCK 32 MISSION BAY San Francisco, California	Log of Boring B32-2 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	Sample	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
	SPT		31		CLAYEY SAND (SC) (continued)	COLMA						
61				SC								
62					SERPENTINITE intensely fractured, friable, moderately weathered, low harness	BEDROCK						
63												
64	SPT		50/ 3"									
65												
66												
67												
68												
69	SPT		50/ 4.5"									
70												
71												
72												
73												
74												
75												
76												
77												
78												
79												
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 69.4 feet.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet at 2:45 pm on 4/27/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

Treadwell&Rollo

Project No.: 4086.17 | Figure: A-2c

TEST GEOTECH LOG 408617.GPJ TR.GDT 8/3/07

Log of Boring B32-3

PROJECT: **BLOCK 32 MISSION BAY** San Francisco, California

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 4/25/07 | Date finished: 4/26/07

Drilling method: Rotary Wash

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

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

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: +99.5 feet[2])		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–3			SC	CLAYEY SAND with GRAVEL (SC) dark gray, loose, moist, with fragments of brick and concrete	FILL						
3	S&H	5	SC		FILL						
5	SPT	9	SC	olive-brown, trace gravel	FILL						
7			SC	▽ (4/25/07 at 3:30 pm)	FILL						
8	SPT	4	CL	CLAY (CL) black, soft to medium stiff, wet, majority of sample is wood	FILL						
9.5			SC	CLAYEY SAND with GRAVEL (SC) dark brown, loose, wet, with fragments of bricks	FILL						
10.5	S&H	6	SC		FILL				13.8	23.6	
12	SPT	9	SC		FILL						
13			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
17	ST	50 psi	CH		BAY MUD						
24.5	ST	75 psi	CH	Consolidation Test, see Figure B-3	BAY MUD					50.9	71
30											

TEST GEOTECH LOG 408617.GPJ TR.GDT 8/3/07

Treadwell&Rollo

Project No.: 4086.17 | Figure: A-3a

DEPTH (feet)	Sampler Type	Sample	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
30–34				CH	CLAY (CH) (continued)	BAY MUD						
34–36	ST		100 to 250 psi									
35–39				SP	SAND (SP) gray, wet							
39–41.5	SPT		51	CL	CLAY with GRAVEL (CL) yellow-brown with olive mottling, hard, wet							
41.5–44					CLAYSTONE intensely fractured, weak, moderately weathered, low hardness	BEDROCK						
44	SPT		50/ 3"									
49–50	SPT		64		plastic							
52					SHALE intensely fractured, friable, moderately weathered, low hardness							
54	SPT		50/ 5"									
59–60	SPT		69		plastic							

PROJECT: **BLOCK 32 MISSION BAY** San Francisco, California

Log of Boring B32-3

DEPTH (feet)	Sampler Type	Sample	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
60	SPT		69		SHALE (continued)	BEDROCK						
64	SPT		50/ 3"		friable							
69	SPT		50/ 3"									

Depth scale: 61–90 feet

Boring terminated at a depth of 69.25 feet.
Boring backfilled with cement grout.
Groundwater encountered at 7 feet at 3:30 pm on 4/25/07.

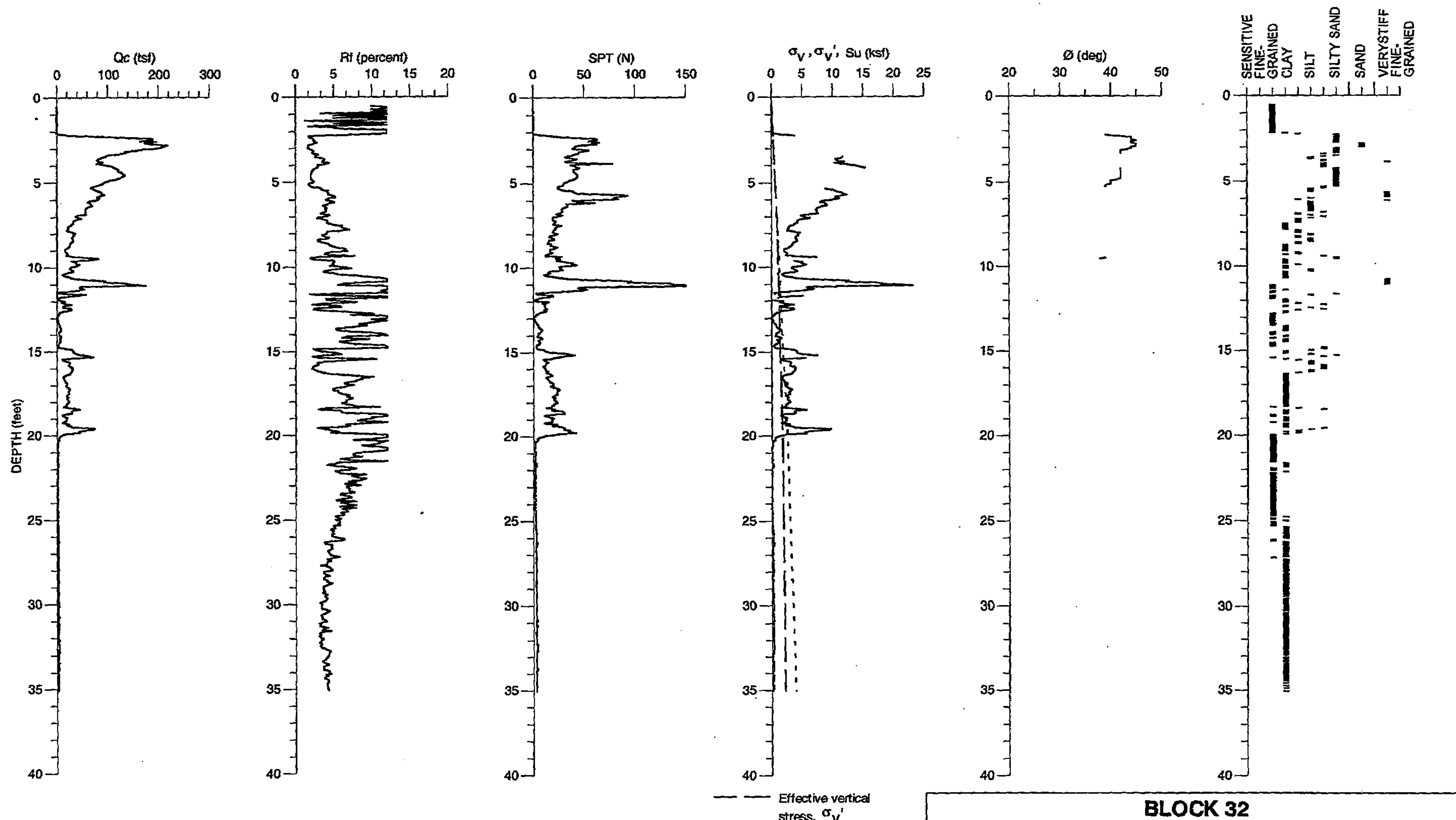
[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on San Francisco City Datum plus 100 feet.

Treadwell&Rollo

Project No.: 4086.17

Figure: A-3c

TEST GEOTECH LOG 408617.GPJ TR.GDT 8/3/07



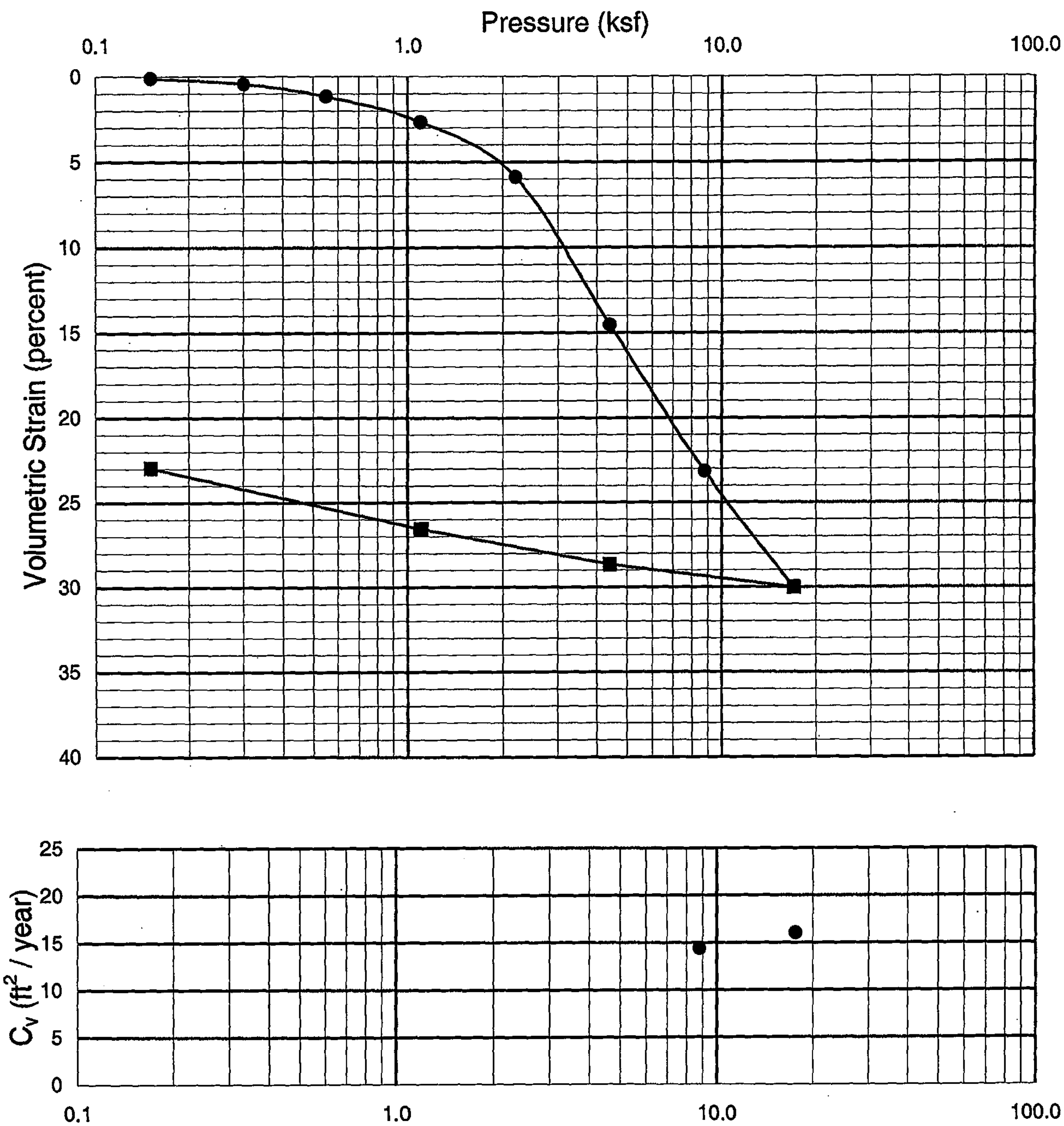
Terminated at 79.6 feet.
Groundwater assumed to be at a depth of 7.0 feet bgs.
Date performed: 08/08/06.
Elevation: 100.6 feet, Datum: San Francisco City Datum +100 feet.

BLOCK 32
MISSION BAY EAST
San Francisco, California

CONE PENETRATION TEST RESULTS
C32-1

Date 07/18/07	Project No. 4086.17	Figure A-8

Treadwell&Rollo



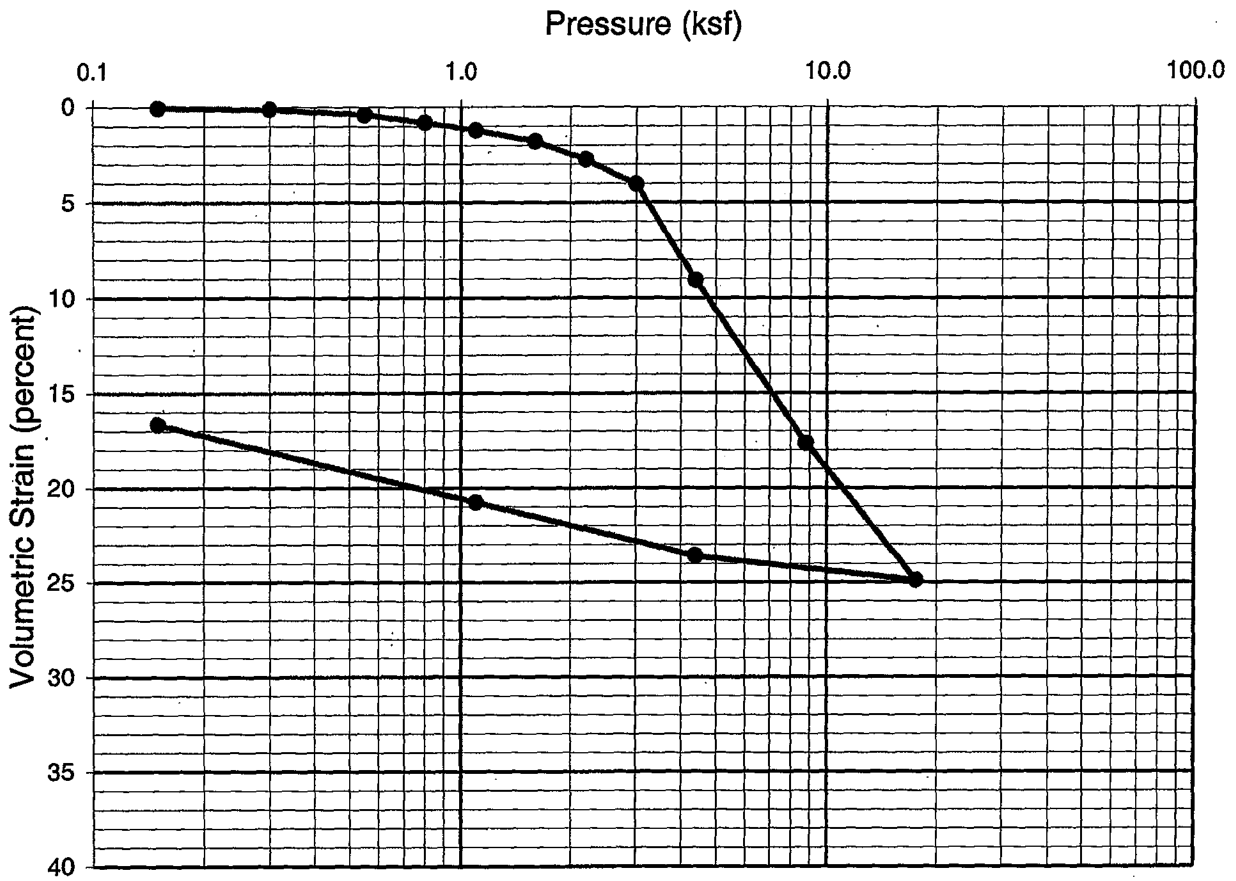
Sampler Type: Shelby Tube				Condition	Before Test			After Test	
Diameter (in) 2.41		Height (in)	1.00	Water Content	w_o	70.4 %	w_f	48.1	%
Overburden Pressure, p_o	1,600	psf		Void Ratio	e_o	1.91	e_f	1.30	
Preconsol. Pressure, p_c	1,900	psf		Saturation	S_o	99 %	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.64			Dry Density	γ_d	58 pcf	γ_d	48	pcf
Recompression Ratio, $C_{\varepsilon r}$	0.04						G_s	2.70	(assumed)
Classification CLAY (CH), dark gray to gray				Source		B32-1 @ 16.5 feet			

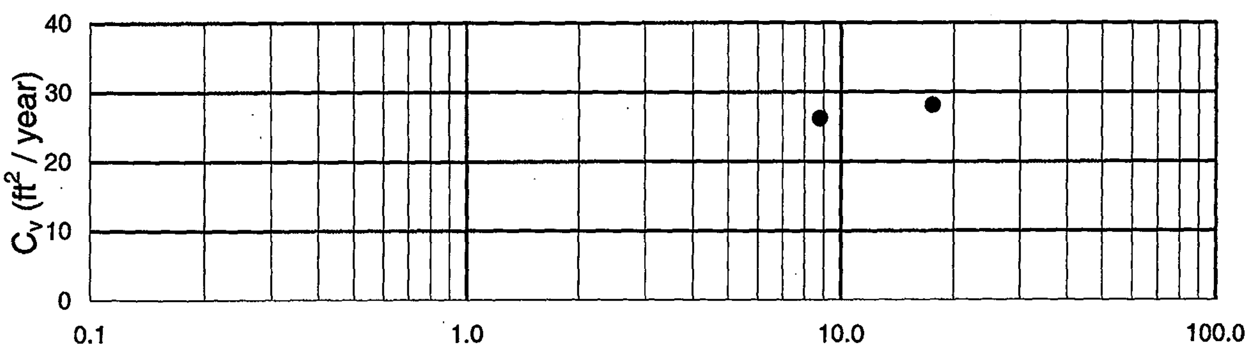
BLOCK 32
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 08/03/07 | Project No. 4086.17 | Figure B-1





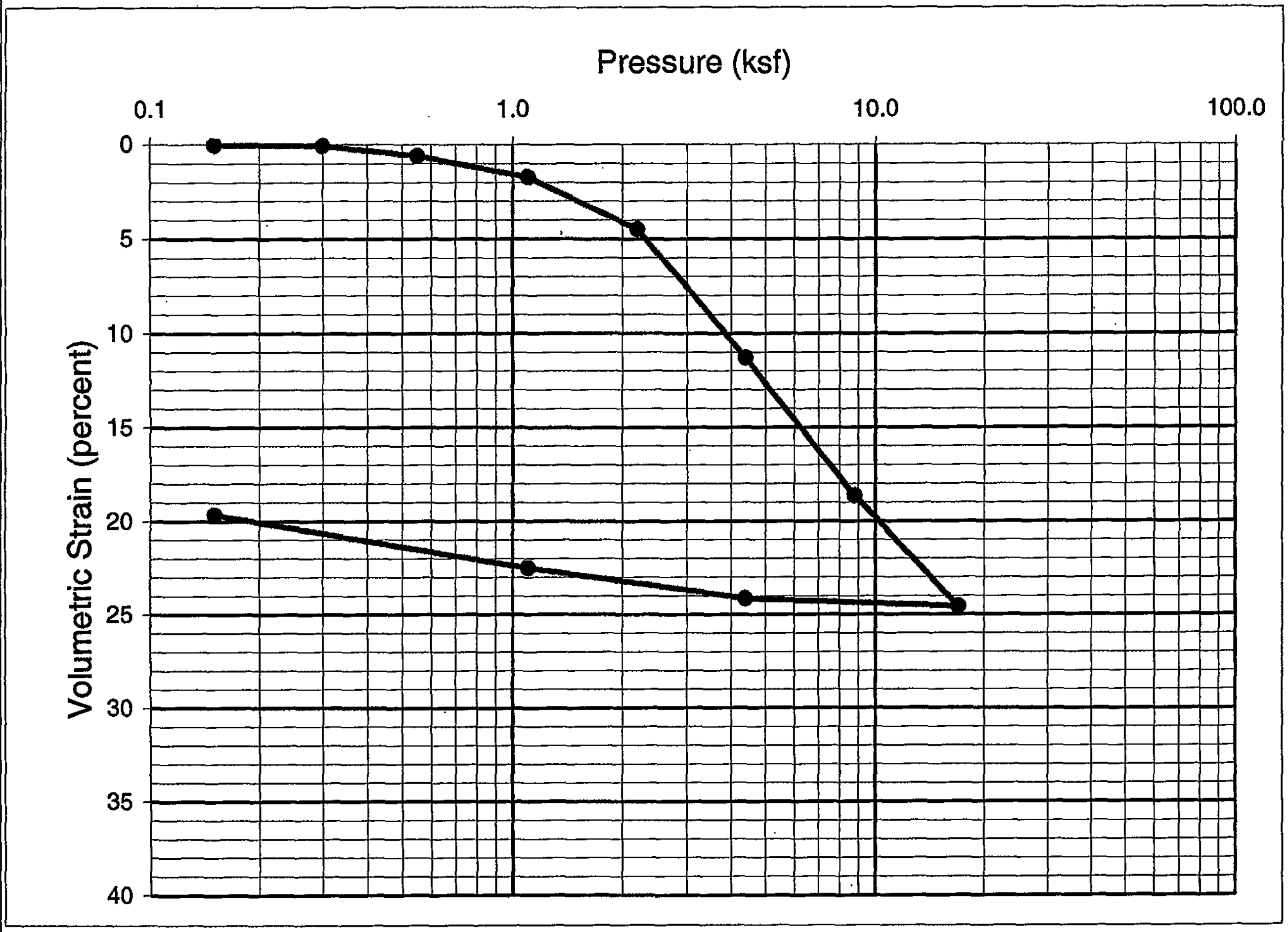
Sampler Type: Shelby Tube				Condition		Before Test		After Test	
Diameter (in) 2.42		Height (in) 1.01		Water Content	w_o	57.6 %		w_f	43.3 %
Overburden Pressure, p_o	1,900 psf			Void Ratio	e_o	1.56		e_f	1.17
Preconsol. Pressure, p_c	2,900 psf			Saturation	S_o	100 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.29			Dry Density	γ_d	66 pcf		γ_d	78 pcf
LL		PL		PI			G_s 2.70	(assumed)	
Classification CLAY (CL), gray					Source	B32-1 @ 24'			

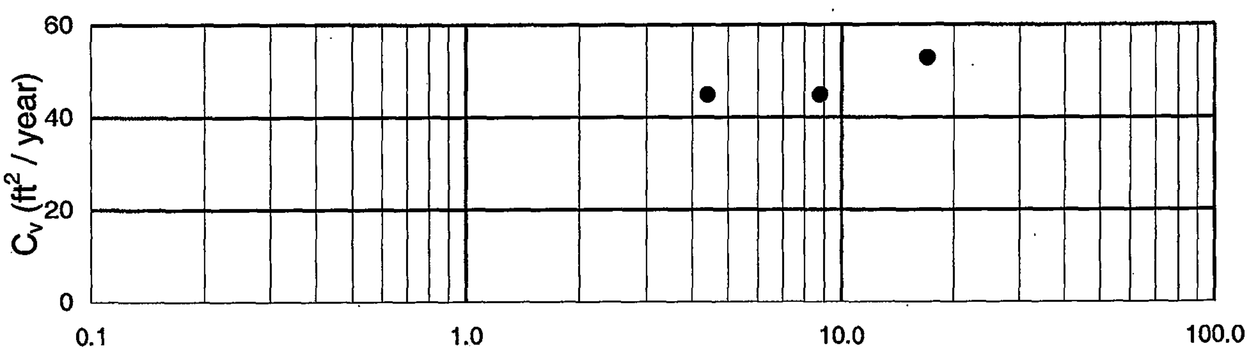
BLOCK 32 - MISSION BAY EAST
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 08/03/07 | Project No. 4086.17 | Figure B-2





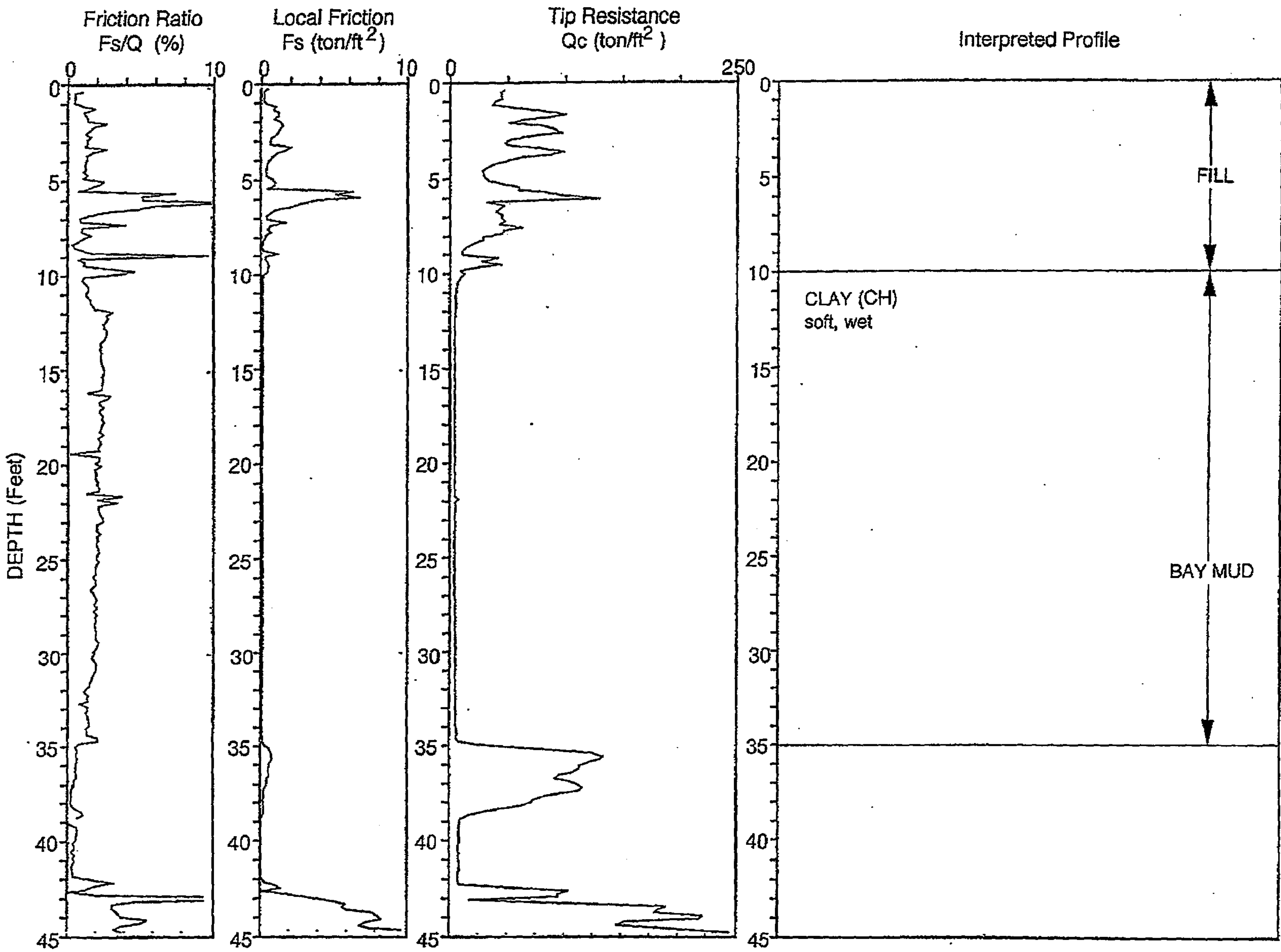
Sampler Type: Shelby Tube			Condition	Before Test			After Test		
Diameter (in) 2.41	Height (in)	1.00	Water Content	w_o	50.9	%	w_f	35.9	%
Overburden Pressure, p_o	1,600	psf	Void Ratio	e_o	1.39		e_f	0.97	
Preconsol. Pressure, p_c	2,000	psf	Saturation	S_o	99	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.25		Dry Density	γ_d	71	pcf	γ_d	86	pcf
LL	PL		PI				G_s	2.70	(assumed)
Classification CLAY (CH), gray				Source	B32-3 @ 24'				

BLOCK 32 - MISSION BAY EAST
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 08/03/07 | Project No. 4086.17 | Figure B-3

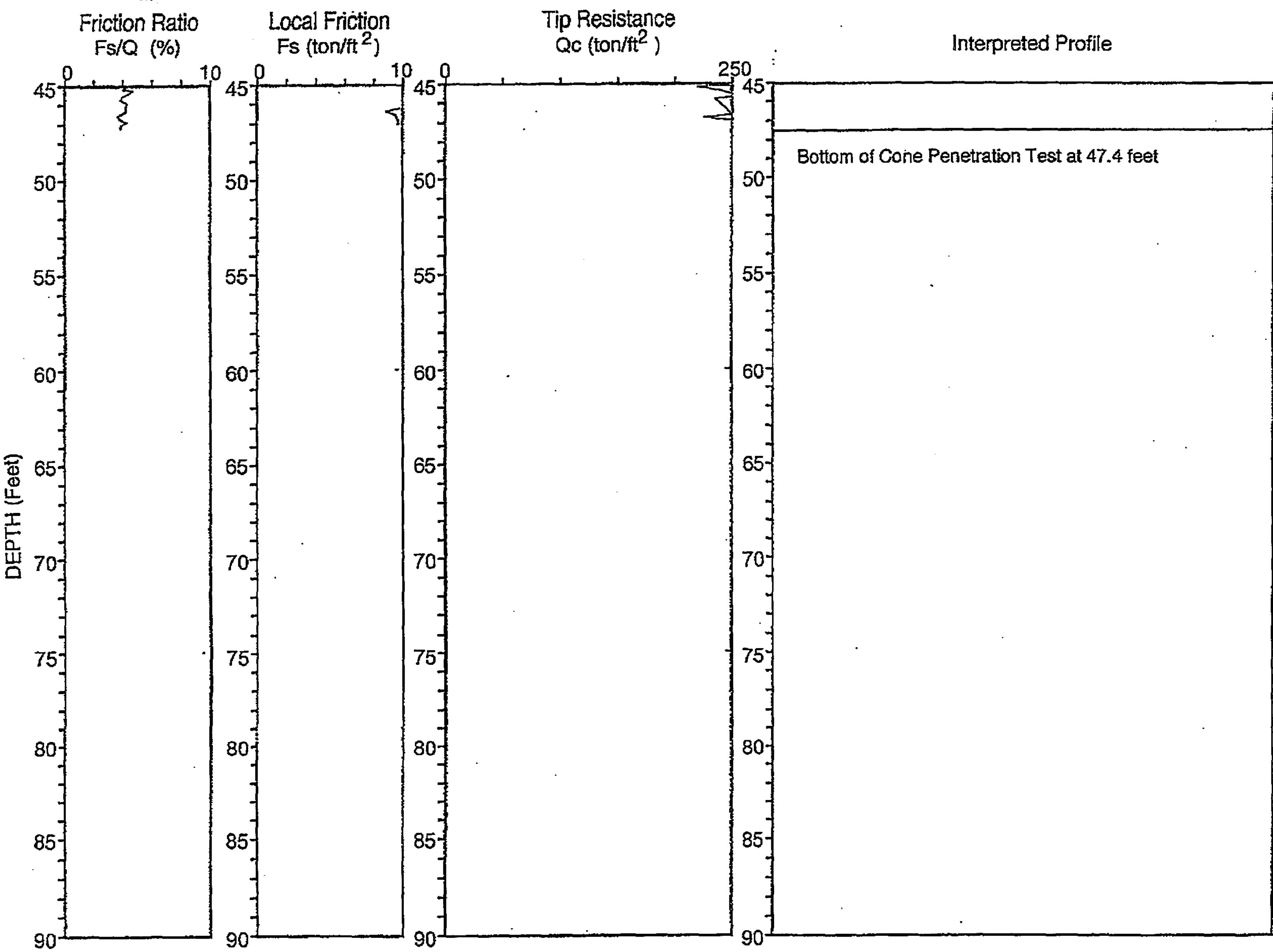


Operator: VBI In-Situ Testing

Date: 12-3-92

130

TREADWELL & ROLLO, INC. Consulting Engineers and Scientists	LOG OF CPT-M3	
Trans Pacific Geotechnical Consultants, Inc.	Project No. 1273-004	Plate A-18a



Operator: VBI In-Situ Testing

Date: 12-3-92

MISSION BAY INFRASTRUCTURE
San Francisco, California

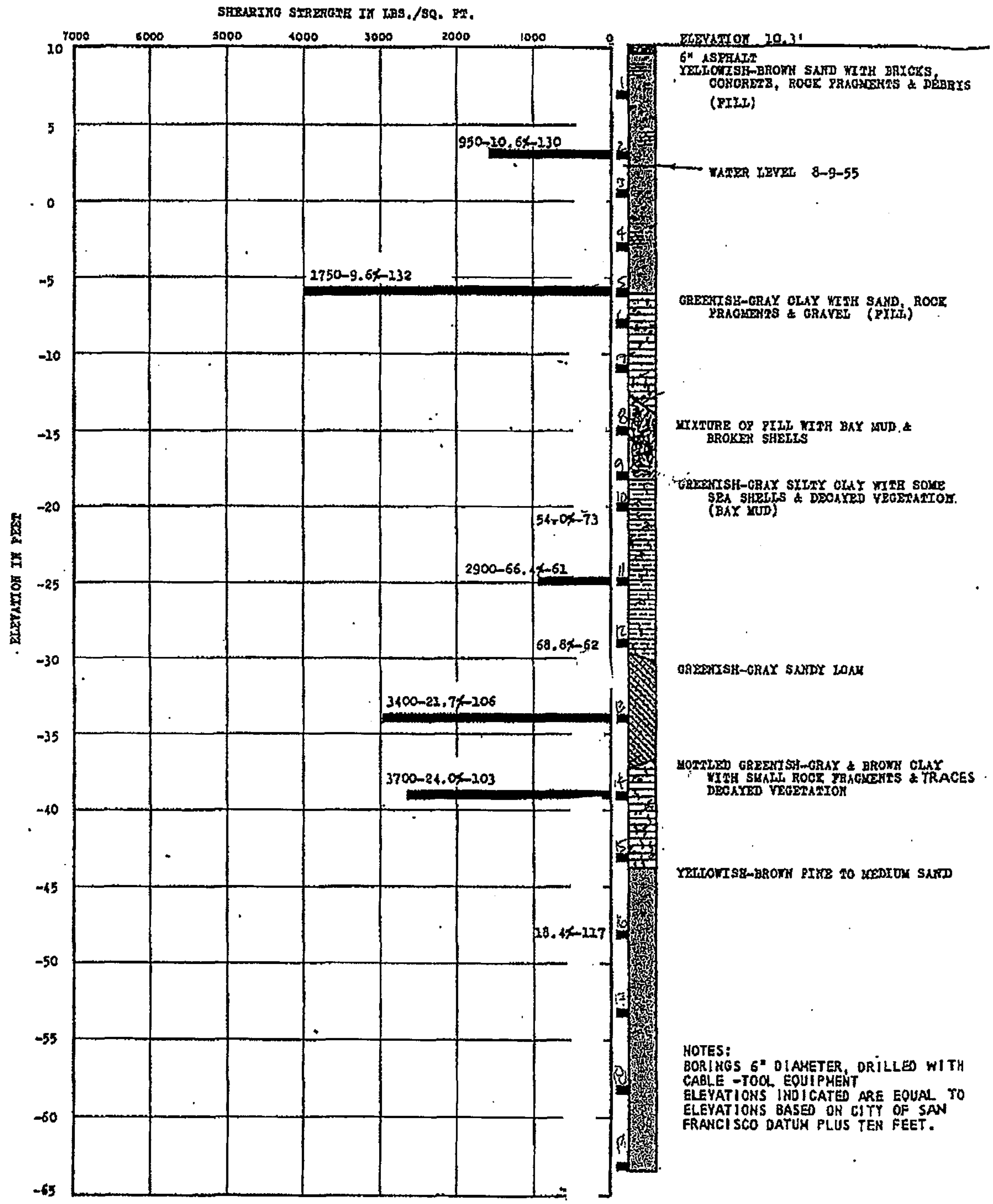
Trans Pacific Geotechnical Consultants, Inc.

LOG OF CPT-M3 (continued)

Project No. 1273-004

Plate A-18b

BORING 1
DRILLED 8-5-55 & 8-6-55



NOTES:
BORINGS 6" DIAMETER, DRILLED WITH CABLE -TOOL EQUIPMENT
ELEVATIONS INDICATED ARE EQUAL TO ELEVATIONS BASED ON CITY OF SAN FRANCISCO DATUM PLUS TEN FEET.

LOG OF BORING

DAMES & MOORE
SOIL MECHANICS ENGINEERS

PLATE 2A

Treadwell&Rollo

APPENDIX E
Probabilistic Seismic Hazard Analysis

APPENDIX E

PROBABILISTIC SEISMIC HAZARD ANALYSIS

This appendix presents the details of our estimation of the level of ground shaking at the site during future earthquakes. Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a probabilistic seismic hazard analysis (PSHA), which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are that:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data
- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake
- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

To develop site-specific design response spectra for the project, we:

- performed a probabilistic seismic hazard analysis (PSHA) to develop uniform hazard response spectrum for rock outcropping with a hazard level corresponding to a 10 percent probability of exceedance in 50 years (475-year return period), consistent with the definition of Design Basis Earthquake (DBE) in 2001 San Francisco Building Code (SFBC)
- performed spectral matching of three recorded time-histories to the rock spectrum for use as input motions in ground response analyses
- performed ground response analyses to compute response spectra at the ground surface for the DBE hazard level
- developed recommended, smooth, horizontal spectrum for DBE.

The PSHA to develop the DBE rock spectrum was performed using the computer code EZFRISK 7.23 (Risk Engineering 2007). The approach used in EZFRISK is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using rock attenuation relationships that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault.

E1.0 PROBABILISTIC MODEL

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. The fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i v_i \iint P[Z > z \mid m, r] f_{M_i}(m) f_{R_i|M_i}(r; m)\, dr\, dm$$

where:

v_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source I

P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z

$f_{Mi}(m)$ and $f_{Ri|Mi}(r;m)$ = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

E2.0 SOURCE MODELING AND CHARACTERIZATION

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 E-1.

TABLE E-1

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

The segmentation of faults, mean characteristic magnitudes, and recurrence rates were modeled using the data presented in the WGCEP (2003) and Cao et al. (2003) reports. We also included the floating sources as described in the WGCEP (2003) in our seismic hazard model. Table E-2 presents the distance and direction from the site to the fault, mean characteristic magnitude, mean slip rate, and fault length for individual fault segments. We used the California fault database identified as "USGS02" in EZFRISK 7.14 which we understand was obtained directly from USGS as a dataset with multiple fault segments, and each segment being characterized with multiple magnitudes, occurrence or slip rates, and weights (McGuire 2005).

TABLE 2

Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Fault Length (km)
San Andreas - 1906 Rupture (SAS+SAP+SAN+SAO)	12.6	West	7.90	19	473
San Andreas - floating	12.6	West			
San Andreas - Peninsula (SAP)	12.6	West	7.15	17	85
San Andreas - SAP+SAN+SAO	12.6	West	7.83		411
San Andreas - SAS+SAP	12.6	West	7.42	17	147
San Andreas - SAS+SAP+SAN	12.6	West	7.76		338
Hayward-Rodgers Creek - NH	16.4	Northeast	6.49	9	35
Hayward-Rodgers Creek - NH+RC	16.4	Northeast	7.11	9	98
Hayward-Rodgers Creek - SH+NH	16.4	Northeast	6.91	9	88
Hayward-Rodgers Creek - SH+NH+RC	16.4	Northeast	7.26	9	151
San Andreas - SAN	16.6	West	7.45	24	191
San Andreas - SAN+SAO	16.6	West	7.70	24	330
Hayward-Rodgers Creek - SH	16.7	East	6.67	9	53
San Gregorio - SGN	19.0	West	7.23	7	110
San Gregorio - SGS+SGN	19.0	West	7.44	5	176
Mt Diablo - MTD	32.9	East	6.65	2	25
Calaveras - CC+CN	33.7	East	6.90		104
Calaveras - CN	33.7	East	6.78	6	45
Calaveras - CS+CC+CN	33.7	East	6.93		123
Hayward-Rodgers Creek - RC	35.8	North	6.98	9	63
Concord/GV - CON	37.8	East	6.25	4	20
Concord/GV - CON+GVS	37.8	East	6.58		42
Concord/GV - CON+GVS+GVN	37.8	East	6.71		56
Monte Vista-Shannon	38.7	Southeast	6.80	0.4	41
Concord/GV - GVS	40.7	Northeast	6.24	5	22
Concord/GV - GVS+GVN	40.7	Northeast	6.24	5	36
Point Reyes	43.9	West	6.80	0.3	47
West Napa	45.9	Northeast	6.50	1	30
Greenville - GN	50.4	East	6.66	2	27
Greenville - GS+GN	50.4	East	6.94	2	51
Hayward - South East Extension	54.8	Southeast	6.40	3	26
Concord/GV - GVN	58.6	Northeast	6.02	5	14
Great Valley 6	61.1	East	6.70	1.5	45
Calaveras - CC	62.5	Southeast	6.23	15	59
Calaveras - CS+CC	62.5	Southeast	6.36	15	78
Greenville - GS	64.2	East	6.60	2	24

TABLE E-2 (continued)
Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Fault Length (km)
Great Valley 5	66.1	East	6.50	1.5	28
Great Valley 4	73.0	Northeast	6.60	1.5	42
San Andreas - Santa Cruz Mnts. (SAS)	74.1	Southeast	7.03	17	62
Great Valley 7	76.1	East	6.70	1.5	45
Hunting Creek-Berryessa	78.1	North	6.90	6	60
Sargent	80.4	Southeast	6.80	3	53
Zayante-Vergeles	84.0	Southeast	6.80	0.1	56
Maacama-garberville	93.8	North	6.90	9	
Monterey Bay-Tularcitos	97.1	South	7.10	0.5	84

E3.0 ATTENUATION RELATIONSHIPS

Four rock attenuation relationships were used in our study. These were: Abrahamson and Silva (1997), Idriss (1993), Campbell (1997), and Sadigh et al. (1997). The attenuation relationships used in the study were developed using different earthquake databases which treat the magnitude and distance effects differently. Therefore, the average of the rock relationships was used to develop the recommended rock spectrum.

E4.0 ROCK PSHA RESULTS

Figure E-1 presents the results of the PSHA for the DBE using the various attenuation relationships and the average of the results from the four relationships. The average result is recommended for the DBE level of shaking. Figure E-2 presents the recommended rock spectrum for DBE, as well as a comparison with the 2001 SFBC rock spectrum.

E5.0 TIME HISTORY MATCHING FOR ROCK SPECTRUM

To develop time histories that are compatible with the recommended rock spectrum shown on Figure E-2, we performed spectral matching of the rock spectrum with actual recorded ground motions. The selection of a recorded time history is an important step in developing the ground motion. The intent in this selection process is to choose time histories that have a similar magnitude and distance to the design ground motion. In addition, the use of different earthquakes captures the unique and different character of each particular earthquake. Table E-3 presents the earthquake time histories used in the spectral matching for rock.

TABLE E-3
Earthquake Time Histories Used
For Matching Rock Spectra

Earthquake	Recording	Magnitude	Closest Distance to Rupture (km)	Peak Acceleration (g)
Loma Prieta, 1989	Corralitos	6.9	5	0.644
Kocaeli, 1999	Gebze	7.4	17	0.244
Landers, 1992	Joshua Tree	7.4	12	0.274

The tabulated reference time histories were modified such that their response spectrum matched the target spectrum. The computer program EZFRISK 7.23 was used to perform the spectral matching. The spectral matching was performed in the time domain. Figures E-3 through E-5 present the acceleration, velocity and displacement of the matched time histories along with the comparison between the target and the matched spectrum.

E6.0 GROUND RESPONSES ANALYSES AND RECOMMENDED RESPONSE SPECTRUM

To develop a site-specific response spectrum, the ground motion should be modified to take into account the soil conditions at the site. To capture the variations in the subsurface conditions at the site, we developed two idealized soil profiles based on data from current and previous investigations at the site.

The first profile, designated "shallow" soil profile consisted of 15 feet of sandy/gravelly fill over 25 feet of Bay Mud which is in turn underlain by 10 feet of dense Colma sand. Bedrock was modeled at a depth of 50 feet below the ground surface.

The second profile, designated "deep" soil profile consisted of 9 feet of sandy/gravelly fill over 45 feet of Bay Mud which is turn underlain by 13 feet of stiff clay. The clay stratum overlies a 30 feet thick layer of dense to very dense Colma sand which is in turn underlain by 9 feet of stiff Old Bay Clay. Bedrock was modeled at a depth of 106 feet below the ground surface.

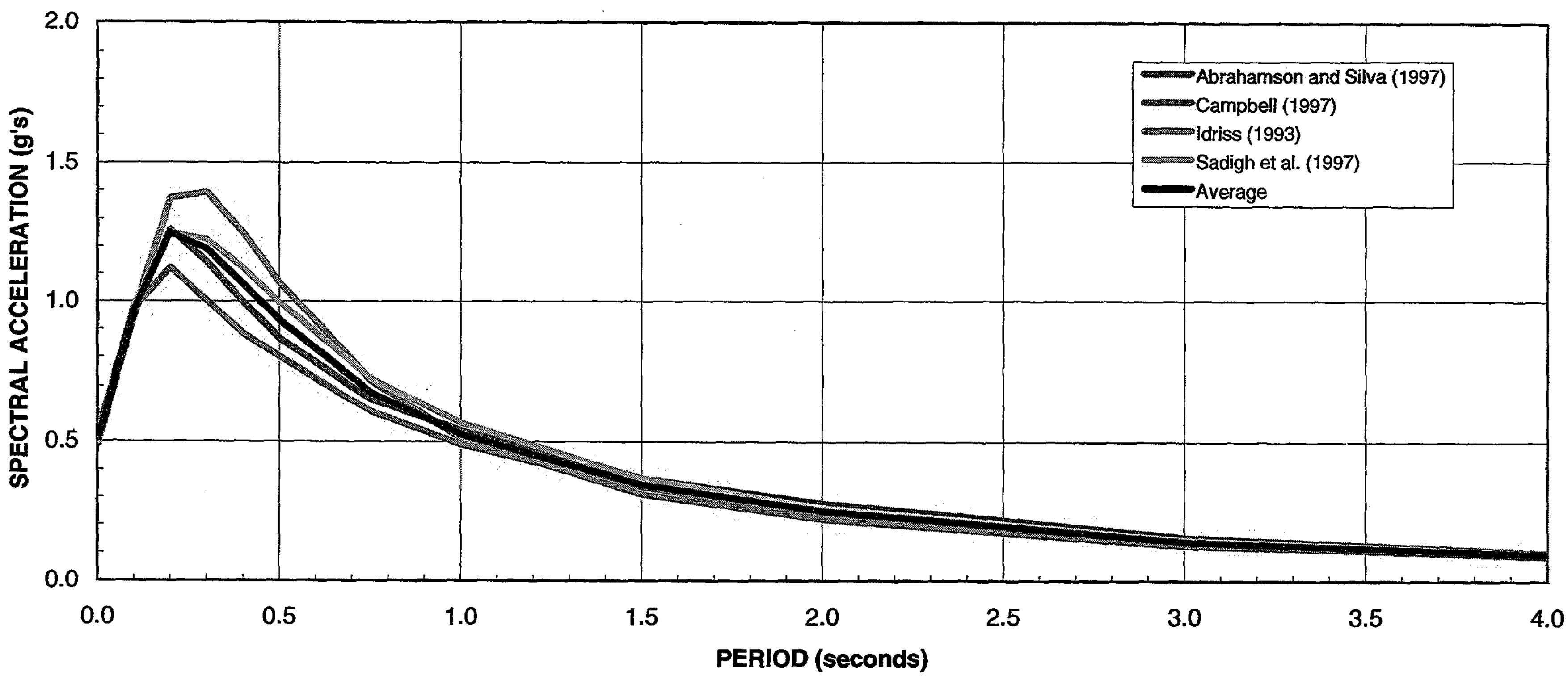
Response spectra at the ground surface were computed using the computer program SHAKE91. SHAKE91 is a one-dimensional, site response analysis based on vertically propagating, horizontal shear waves. The program mathematically transmits input bedrock motions vertically through an idealized soil column to the ground surface. To account for the non-linear characteristics of soil, this program uses equivalent-linear procedures with strain compatible shear moduli and damping ratios.

Based on the results of the geotechnical investigations, potentially liquefiable sand was encountered at the site; hence the site is classified as S_F. Therefore, we also modeled the effect of liquefaction in our analyses. Recorded data from previous earthquakes (Loma Prieta and Kobe) have shown that the effect of liquefaction is to damp out the high frequency (short period) ground motion. However, the same data suggest that some portion of the high frequency part of the ground motion may be transmitted to the ground surface prior to the initiation of liquefaction. Therefore, in our analyses we considered both the effects of liquefaction and no liquefaction on the computed surface spectra. The results of the SHAKE analyses for the "shallow" and "deep" profiles for non-liquefied condition are presented on Figures E-6 and E-7, respectively. Similar plots of the results for the liquefied condition are presented on Figures E-8 and E-9. Averages of the computed spectra are also shown on these figures. Figure E-10 presents the average results for each profile and condition, as well as the average of all the results. The smooth recommended spectrum for DBE is shown on Figure E-11. Digitized values of the recommended spectrum for a damping ratio of 5 percent are presented in Table E-4.

TABLE E-4

Recommended Spectral Acceleration (g)
Damping Ratio of 5 percent

Period (seconds)	Recommended DBE Spectral Acceleration
0.00	0.450
0.10	0.536
0.20	0.751
0.30	0.924
0.40	1.056
0.50	1.149
0.60	1.204
0.70	1.225
0.80	1.216
0.90	1.182
1.00	1.126
1.10	1.055
1.20	0.971
1.30	0.881
1.40	0.788
1.50	0.695
1.60	0.606
1.70	0.524
1.80	0.451
1.90	0.388
2.00	0.350
2.50	0.233
3.00	0.174
4.00	0.111



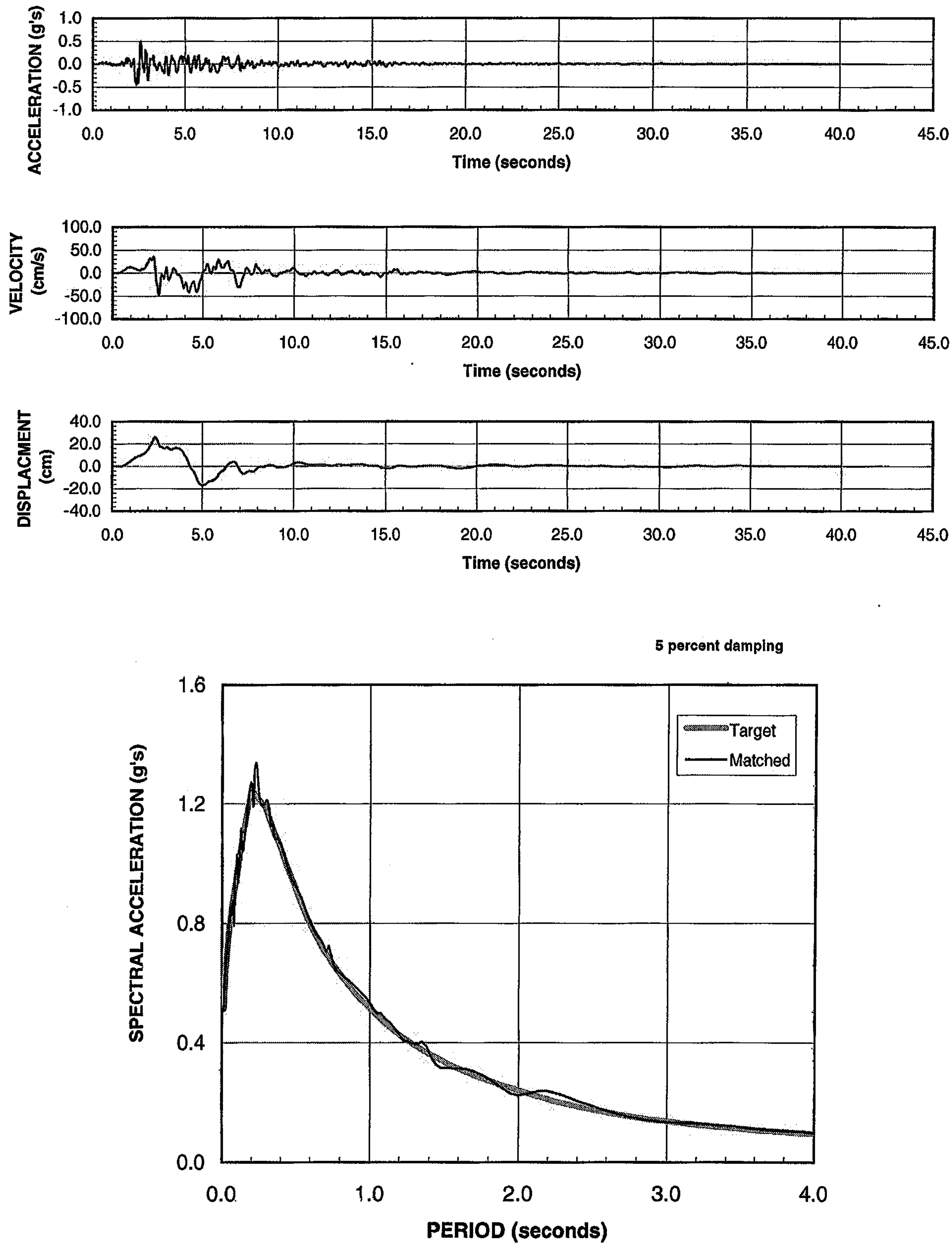
Damping Ratio = 5%

BLOCK 30
MISSION BAY
San Francisco, California

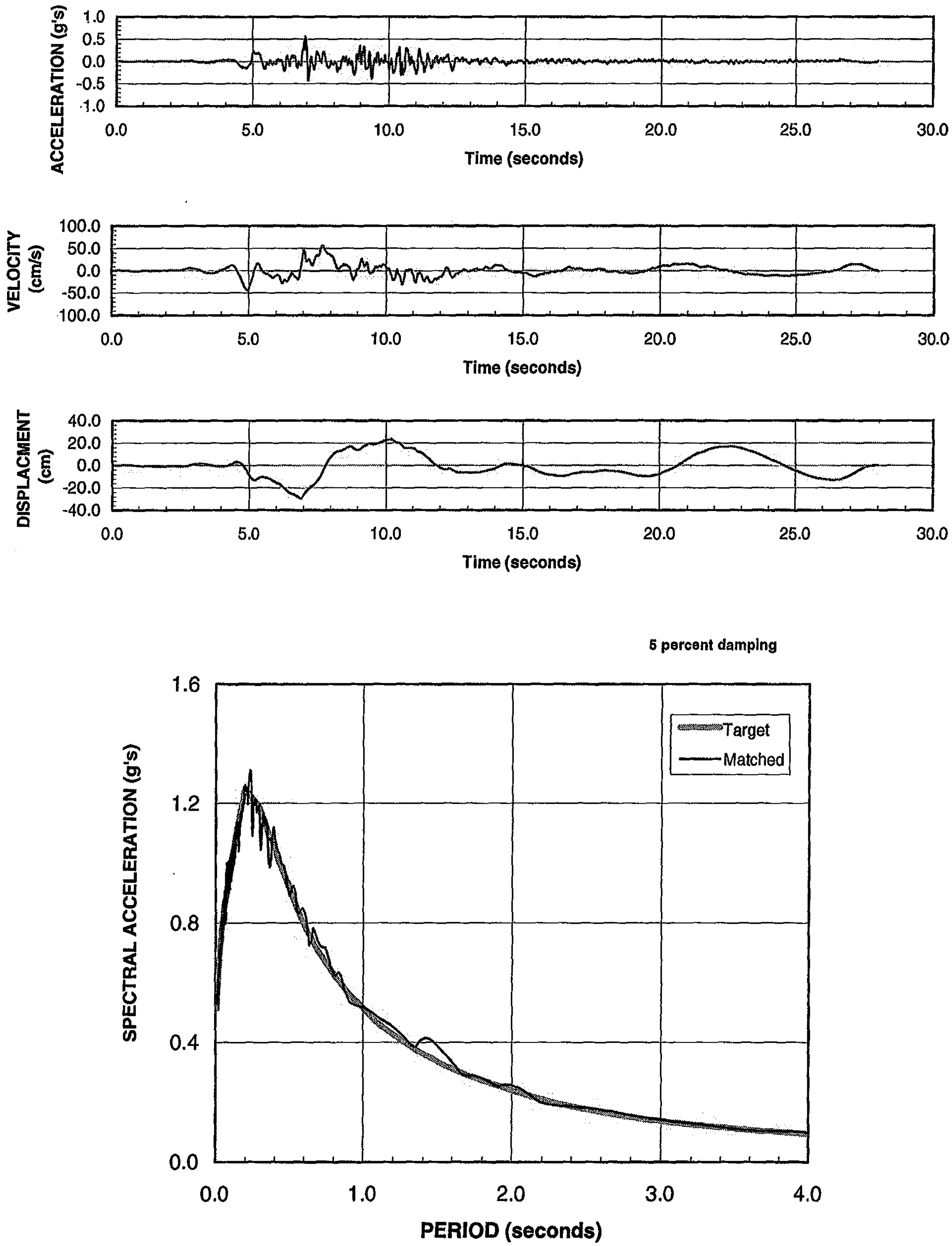
PSHA ROCK SPECTRA, 10 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

Date 07/25/07	Project No. 4086.16	Figure E-1

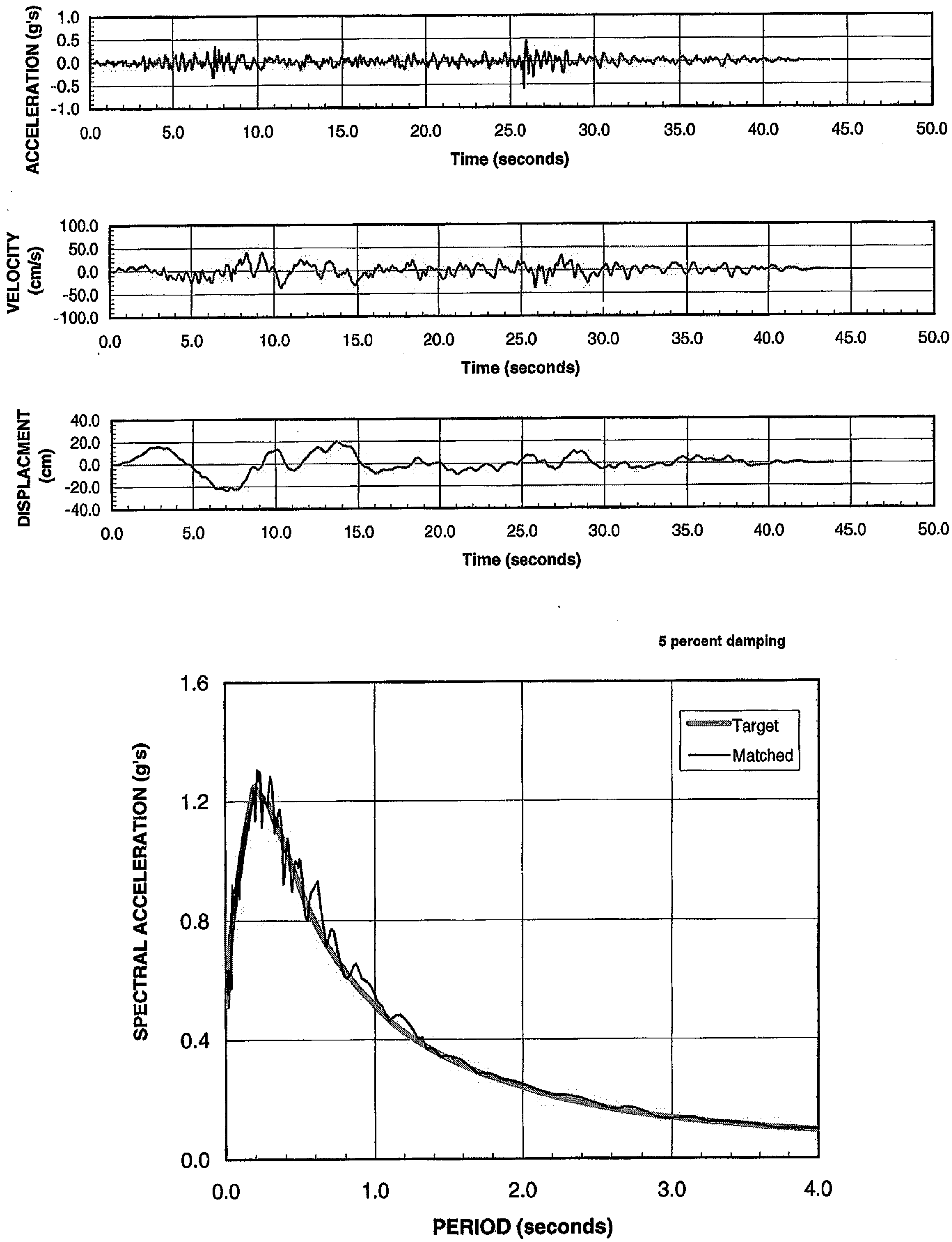
Treadwell&Rollo



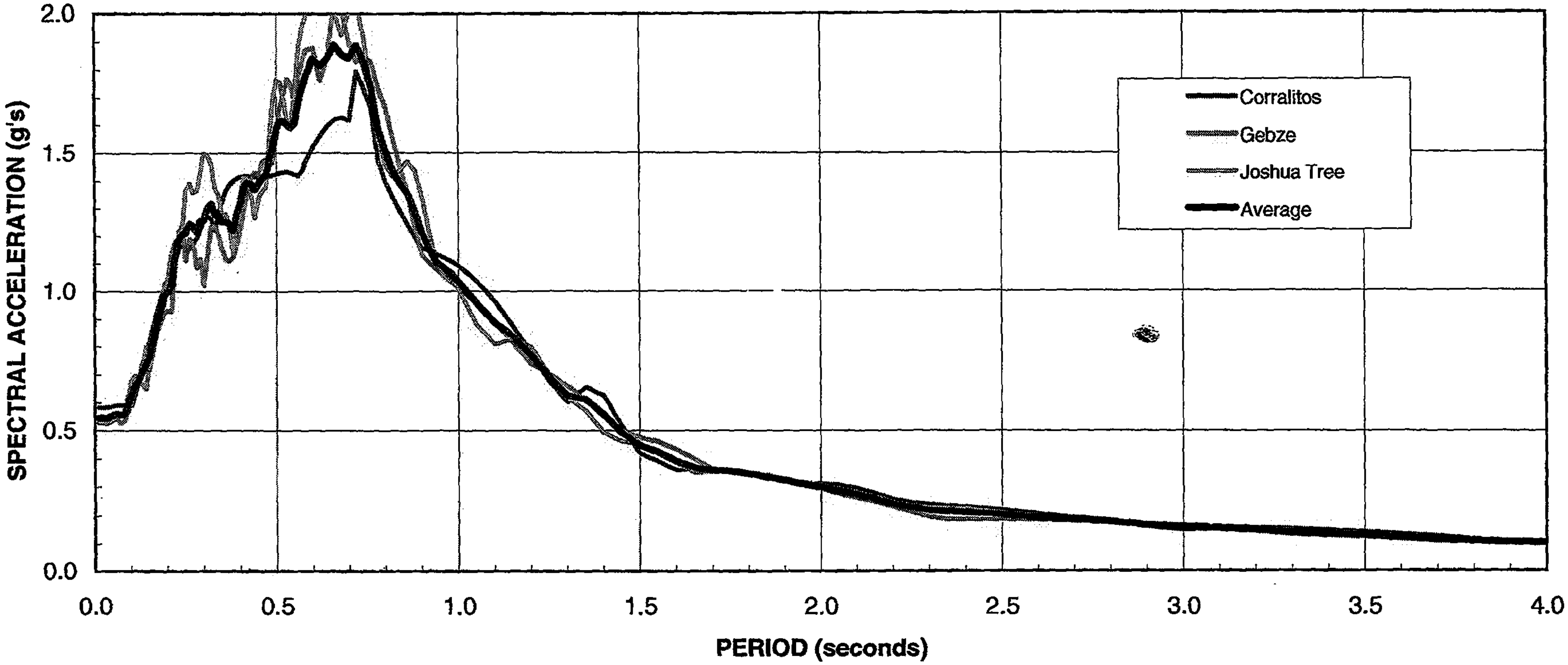
BLOCK 30 **MISSION BAY** San Francisco, California	**DBE MATCHED ROCK TIME HISTORY AND RESPONSE SPECTRUM 1989 LOMA PRIETA EARTHQUAKE CORRALITOS - 0 DEGREE**
Treadwell&Rollo	Date 07/30/07 | Project No. 4086.16 | Figure E-3



BLOCK 30 **MISSION BAY** San Francisco, California	**DBE MATCHED ROCK TIME HISTORY AND RESPONSE SPECTRUM 1999 KOCAELI EARTHQUAKE GEBZE - 0 DEGREE**
Treadwell&Rollo	Date 07/30/07 | Project No. 4086.16 | Figure E-4



BLOCK 30 **MISSION BAY** San Francisco, California	**DBE MATCHED ROCK TIME HISTORY AND RESPONSE SPECTRUM 1992 LANDERS EARTHQUAKE JOSHUA TREE - 0 DEGREE**
Treadwell&Rollo	Date 07/30/07 | Project No. 4086.16 | Figure E-5



Damping Ratio = 5%

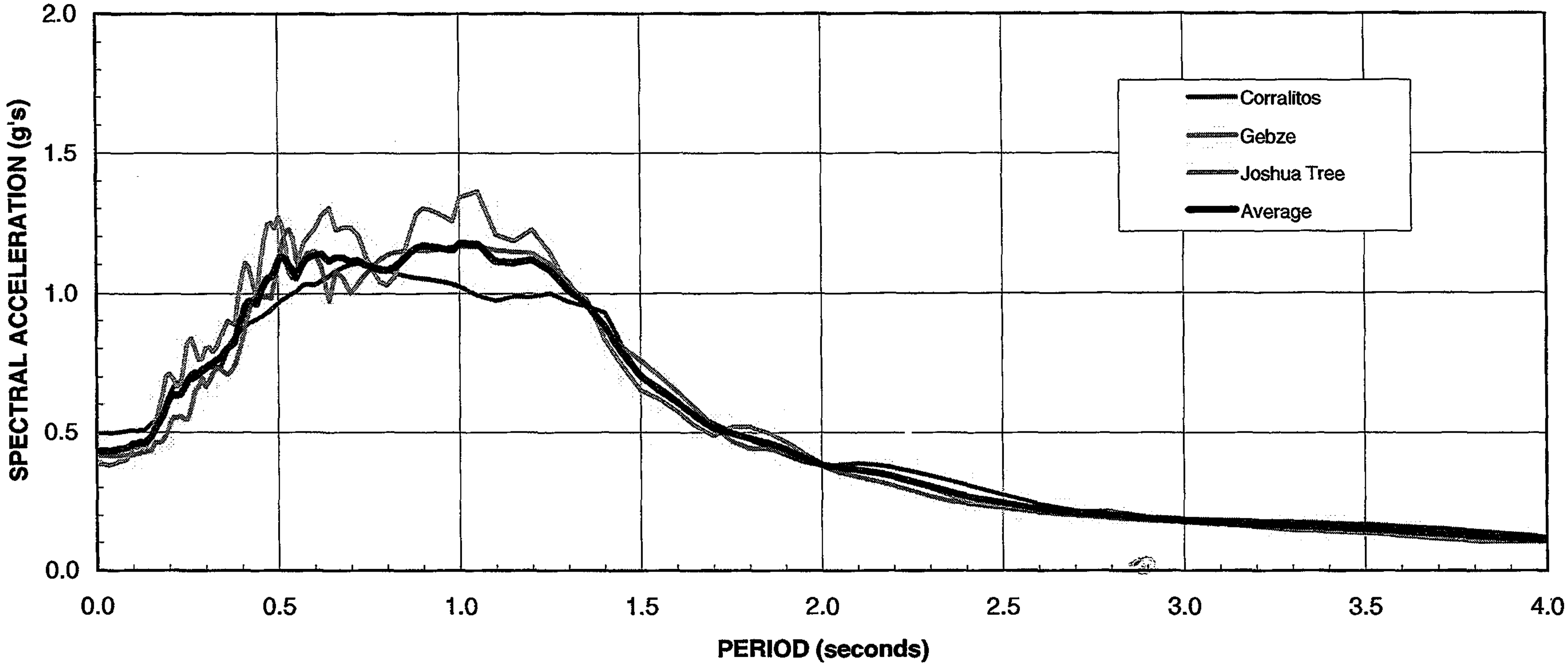
Note: DBE has a 10% probability of exceedance in 50 years

BLOCK 30
MISSION BAY
San Francisco, California

DBE SHAKE RESULTS - SHALLOW PROFILE
(non-liquefied)

Date 07/25/07	Project No. 4086.16	Figure E-6

Treadwell&Rollo



Damping Ratio = 5%

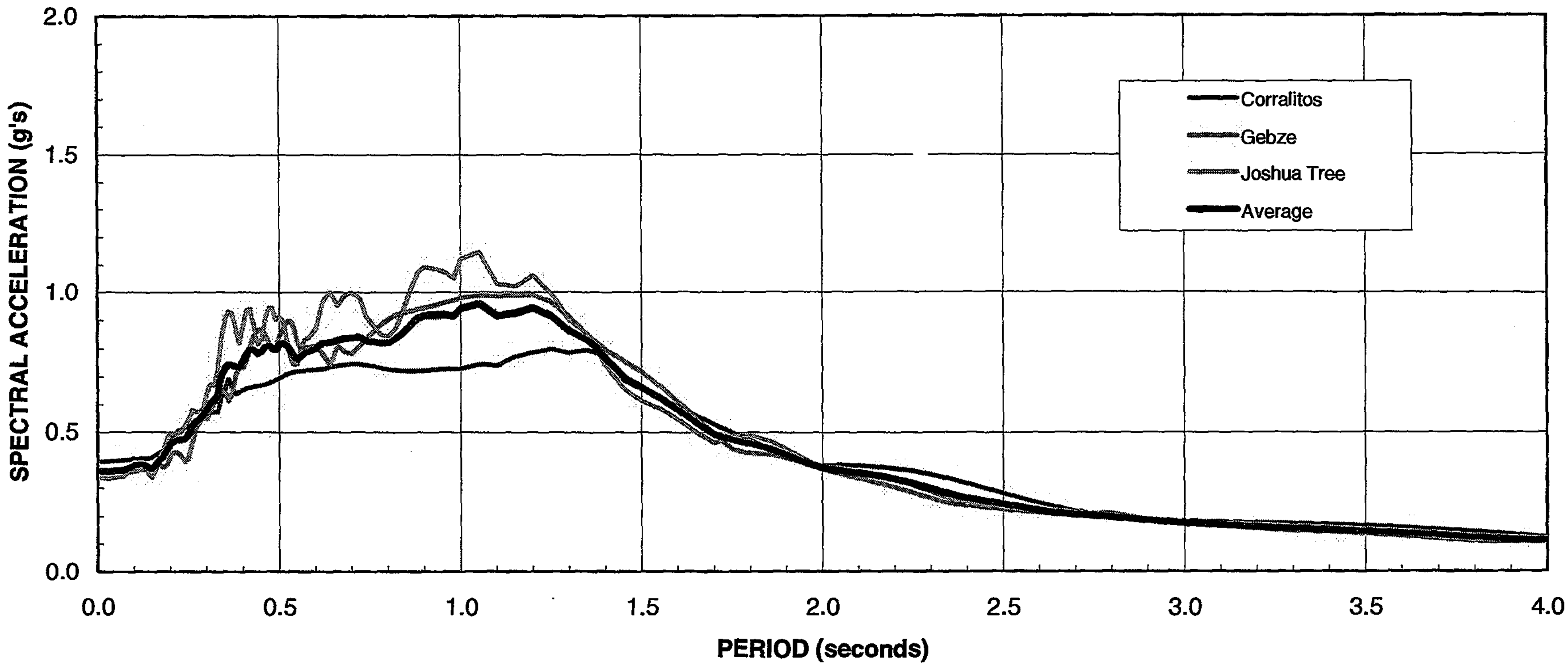
Note: DBE has a 10% probability of exceedance in 50 years

BLOCK 30
MISSION BAY
San Francisco, California

DBE SHAKE RESULTS - DEEP PROFILE (non-liquefied)

Date 07/25/07	Project No. 4086.16	Figure E-7

Treadwell&Rollo

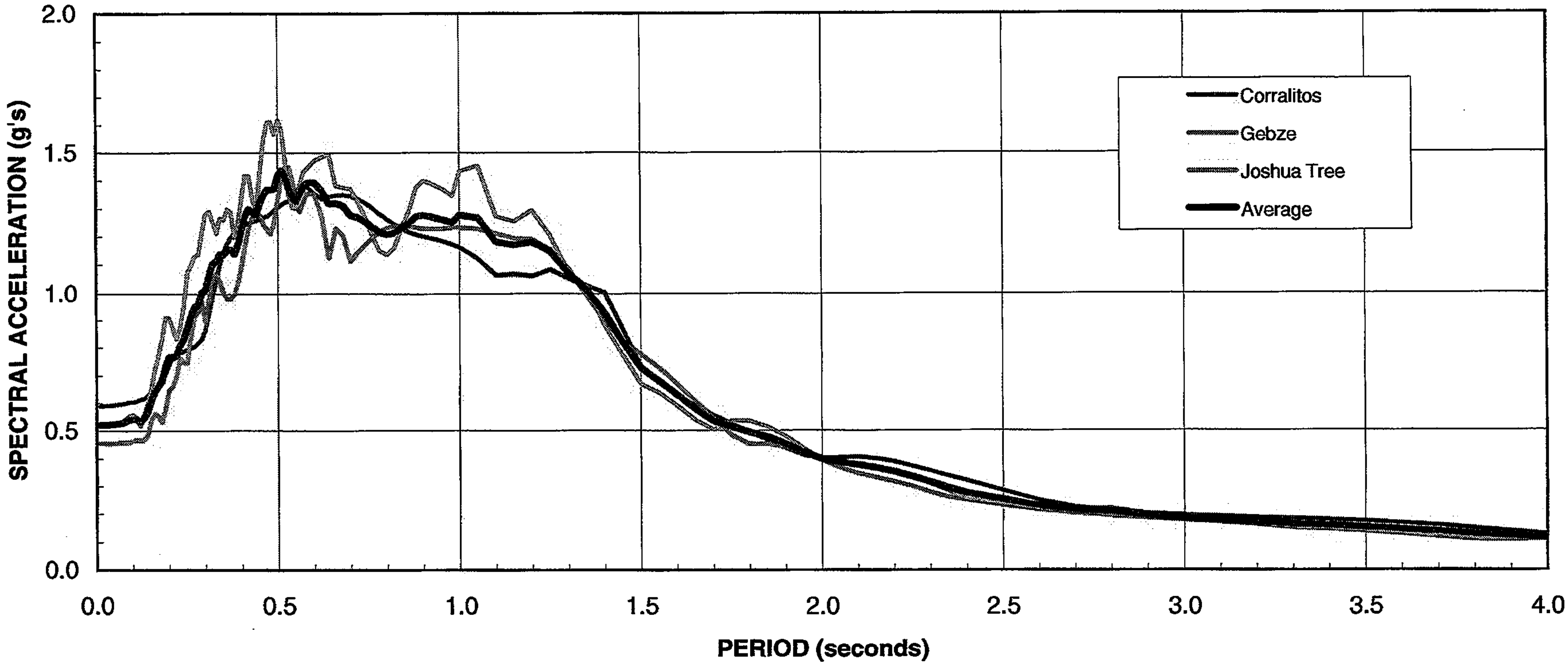


Damping Ratio = 5%

Note: DBE has a 10% probability of exceedance in 50 years

BLOCK 30 MISSION BAY San Francisco, California		
DBE SHAKE RESULTS - SHALLOW PROFILE (liquefied)		
Date 07/25/07	Project No. 4086.16	Figure E-8

Treadwell&Rollo



Damping Ratio = 5%

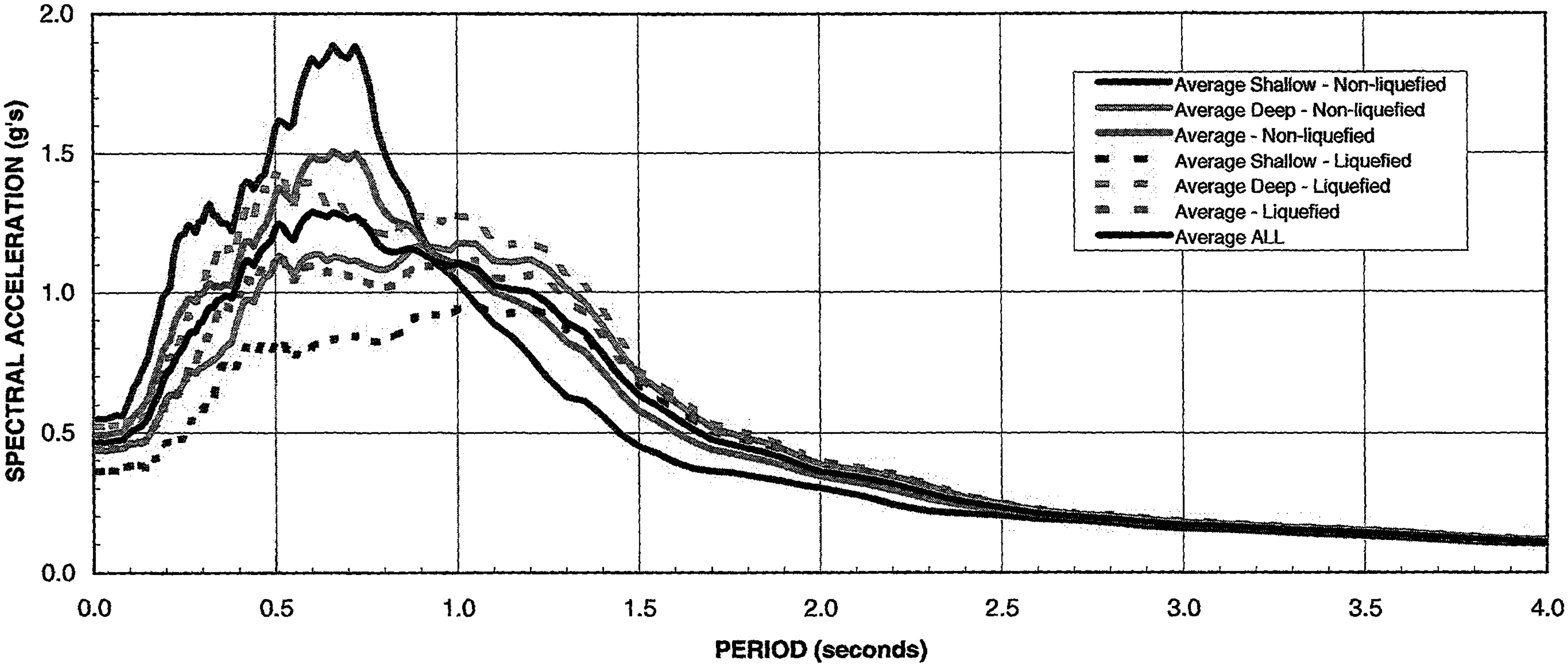
Note: DBE has a 10% probability of exceedance in 50 years

BLOCK 30
MISSION BAY
San Francisco, California

DBE SHAKE RESULTS - DEEP PROFILE (liquefied)

Date 07/25/07	Project No. 4086.16	Figure E-9

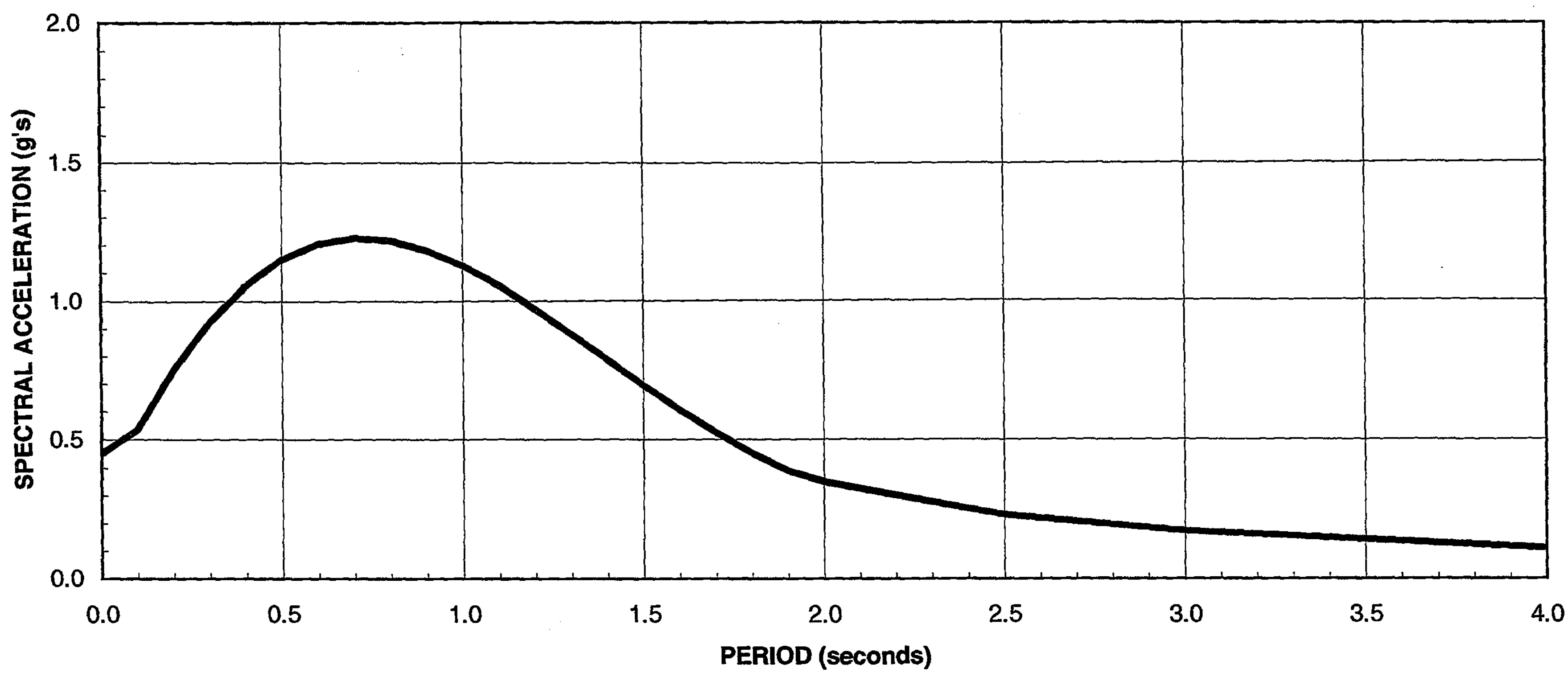
Treadwell&Rollo



Damping Ratio = 5%

Note: DBE has a 10% probability of exceedance in 50 years

BLOCK 30 **MISSION BAY** San Francisco, California		
AVERAGE DBE SHAKE RESULTS		
Date 07/25/07	Project No. 4086.16	Figure E-10
Treadwell&Rollo		



Damping Ratio = 5%

Note: DBE has a 10% probability of exceedance in 50 years.

BLOCK 30
MISSION BAY
San Francisco, California

RECOMMENDED DBE SPECTRUM

Date 07/25/07	Project No. 4086.16	Figure E-11

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