# REVISED GEOTECHNICAL INVESTIGATION
# 301 MISSION STREET
## San Francisco, California

**Millennium Partners**
**San Francisco, California**

**13 January 2005**
**Project No. 3157.02**

**Treadwell&Rollo**
**Environmental and Geotechnical Consultants**

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Mr. Steve Patterson
Millennium Partners
735 Market Street, 3rd Floor
San Francisco, California 94102

Subject: Revised Geotechnical Investigation
301 Mission Street
San Francisco, California

Dear Mr. Patterson:

Treadwell & Rollo, Inc. is pleased to present this geotechnical investigation report for the proposed 301 Mission Street project in San Francisco. This report presents our revised findings, conclusions and recommendations for the project site and replaces our previous geotechnical report dated 14 August 2000 and the two supplemental reports dated 2 July 2004 and 1 September 2004. Additional copies have been distributed as indicated at the end of this report. This letter omits detailed findings, conclusions and recommendations; therefore, anyone relying on the report should read it in its entirety.

Subsurface conditions at the site consist of heterogeneous fill over Marine Deposits underlain by clayey sand with interbedded layers of sandy clay, and Old Bay Clay to the maximum explored depth of about 220 feet below the existing ground surface. The proposed development will consist of a 60-story tower comprised of residential and retail space, a nine-story structure with residential and retail space, and a three-story-high atrium and lobby. The tower portion of the site will have one basement level, while the nine-story building and atrium will have five levels of underground parking. We recommend the tower structure be supported on a pile foundation system with the other portions on a mat foundation, as discussed in the following report.

The recommendations contained in this report are based on a limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. We should be retained to observe site excavation and shoring, compaction of backfill, and installation of pile foundations, during which time we may make any changes to our recommendations, if necessary.

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

Sincerely yours,
TREADWELL & ROLLO, INC.

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# GEOTECHNICAL INVESTIGATION
# 301 MISSION STREET
# San Francisco, California

## 1.0 INTRODUCTION

This report presents the results of our geotechnical investigation and revised recommendations for the proposed development at 301 Mission Street in San Francisco, California. The project site occupies a portion of Assessor's Block No. 3719 and is bound by Mission Street to the north[1], the Transbay Bus Terminal to the south, Fremont Street to the west, and Beale Street to the east as shown on the Site Location Map, Figure 1. Presently, the project site is comprised of four addresses: 129 Fremont Street, 124 Beale Street, 301 and 345 Mission Street, as shown on the Site Plan, Figure 2.

Treadwell & Rollo, Inc. performed a geotechnical investigation for the proposed project as was planned in 2001 and presented our conclusions and recommendations in a report dated 14 August 2001. Subsequently, we issued two design memoranda dated 11 December 2002 and 16 October 2003 and two supplemental reports dated 2 July 2004 and 1 September 2004 which addressed changes in the planned project. The 14 August 2001 report included design parameters for a 52-story tower, an adjacent 12-story structure, and interconnecting 5-story atrium with the entire project site underlain by three levels of underground parking. The 2 July 2004 letter contained supplemental recommendations for a 60-story tower with an adjacent 9-story structure, connected by a 2-story atrium underlain by four to six basement levels. The 1 September 2004, included the results of additional geotechnical field work and refined the recommendations given in the 2 July 2004 letter for four basement levels.

This report supersedes the previous two memoranda and three reports and provides our conclusions and recommendations for the project as currently planned, which includes the 60-story tower over one basement level adjacent to a 3-story atrium connected to a 9-story

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[1] Assumed project north is along Fremont Street, toward Mission Street.

structure. The atrium and the connecting 9-story structure will be constructed over five basement levels, collectively called the podium building.

## 2.0 PROJECT DESCRIPTION

Plans by Gary Edward Handel + Associates, the project architect, show the proposed development consists of a 60-story residential tower, a 9-story structure for retail and living space, and a 3-story-high atrium and lobby which connects the two structures and will contain amenities for the residents, such as a health club and pool. One basement level is planned below the tower and five levels of underground parking are planned under the 9-story structure and atrium. The excavation for the tower (including foundation) will extend about 25 feet below existing ground surface. The excavation for the 5 basements levels and foundation will extend about 60 feet below the ground surface. Therefore, on the basis of the available topographic information, which shows that the average surrounding grade at approximately Elevation 4 feet[2], we estimate the finished floor of the lowest level of the parking garage will be at about Elevation -52 feet, while the top of the basement slab below the tower will be about Elevation -11 feet. The footprints of the proposed buildings and the two excavations are shown on Figure 3.

## 3.0 SCOPE OF SERVICES

A detailed geotechnical investigation was performed; the results of which are included herein. To supplement existing subsurface information, seven borings were drilled during two separate field investigations in June of 2001 and May 2004. Soil cuttings generated during drilling were either spread on-site or stored on-site in 55-gallon drums, tested for environmental contamination and appropriately disposed of off-site.

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[2] All elevations referenced in this report are based on the San Francisco City datum (SFCD). Elevations used in this report are interpolated from spot elevations provided on an ALTA Survey prepared by Martin M. Ron Associates, Inc., for a portion of Assessor's Block No. 3719, dated 11 June 2001.

Selected soil samples recovered from the borings were tested to measure moisture content, dry density, gradation, Atterberg Limits, consolidation, and shear strength. Using the results of our field exploration, laboratory testing, and engineering analysis, we developed geotechnical conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including evaluation of liquefaction potential and associated ground deformation
- appropriate foundation type(s)
- design criteria for the recommended foundation type(s)
- estimates of foundation settlement
- site grading and excavation, including criteria for fill quality and compaction
- lateral earth pressures for design of below-grade walls
- shoring
- dewatering
- site-specific response spectrum
- 2001 San Francisco Building Code near-source and site factors
- construction considerations

## 4.0 FIELD INVESTIGATION

Prior to performing the field investigation, we reviewed available subsurface information from previous geotechnical investigations performed in the site vicinity, which are listed in the references section of this report.

### 4.1 Borings Performed for the Geotechnical Investigation

To evaluate subsurface conditions beneath the site, we performed two separate field investigations. In June of 2001, we drilled five exploratory borings (designated as B-1 through

B-5). In May of 2004, we drilled two additional borings (designated as B-6 and B-7). The approximate locations of these borings are shown on Figure 2. Because of the presence of existing buildings at the site, and underground utility and overhead obstructions on the adjacent streets, geotechnical borings were drilled within the vacant lot only (see Section 6.1). Prior to commencing drilling, we obtained a soil boring permit from the Monitoring Wells Section of the San Francisco Department of Public Health (SFDPH), and notified Underground Service Alert (USA).

The borings were drilled to depths ranging from 60.5 to 220 feet below the existing ground surface. Drilling was performed by Pitcher Drilling Company of Palo Alto, California, using truck-mounted rotary wash drilling equipment, under the direction of our field engineer.

During drilling, our engineer logged the borings and obtained representative samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A on Figures A-1 through A-8. The material encountered was classified according to the soil classification system described on Figure A-9.

Soil samples were obtained using the following sampler types:

- Standard Penetration Test (SPT) sampler with a 2.0-inch-outside diameter and a 1.5-inch-inside diameter, without liners
- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch-outside diameter, 2.5-inch-inside diameter, lined with brass tubes with an inside diameter of 2.43 inches
- Osterberg (O) piston sampler using 3.0-inch outside diameter, thin-walled Shelby tubes
- Thin-walled Shelby Tubes (ST) with 3.0-inch-outside diameter

The SPT and S&H samplers were driven with a 140-pound, above-ground, safety hammer falling 30 inches. The blow counts required to drive the S&H sampler the final 12 inches of an 18-inch drive (N-values) were converted to approximate SPT N-values using a conversion factor of 0.6

and are shown on the boring logs. Where the SPT sampler was used, the actual blow counts are shown on the boring logs. The Osterberg sampler and Shelby Tubes were advanced into the soil using hydraulic pressure. The hydraulic pressure required to advance the Osterberg sampler and Shelby Tubes is shown on the boring logs.

After completion, the borings were backfilled with cement-bentonite grout under the observation of a San Francisco Department of Public Health inspector.

### 4.2 Borings Performed for the Environmental Investigation

On 5 July 2001, Treadwell & Rollo, Inc. performed six shallow borings at the site as part of the environmental investigation. The borings, designated as TR-1 through TR-6, were hand-augered inside existing buildings to depths ranging from 3.5 to 8 feet below existing basement or ground floor slabs at the approximate locations shown on Figure 2. The logs of the borings performed as part of our environmental investigation are presented on Figures B-1 through B-6 in Appendix B.

### 4.3 Borings Performed by Dames & Moore

Two borings (DM-1 and DM-3) performed by Dames & Moore for previous investigations in the vicinity of the site were also used in our evaluations. See Figure 2 for the approximate locations of these borings and Appendix E for copies of the logs.

## 5.0 LABORATORY TESTING

Soil samples obtained during our field investigation were re-examined to confirm field classifications, and representative samples were selected for testing. Samples were tested to measure moisture content, dry density, gradation, Atterberg Limits, unconsolidated-undrained triaxial shear strength, and consolidation characteristics. The laboratory test results are presented on the boring logs and in Appendix C on Figures C-1 through C-15.

## 6.0 SITE AND SUBSURFACE CONDITIONS

The surface, subsurface and groundwater conditions across the site are described in the following sections.

### 6.1 Surface Conditions

The project site has plan dimensions of approximately 183.5 by 275 feet, and occupies just under 50,500 square-feet of the northern portion of Assessor's Block No. 3719 in San Francisco.

Three existing buildings and a vacant lot presently occupy the site as shown on Figure 2. The existing buildings include: 1) a 6-story concrete/brick building with one basement at 301 Mission Street, which may be timber-pile supported, 2) a 6-story concrete building with one basement at 124 Beale Street, and 3) a 2-story concrete building with no basement at 129 Fremont Street.

A structure with one basement level previously existed at 345 Mission Street, which is now the vacant lot (at the corner of Mission and Fremont Streets). The structure was demolished and the vacant lot was created by filling the basement with rubble and building demolition debris. The old basement slab and foundations are still present beneath the site. The type of foundation system the building was supported on is unknown, as foundation plans for the previous building are not available at this time. However, on the basis of our field investigation, it appears the structure was supported on shallow concrete foundations below the basement slab.

The site is relatively level with sidewalk/ground surface ranging from approximately Elevation 1.5 to 4 feet across the site.

### 6.2 Subsurface Conditions

The site is bayward of the historic 1852 San Francisco high tide line; therefore, it is within the Article 22A (Maher Ordinance) zone of San Francisco. Construction projects located within the

Maher zone that will disturb more than 50 cubic yards of soil are required, by the ordinance, to have their site history and soil quality assessed. Studies required by Article 22A were performed as part of our environmental studies and are presented in a separate report.

On the basis of our interpretation of conditions encountered in the borings, two idealized subsurface profiles have been prepared and are presented on Figures 4 and 5. The locations of the profiles are shown on Figure 2.

The borings indicate the site is blanketed by up to 23 feet of fill. The fill generally consists of very loose to loose sandy gravel and gravelly sand with large amounts of rubble, which includes concrete, wood and brick debris. An old basement slab, about five to twelve inches of concrete, was encountered approximately 11 feet below the ground surface in each of our test borings. In borings B-3 and B-5, about three feet of concrete was encountered below the old basement slab, to depths of almost 17 and 15 feet below ground surface, respectively. In borings B-6 about six feet of concrete was encountered below the old basement slab, to depths of about 17 feet below ground surface. This concrete is likely the remnants of the foundation system for the structure that previously existed at the 345 Mission Street lot.

The fill is underlain by relatively compressible Marine Deposits extending to depths ranging from 41 to 45 feet below the site grade, corresponding to Elevations ranging from -37.5 to -41.5 feet. On the basis of the subsurface data, it appears the Marine Deposits could extend down to about Elevation -45 feet along the Mission Street boundary of the site. The Marine Deposits consist primarily of very soft to medium stiff clay, clay with sand and sandy clay interbedded with very loose to medium dense sand and clayey sand. Consolidation tests performed on representative samples of the clay indicate it is overconsolidated[3].

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[3] Overconsolidated soil has experienced greater loads than the present weight of soil overburden.

Below the Marine Deposits, dense to very dense sand with varying amounts of clay and silt was encountered. The sand extended to depths ranging from 80 to 101 feet below the site grade, corresponding to Elevations ranging from -76.5 to -98 feet. Some interbedded layers of medium dense sand, also with varying amounts of clay and silt and approximately seven to twelve feet in thickness, were encountered in borings B-1, B-2, B-3 and B-4 within the dense to very dense sand layer. A five- to eleven-foot-thick layer of medium stiff to stiff sandy clay was also encountered within the dense to very dense sand layer in borings B-3, B-5, B-6, and B-7 at depths of about 60 to 70 feet. Laboratory tests on this material from other projects in the vicinity indicate it is normally consolidated[4].

The sandy soil is underlain by stiff to hard clay, sandy clay and clay with sand, locally known as Old Bay Clay, that ranges from 103.5 to 112 feet thick. The Old Bay Clay extends to a depth of about 200 feet below the site grade, corresponding to Elevation -196 feet. Consolidation tests performed indicate the soil is overconsolidated. The Old Bay Clay is underlain by very stiff to hard clay and sandy clay and very dense sand and silty sand to the maximum explored depth (approximately 220 feet).

### 6.3 Groundwater

The groundwater level in our geotechnical borings was generally obscured by the drilling fluid, and because of requirements to backfill the borings immediately after drilling, groundwater levels could not be allowed to stabilize. At borings B-1 and B-3, unstabilized groundwater levels were noted during drilling at depths of 13 and 10 feet below ground surface (corresponding to Elevations -9.5 and -6.5 feet), respectively.

The environmental borings (TR-1 through TR-6) were hand-augered, which allowed for groundwater level measurements. Groundwater was measured in the environmental borings at

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[4] Normally consolidated soil has not experienced greater loads than the present weight of soil overburden.

Elevations ranging from -9 to -11.5 feet. The approximate elevations where groundwater was encountered is noted next to the environmental boring locations shown on Figure 2.

On the basis of the available information at nearby sites, including the 199 Fremont Street site, we estimate the groundwater level at the project site is about 10 to 12 feet below the existing ground surface. We anticipate the groundwater level will vary seasonally a few feet depending on rainfall amounts and time of year. On the basis of the available groundwater information at the site vicinity we judge the high groundwater level within the project site is near Elevation -3 feet.

## 7.0 SEISMIC CONSIDERATIONS

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground rupture, liquefaction and differential compaction. Our evaluation of seismic considerations for the project site is presented in the following sections.

### 7.1 Regional Seismicity

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 6. For each of the active faults, the distance from the site and estimated maximum or mean characteristic Moment magnitude[5] [Working Group on California Earthquake Probabilities (WGCEP) (2003) and Cao et al. (2003)] are summarized in Table 1.

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

## TABLE 1

## Regional Faults and Seismicity

| Fault Segment | Approximate Distance from Site (km) | Direction from Site | Mean Characteristic/ Maximum Magnitude |
|---|---|---|---|
| San Andreas – 1906 Rupture | 13.4 | West | 7.90 |
| San Andreas – Peninsula | 13.4 | West | 7.15 |
| North Hayward | 16 | East | 6.49 |
| Hayward-Rodgers Creek | 16 | East | 7.26 |
| South Hayward | 17 | East | 6.67 |
| San Gregorio | 19 | West | 7.44 |
| Mt Diablo | 33 | East | 6.65 |
| Rodgers Creek | 33 | North | 6.98 |
| Calaveras | 34 | East | 6.93 |
| Concord/Green Valley | 37 | East | 6.71 |
| Monte Vista-Shannon | 41 | Southeast | 6.80 |
| Point Reyes | 42 | West | 6.80 |
| West Napa | 44 | Northeast | 6.50 |
| Greenville | 51 | East | 6.94 |
| Hayward – South East Extension | 57 | Southeast | 6.40 |
| Great Valley 6 | 61 | East | 6.70 |
| Great Valley 5 | 65 | East | 6.50 |
| Great Valley 4 | 72 | Northeast | 6.60 |
| San Andreas – Santa Cruz Mnts. | 77 | Southeast | 7.03 |
| Sargent | 83 | Southeast | 6.80 |
| Monterey Bay-Tularcitos | 100 | Southeast | 7.10 |

Figure 6 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 7) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_w$, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a $M_w$ of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a $M_w$ of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a $M_w$ of 6.9, approximately 95 km from the site.

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

In 2003 the Working Group on California Earthquake Probabilities (WGCEP 2003) at the U.S. Geologic Survey (USGS) predicted a 70 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 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 | 32 |
| San Andreas | 21 |
| Calaveras | 18 |
| San Gregorio | 10 |
| Concord-Green Valley | 6 |
| Greenville | 6 |
| Mount Diablo | 4 |

## 7.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[6], differential compaction[7] and ground rupture. We used the results of the test borings to evaluate the potential of liquefaction and differential compaction at the project site.

### 7.2.1 Liquefaction and Differential Compaction

The site is in an area of San Francisco that is designated as a seismic hazard area by the California Division of Mines and Geology (CDMG 2000). The primary purpose of this designation is to identify areas of potential soil liquefaction. Typically the soil layers of concern for liquefaction are uncontrolled sandy fill and loose to medium dense native sand.

We evaluated the potential of liquefaction and differential compaction at the proposed project site. Below the podium structure footprint (atrium/9-story building), the site will be excavated to a depth of about 60 feet to accommodate the basement levels. Therefore, the loose to medium dense sand encountered in our investigation will be removed within the podium footprint. Therefore, seismically-induced settlement will be negligible below the podium foundation level.

However, layers of saturated, loose to medium dense sand exist below the proposed tower basement excavation, within the Marine Deposits and below. The results of our analyses indicate these layers are susceptible to liquefaction during a moderate to large earthquake on one of the nearby faults. We estimate liquefaction-induced settlement on the order of 1 inch may

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

[7] Differential compaction is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

occur beneath the shallower tower basement. However, this settlement will not effect the tower since it will be supported on a pile foundation that extends through these layers.

Outside of the excavation, we judge that significant subsidence of streets and sidewalks could occur during an earthquake. This settlement is expected to be random and erratic, and will most likely disrupt utilities and damage sidewalks and streets.

### 7.2.2 Ground Rupture

Historically, ground surface ruptures closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is very low.

## 8.0 DISCUSSION AND CONCLUSIONS

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

- the magnitude of seismically-induced ground settlement resulting from liquefaction
- the presence of compressible Marine and Old Bay Clay Deposits below the tower footprint
- the depth of excavation for the basement levels (tower and podium excavations)
- the presence of Marine Deposits at the proposed base of the tower excavation
- the presence of groundwater at a level higher than the proposed excavation depths
- issues resulting from the difference in depth between the tower and podium excavations

These geotechnical concerns and their impact on the proposed grading, foundation design, and construction are discussed in the following sections. Discussion of environmental issues associated with excavation of the onsite fill is presented in our environmental report.

## 8.1 Foundations

### 8.1.1 Tower

We considered deep (piles) and shallow (mat) foundations for the support of the proposed tower structure. The sandy fill encountered in the upper 12 to 23 feet of the borings will be removed in its entirety during excavation for the proposed basement. However, Marine Deposits will be exposed at the base of the planned excavation and are unsuitable for support of a mat foundation. In addition, medium dense sandy layers encountered are expected to liquefy in the event of a major earthquake, as discussed in Section 7.2.1. Therefore, we judge a mat foundation would not be appropriate for the proposed 60-story tower.

On the basis of the results of our analyses and evaluation, we conclude the proposed structure should be supported on piles. Piles would derive their capacity from a combination of skin friction in the medium dense to very dense sand and medium stiff to stiff clay, and end bearing in the dense to very dense sand. From our experience with similar projects, we conclude precast, prestressed concrete piles or an auger displacement pile system (details are described in Section 9.2) are the most appropriate pile types for the project. We understand on the order of about 1,000 piles will be required to support the tower. Although piles will transfer building loads to less compressible strata, some settlement of the pile foundations will still occur. The settlement of the large group of piles will be due to the consolidation settlement of the underlying overconsolidated Old Bay Clay. We estimate settlements on the order of four to six inches could occur under the tower.

### 8.1.2 Podium Structure (Atrium/9-Story Building)

The podium structure will include a five level of underground portion which will require an excavation on the order of about 60 feet deep. The excavation will remove the fill and the marine deposits in their entirety. The subgrade will mostly consist of the dense to very dense sand with possible zones of sandy clay. On the basis of the subsurface conditions we recommend the podium structure be supported on a reinforced mat provided the calculated settlements are acceptable. The estimated settlements range from about 1 to 3 inches. The estimated settlement under the 9-story building is about 1 to 1.5 inches. These settlements were calculated using the foundation pressures provided by DeSimone Consulting Engineers (DCE) dated 17 June 2004. The largest settlements would occur near the boundary of the podium and adjacent tower. These are due to the effect of the tower loads and their shadowing effect on the adjacent structure.

## 8.2 Construction Considerations

The main construction considerations are shoring requirements and dewatering for the basement excavations. Additional concerns are the need for predrilling to facilitate pile installation, the presence of concrete rubble and debris in the near-surface fill, and the Marine Deposits that will be exposed at the bottom of the basement excavation. These issues are discussed in the following sections.

### 8.2.1 Shoring

#### 8.2.1.1 Tower

We understand the finished floor for the tower basement will be about 15 feet below existing ground surface. Currently, a 10-foot thick pile supported mat is being considered for the tower. This will require an excavation of about 25 feet. Because there is insufficient space to slope the sides of the excavation, shoring will be required. Several methods of shoring are available, and the system selected should take into account the requirements for protecting adjacent property as well as cost. We have qualitatively evaluated the following systems:

- soil nailing
- sheet piles
- conventional soldier pile and lagging
- soldier pile tremie concrete (SPTC) or mixed-in-place soil/cement walls

Soil nailing is a method of shoring using grouted reinforcing bars (nails), which are typically spaced, horizontally and vertically, between 4 and 6 feet. Considering the excavation will be performed primarily in sandy soil and there is a high groundwater level at the site, we do not recommend soil nailing for this project.

Sheet piles with internal bracing may be appropriate but it would likely be difficult to drive the sheet piles through the fill due to the presence of concrete and brick debris.

We conclude soldier pile and lagging is a feasible shoring system. However, it would require extensive dewatering which may be cost-prohibitive. Additionally, it would be difficult to install lagging in areas where perched water is encountered. Perched water can transport soil through the lagging resulting in the creation of voids behind the lagging.

Soldier pile tremie concrete (SPTC) or mixed-in-place soil/cement walls would likely be the most watertight shoring systems and thus require the least dewatering. In addition, SPTC or mixed-in-place soil/cement walls would be relatively rigid and could significantly limit lateral deflections and ground subsidence related to the excavation. The disadvantages of these systems are cost and space requirements. Installation for these systems will require a width of about three feet around the perimeter of the site.

Lateral resistance against movement may be mobilized by extending the shoring below the bottom of the excavation and using internal braces or tiebacks. Tiebacks will have relatively low capacities in the fill and Marine Deposits that extend to approximately Elevation -41 feet. Because the depth of excavation (25 feet) is relatively shallow, tiebacks with low capacities may still be feasible. However, the use of tiebacks as lateral support for the tower excavation will be limited to the Mission and Fremont Streets sides because an excavation is planned for the podium along the east side and the Caltrans Transbay Terminal facility is on the south side. Our experience leads us to believe that Caltrans will not allow installation of tiebacks below the pile supported Transbay Terminal facility. Therefore internal bracing should be anticipated along the east and south sides and can be either cross-lot or inclined rakers.

We conclude that the SPTC and soil/cement walls are the best options to shore the tower excavation. The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. However, the shoring should be designed by a structural engineer knowledgeable in this type of construction, and we should review the design to confirm it incorporates our concerns regarding the shoring.

#### 8.2.1.2 Podium Structure

We understand the finished floor for the five-level basement will be about 52 feet below existing ground surface. Currently, an 8-foot thick concrete mat is planned to support the podium structure. This will require an excavation of about 60 feet to accommodate basements and mat. Because there is insufficient space to slope the sides of the deep excavations, shoring will be required.

We understand mixed-in-place soil/cement walls are being considered by the design team for shoring. This would likely be the most watertight shoring systems and thus require the least dewatering. In addition, mixed-in-place soil/cement walls would be relatively rigid and could significantly limit lateral deflections and ground subsidence related to the excavation. Considering the adjacent facilities, subsurface conditions, and the depth of excavation, we concur that this is the most appropriate shoring system. It should be noted, however, that installation of this system will require a width of about three feet around the perimeter of the site.

Lateral resistance against movement may be mobilized by extending the shoring below the bottom of the excavation and using internal braces. As discussed in the previous section, tiebacks will have low capacities in the fill and Marine Deposits that extend to approximately Elevation -40 feet and therefore impractical. Internal bracing can be either cross-lot or inclined rakers.

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

### 8.2.2 Dewatering

Current plans for the tower and the podium will result in excavations which will be below the design ground water level. The design ground water level should be taken as Elevation -3 feet. Assuming an approximate ground surface elevation of about +4 feet, the tower excavation will extend to about Elevation -21 feet (about 18 feet below design groundwater), while the excavation for the podium will extend to about Elevation -56 feet (about 53 feet below design groundwater). The groundwater level at the site should be lowered to a depth of at least three feet below the bottom of the planned maximum excavations and maintained at this level until sufficient weight and/or uplift capacity is available to resist the hydrostatic uplift forces on the bottom of the structure. The project structural engineer should evaluate when the dewatering can be stopped.

The efficiency of the dewatering system will depend to some extent on the type of shoring system used. For example, a soil/cement mix wall would likely be relatively more water-tight than a soldier pile lagging wall and thus require less dewatering. The depth of the shoring will also affect the quantity of water required to be extracted to effectively dewater the site. Relatively impervious shoring extending into the Old Bay Clay would reduce dewatering.

The selection and design of the dewatering system should be the responsibility of the contractor. The contractor will need to obtain a dewatering permit from the City and County of San Francisco for discharging water into the local municipal storm drain system. The dewatering permit requires chemical testing for characterizing the water to be discharged into the storm drain system. The results of the chemical tests performed for the environmental investigation indicate treatment will likely not be required to remove petroleum hydrocarbons prior to discharging pumped groundwater from the site to the sanitary sewer system. Prior to discharging pumped groundwater into the sanitary sewer, the City will require additional groundwater analytical testing for total oil and grease (TOG), total suspended solids (TSS) and chemical oxygen demand (COD). Currently, there is a fee for disposing of construction generated water into the City's wastewater collection system. Selection of the shoring and dewatering systems should be coordinated to minimize overall costs.

Variables which significantly influence the performance of the dewatering system and the quantity of water produced include the number, depth, and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. Different combinations of these variables can be used to dewater the site. The site dewatering should be designed and implemented by an experienced dewatering contractor. However, we should check the dewatering system proposed by the contractor prior to installation.

Excessive site dewatering could result in subsidence of the immediate area due to increases in effective stress in the soil. Therefore, adjacent improvements should be monitored for vertical movement, and groundwater levels outside the excavation monitored through wells while dewatering is in progress. Should excessive settlement or groundwater drawdown be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells.

### 8.2.3 Excavation Monitoring

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle and move laterally. The magnitude of shoring movements and resulting ground deformations are difficult to estimate because they depend on many factors, including the type of shoring system used and the contractor's skill in the shoring installation. We believe ground movements of a properly designed and constructed soil/cement wall shoring system should be within about one to one and a half inches. A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of adjacent structures during excavation. This monitoring system should provide timely data which can be used to modify the shoring system during construction if needed. In addition, geotechnical instrumentation including inclinometers and piezometers should be installed to monitor movement of the shoring system and the groundwater level during excavation and construction.

### 8.2.4 Pile Driving

The on-site fill includes rubble, and old slabs and foundations that may damage the piles during driving if piles are driven from the existing ground surface. In this event, pile locations should be predrilled and cased through the fill and other obstructions prior to driving the piles. Predilling will help maintain pile alignment, and reduce pile damage and heave of adjacent improvements.

In addition, predrilling may be required to ensure that the piles gain sufficient embedment into the bearing layer and are also below the bottom of the adjacent podium excavation. In addition, predrilling will decrease the amount of subgrade heave caused by the displacement of the soil during pile driving. Detailed predrilling requirements will be determined from an indicator pile program. For cost estimating purposes (drilling and disposal), assume 35 feet of predrilling will be required, measured from the bottom of the mat.

### 8.2.5 Unstable Subgrade

Saturated, soft to medium stiff clay and loose to medium dense sand may be encountered at the subgrade level of the tower and podium excavations, respectively. This soil may become unstable under the weight of the construction equipment. To provide a suitable working surface in these areas, it may be necessary to stabilize the subgrade by removing 18 to 24 inches of the soft subgrade and replacing it with a geotextile fabric and gravel fill to provide a working surface.

## 9.0 RECOMMENDATIONS

Our recommendations regarding site preparation and grading, pile design, mat design, lateral earth pressures for basement walls, seismic design and shoring design are presented in this section of the report.

### 9.1 Site Preparation and Grading

We anticipate excavation for this project can be made using conventional earth moving equipment. Old slabs and foundations (including timber piles), and other obstructions may be encountered during shoring installation and excavation within the sandy fill and Marine deposits.

Onsite sandy fill is suitable for reuse as backfill provided it is acceptable from an environmental standpoint, and meets the requirements given below for general fill. Soil below the groundwater will require drying by aeration prior to its reuse as compacted fill. All materials to be used as fill, including onsite soil, should be free of organic material, contain no rocks or lumps larger than three inches in greatest dimension, and have a low expansion potential (defined by a liquid limit of less than 40 and a plasticity index lower than 12). Fill should be placed in lifts not exceeding eight inches in loose thickness and compacted to at least 95 percent relative compaction[8]. During construction, we should check that the on-site and any proposed import material is suitable for use as fill.

In areas where wet, compressible Marine Deposits are encountered at the subgrade level, pumping or yielding may occur under the weight of construction equipment. To provide a suitable working surface, it may be necessary to stabilize the subgrade before construction can proceed. An acceptable method to stabilize the subgrade is to excavate the weak soil and place a geotextile (Mirafi 500X or equivalent); then import granular material such as baserock to provide a working surface. We estimate that about 18 to 24 inches of gravel or crushed rock will be sufficient.

## 9.2 Pile Foundations

We recommend either driven pile or auger displacement pile foundations be used to support the proposed 60-story tower. The piles will derive their support from skin friction in the medium dense to very dense sand and medium stiff to stiff clay, and end bearing in the dense to very dense sand. Compression, uplift, and lateral pile capacities for the recommended piles are presented in the following subsections.

### 9.2.1 Driven Piles

#### 9.2.1.1 Axial Pile Capacity

We recommend 14-inch-square prestressed precast concrete piles driven to acceptable end bearing in the very dense sand be used. Piles driven at least 5 to 10 feet into the dense sand and to acceptable driving resistance (established during indicator pile driving) may be designed using an allowable compressive capacity of 260 kips for 14-inch-square, prestressed, precast concrete piles (dead plus live load conditions). This capacity may be increased by one-third for total load

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

conditions. The recommended pile capacity relates only to pile support. The structural designer should check the structural capacity.

Because of the variability in the density of sand layer across the site, refined pile lengths cannot be determined prior to driving. For estimating purposes, we recommend the top of bearing contours presented on Figure 8, plus 10 feet, be used to determine pile lengths. Prior to the start of production pile driving, we recommend an indicator pile program be performed to verify the elevation of the top of the bearing layer.

For the proposed finished basement slab elevation and assuming a ten-foot-thick pile supported mat, (pile cutoff at Elevation -21 feet), we estimate lengths for end bearing piles will range from approximately 47 to 65 feet. A better estimate of pile lengths should be determined from an indicator pile program as discussed in Section 9.2.3. Piles should be spaced no closer than three pile widths center to center to avoid reductions to the axial capacities due to group effects.

Based on the available subsurface information and our experience, we expect some piles may not meet refusal. Refusal criteria will be developed following the results of the indicator pile program. Such piles may be assigned a reduced allowable capacity on the basis of the driving resistance criteria and final embedment depth. Additional or longer piles may need to be driven to meet the loading requirements as determined by the structural engineer. It may be possible to identify areas where friction piles would be required through the indicator pile driving program (discussed in Section 9.2.3).

Piles will develop resistance to temporary uplift loads through skin friction in the Marine Deposits, and medium dense to very dense sand. Pile uplift capacities may be obtained from the curve presented on Figure 9.

### 9.2.1.2 Lateral Pile Capacity

The lateral capacity of piles will depend on the amount of deflection and bending moment that can be tolerated. Lateral loads and corresponding moments have been calculated for both free-head and fixed-head conditions, with a top deflection of 1/2 inch. The resulting bending moment profiles for single piles are presented on Figure 10. The pile was analyzed under a compressive load of 260 kips and a minimum pile tip elevation of -76 feet. Figure 10 was developed for 45-foot long piles, with a cutoff Elevation at -21 feet. The geotechnical parameters used in the lateral pile capacity analyses do not include a factor of safety.

For pile groups where the center-to-center spacing is less than eight pile widths in the direction of loading, the single pile lateral capacities should be reduced. Reduction factors, corresponding to the pile width center to center spacing, are given in Table 3.

**TABLE 3**
**Pile Group Reduction Factors for Varying Pile Center to Center Spacing**

| Pile Center to Center Spacing | Reduction Factor |
|---|---|
| 3 | 0.35 |
| 4 | 0.55 |
| 5 | 0.68 |
| 6 | 0.80 |

However, the moment profile for a single pile with an unfactored load should be used to check the design of individual piles in a group. We can provide lateral load analyses for different spacing configurations when the arrangement, number, and spacing of piles have been established.

### 9.2.2 Auger Displacement Piles

#### 9.2.2.1 Axial Pile Capacity

As an alternative, auger displacement piles can be used for foundation support. This piling system minimizes concerns with pile-driving induced vibrations and noise. One type of auger displacement pile consists of a 12.75-inch diameter closed-end steel pipe pile that has a wall thickness of 3/8 inch. The bottom two feet of the pile is tapered and has drill teeth that extend to a width slightly wider than the outside diameter of the pile shaft. The hollow pipe is screwed to a pre-determined depth or until refusal is met. Once installed, the hollow pipe is filled with structural concrete. From our experience, this type of piling system is more cost-effective than the typical drilled pier option. If these piles are installed to refusal (mostly likely in the underlying very dense sand), the piles can be designed for an allowable dead plus live load of 300 kips (Factor of Safety = 2.0). This capacity may be increased by 1/3 for total loads, including wind or seismic forces. Temporary uplift capacities (tension) may be taken as frictional to a maximum of 50% of the compression load; this does not include the weight of the piles, which may be added at the discretion of the structural engineer. The structural capacity of the pile may govern the design, and it should be checked by the project structural engineer. Piles should be spaced no closer than three pile diameters center to center to avoid reductions to the axial capacities due to group effects. In addition, an indicator pile program and pile load tests should be performed to verify the lengths and the capacities stated above.

Our field engineer should be on-site during pile installation to observe the soil encountered and to verify the piles are founded in suitable material.

#### 9.2.2.1 Lateral Load Resistance

The piles should develop lateral resistance due to the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on 1) the stiffness of the pile, 2) the strength of the surrounding soil, 3) axial load on the pile, 4) the allowable deflection at the top of the pile, 5) fixity at the top of the pile (fixed or free head), 6) the allowable bending moment capacity of the pile and 7) the pile spacing of the surrounding piles. If this pile type is selected for this project, we can provide load versus deflection and bending moment profiles and present our results in a subsequent memorandum.

### 9.2.3 Indicator Pile Program

Before production concrete piles are cast or steel piles are ordered, we recommend at least 25 indicator piles be installed to observe the driving characteristics of the piles and the performance of the equipment used. Indicator piles should be installed at production pile locations selected by us and approved by the structural engineer. The indicator piles will provide blow count data or drilling data to correlate with information obtained from the test borings, to aid in evaluating predrilling requirements (for driven piles) and to be used as the basis for establishing final production pile lengths. We can provide indicator pile lengths once the indicator pile locations are selected.

We recommend indicator piles be at least 10 feet longer than the lengths of the anticipated production piles. Pile reinforcement (precast piles) for lateral loads should be extended an additional 10 feet to allow pile cutoff of 20 feet, if required.

In the event that the indicator piles are installed from current grade (surrounding street grade), the pile locations should be predrilled and cased through the rubble fill. In addition, the contractor should assume predrilling to the top of the bearing layer. Predrilling should be at least 90 percent of the pile diagonal width and not exceed the diagonal width. The effectiveness of this predrilling criteria will be evaluated as part of the indicator program. Indicator piles should be installed with the same equipment that will be used to drive production piles so that appropriate practical refusal blow count criteria can be established.

For driven piles, we recommend performing a Wave Equation Analysis of Pile (WEAP) for the proposed concrete pile-hammer combination prior to the indicator pile installation. We will use the WEAP results to evaluate the potential pile driving situation including the use of a follower, as appropriate. We also recommend attaching pile driving analyzer (PDA) transducers to four concrete indicator piles selected by us before driving the indicator piles. 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 the PDA results based on one representative blow on each of the four selected indicator piles.

For the auger displacement piles, two of the indicator piles should be tested for static load capacity in both tension and compression. The tests should be performed to twice the design loads in both the tension and compression load tests. The load tests should be in accordance with ASTM D1143 and ASTM D3689 for compression as tension testing, respectively.

### 9.2.4 Pile Installation

Determination of driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length. Special consideration should be given in selecting a hammer that can deliver enough energy to the tip of the piles to drive them efficiently without damaging them. We recommend the piles be driven with a hammer delivering at least 75,000 foot-pounds of energy per blow.

If the piles are driven from the existing ground surface, we recommend predrilling and casing through the existing fill at the pile locations to reduce pile damage and breakage and help maintain pile alignment. The pile location should be drilled or excavated with a diameter larger than the diameter of the follower for a depth extending from the pile-driving grade to the pile cutoff elevation. Any rubble encountered during excavation of pile caps and grade beams should be removed. Furthermore, because of the large number of piles planned for the project, ground and pile heave will be an issue. To reduce this effect, we recommend predrilling should extend to at least the top of the bearing layer. Production predrilling requirements will be developed following the indicator program.

### 9.2.5 Vibration Monitoring

If driven piles are used, the existing improvements adjacent to the site, specifically the Transbay Terminal, should be monitored for pile driving-induced vibrations during pile installation. Survey points should be established at various locations on buildings within 50 feet of the site. To check for movements, these points should be monitored daily during indicator pile driving and weekly during production pile installation. To evaluate the effects of vibrations during driving, ground vibration monitoring should be performed on adjacent buildings during indicator pile driving and if warranted, during production pile driving. If excessive vibrations are recorded, pile driving operations should be halted and different methods of installation should be considered. Peak particle velocity at the ground surface in front of the adjacent structures should not exceed 0.1 inch per second.

## 9.3 Mat Foundation

We recommend that the podium structure be founded on a mat. The structural engineer has indicated that the bearing pressures will range from 2,000 to 6,000 pounds per square foot (psf). In localized areas (less than 10% of the mat area), bearing pressures are as high as 8,000 psf. However, the hydrostatic uplift pressure caused by the groundwater table will exceed the weight of the structure; therefore the structure will have to be held down with tiedown anchors.

For the analysis of the mat, we calculated moduli of vertical subgrade reactions ranging from about 20 to 100 kips per cubic foot (kcf) over the footprint of the building. Specific estimates of predicted settlement and associated subgrade moduli have been provided to DCE Engineers through an iterative process to develop the mat design.

Lateral forces can be resisted by a combination of passive resistance against the vertical face of the mat and basement walls, and friction along the base of the mat. Friction along the bottom of the foundation should be reduced because of the waterproofing at the base of the mat; a value of 0.2 times the dead load is recommended. To calculate the passive resistance, we recommend using the basement wall pressures given in Section 9.5. In the event the passive resistance is used to resist lateral loads, the walls should be designed for the approximate passive earth pressure.

Since it is anticipated that the weight of the building will not be sufficient to resist full hydrostatic uplift pressure, tiedown anchors will be required. Tiedown anchors should extend into the dense to very dense sand and Old Bay Clay beneath the mat and be spaced at least four shaft diameters apart. Uplift resistance will be developed in skin friction between the anchor shafts and the surrounding soil. For estimating purposes, we recommend friction values of 1,500 and 800 psf be used in the sand and Old Bay Clay layers, respectively. Higher values can be obtained depending upon the grout techniques employed by the contractor and the results of pullout tests.

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

- encapsulations used to provide an additional corrosion protection layer over the tendon or bar bond length should consist of a grout filled, corrugated plastic sheathing, or grout filled deformed steel tube; the prestressing steel can be grouted inside the encapsulation prior to inserting the anchor into the drill hole or after the anchor has been placed; centralizers or grouting techniques should provide a minimum of ½ inch of grout cover over the encapsulation

- a sheath filled with corrosion inhibiting compound or grout, or a heat shrinkable tube internally coated with a mastic compound should be used to provide corrosion protection of the unbonded length

- the trumpet should be sealed to the bearing plate and overlap the unbonded length corrosion protection by at least four inches; it should be completely filled with a corrosion inhibiting compound or grout
- all stressing anchorages permanently exposed to the atmosphere should be grout-filled; stressing anchorages encased with at least two inches of concrete do not require a cover

The tiedowns will be installed below the water table; therefore, the contractor should use smooth-cased, auger-cast system (such as a Klemm-rig) to prevent the holes from caving. If water is present in the shaft, grout should be placed using a tremie system. High strength bars or strand may be used as tensile reinforcement in the anchors. For stressing, the free length for a steel bar and for strand should be 10 and 15 feet, respectively. We recommend at least 10 percent of the anchors be performance-tested to at least 150 percent of the design load under our observation. The remainder should be proof-tested to 150 percent of the design load. The movement of each tiedown anchor should be monitored with a free-standing, tripod-mounted dial gauge during proof and performance testing. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is more than 0.04 inches, the load shall be held for an additional 50 minutes. The tiedown anchor should not move more than 0.08 inches between the 6- and 60-minute reading. In addition, total movement at the maximum test load should not exceed 80 percent of the theoretical elastic elongation of the unbonded length and the total deflection of the tiedowns should not exceed ¾ inch at the design load. Replacement anchors should be provided, as directed by the structural engineer, for anchors that fail the test. After testing, all anchors should be loaded to 10 percent of their design load (higher if specified by the structural engineer) and locked off.

### 9.4 Waterproofing

As mentioned previously, the tower and podium basements will extend below groundwater level and should therefore be appropriately waterproofed. The waterproofing should be designed by the waterproofing consultant; however, typically, waterproofing is placed directly on the soil subgrade and be covered by a mud slab (thin layer of lean concrete). The mud slab will reduce the potential for subgrade disturbance and protect the waterproofing from damage during mat construction. The mud slab should also provide a firm, smooth working surface for placement of reinforcing steel.

If it is essential to prevent moisture accumulation on the garage floor, we recommend a back-up moisture barrier be included between the structural mat and a topping slab as an additional precaution. A typical moisture barrier includes a capillary moisture break consisting of at least a six-inch-thick layer of clean, free-draining crushed rock (½- to ¾-inch gradation) overlain by a moisture-proof membrane of at least 10 mil thickness. The membrane should be covered with two inches of sand to protect it during construction and to aid in curing the concrete floor slab. Perforated pipes may be installed in the capillary break to collect any water that accumulates and direct it to a sump or other suitable outlet. Water should not be allowed to accumulate in the drain rock or sand prior to casting the slab.

### 9.5 Basement Walls

Basement walls should be waterproofed. We recommend all below-grade and retaining walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. Lateral earth pressures on basement walls will depend partially on the restraint at the top of the walls. Accordingly, walls should be designed for the pressures presented below, where H is the height of the wall in feet.

**TABLE 3**
**Lateral Earth Pressures Restrained Wall Condition**

| | Static | Seismic |
|---|---|---|
| Above the water table[9] | 60 pcf | 40 pcf + 15H psf |
| Below the water table | 90 pcf | 85 pcf + 15H psf |

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[9] Design groundwater level is Elevation -3 feet.

If surcharge loads fall above an imaginary 45-degree line (from the horizontal) projected up from the bottom of a retaining wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the added pressure on a case-by-case basis. Where truck traffic will pass within 10 feet of retaining walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls.

The 35-foot high wall that will separate the tower and podium structures should be designed to resist an additional surcharge from the tower pile foundation. This surcharge is equal to an equivalent fluid weight of 75 pcf to Elevation -40 feet increasing to 150 pcf to the bottom of the mat foundation (Elevation -56 feet).

The recommended design pressures assume the walls will be properly backdrained above Elevation -3 feet. One acceptable method for backdraining a basement wall is to place a prefabricated drainage panel against the backside of the newly cast wall. If this method of drainage is chosen, we recommend using Mirafi 6200 or equivalent. This product has a bentonite surface providing waterproofing in addition to drainage. The drainage panel should extend down to Elevation -3 feet. The drainage panel will reduce the risk of hydrostatic pressure against the upper portion of the basement wall by allowing water to drain to the groundwater level, about Elevation -3 feet. We should review the manufacturer's specifications regarding the proposed prefabricated drainage panel material to check it is appropriate for the intended use.

To protect against moisture migration, basement walls should be waterproofed and water stops should be placed at all construction joints.

Wall backfill should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

## 9.6 Seismic Design

### 9.6.1 Probabilistic Seismic Hazard Analysis

We expect the site will experience strong ground shaking during a major earthquake on any of the nearby faults. To estimate the ground shaking for the seismic design of the structures, we performed a site-specific probabilistic seismic hazard analysis (PSHA). In response to the request by the project structural engineer, and in accordance with our proposal, we developed design ground motions for a hazard level having 10 percent probability of exceedance in 50 years. This hazard level is consistent with the definitions of the Design Basis Earthquake (DBE) in the 2001 version of the San Francisco Building Code (SFBC).

We performed the PSHA using the computer code EZFRISK 6.22 (Risk Engineering 2004). This approach 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 WGCEP (1999) and CDMG (1996) data. Based on subsurface conditions, the site is categorized as stiff soil (SFBC designation $S_D$). In order to estimate site-specific spectra at the ground surface at this site we used attenuation relationships for stiff soil conditions. These relationships are primarily dependent on the magnitude of the earthquake and the distance from the site to the fault. Details of our analysis are presented in Appendix D.

The proposed tower and podium structures will both have underground portion which at foundation level will both have underground portions which at foundation level will either be about 25 feet or about 60 feet below the ground surface, respectively. It has long been recognized that spectral values show reductions with depth below the ground surface. Such effects have been supported analytically and have shown by recordings from downhole arrays and in comparisons of recordings in the free field and in adjacent structures at their basement levels. Golesorkhi and Gouchon (2000) developed recommended ratios that modify the surface spectrum to account for depth effects for different spectral periods. Furthermore, FEMA 440 Appendix B discusses effects of reduction of surface spectrum as a function of depth of embedment of the foundation. We used ratios by Golesorkhi and Gouchon (2000) to modify the surface spectra and develop the basement level spectra. We recommend the use of the basement level spectra at the foundation level for design. Table 4 presents the recommended spectra.

**TABLE 4**
**Spectral Acceleration (g) for Damping Ratio of 5 percent**
**10 percent probability of Exceedance in 50 years (DBE)**

| Period (sec) | Ground Surface | Basement |
|---|---|---|
| 0.01 | 0.495 | 0.318 |
| 0.1 | 0.842 | 0.590 |
| 0.2 | 1.132 | 0.849 |
| 0.3 | 1.179 | 0.933 |
| 0.4 | 1.153 | 0.933 |
| 0.5 | 1.108 | 0.918 |
| 0.75 | 0.953 | 0.818 |
| 1.0 | 0.811 | 0.745 |
| 2.0 | 0.473 | 0.473 |
| 3.0 | 0.290 | 0.290 |
| 4.0 | 0.199 | 0.199 |
| 5.0 | 0.160 | 0.160 |
| 6.0 | 0.133 | 0.133 |

### 9.6.2 San Francisco Building Code

For seismic design in accordance with the 2001 San Francisco Building Code, we recommend using soil profile type $S_D$. The site is about 13.4 kilometers from the San Andreas Fault, a type A fault; hence near-source factors $N_a$=1.0 and $N_v$=1.064 should be used.

## 9.7 Utilities and Utility Trenches

The design of the underground utilities should consider earthquake-induced settlement may occur in the fill surrounding the site. Flexible utility connections that can accommodate differential movement between the ground and the proposed structure should be used.

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

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in lifts of eight inches or less, moisture-conditioned to near the optimum moisture content, and compacted to at least 95 percent relative compaction.

## 9.8 Shoring

The proposed excavation will need to be shored. The shoring should be designed to limit ground deformations to less than an inch.

We recommend that a soil-cement mixed in-place wall with internal bracing be used to support the sides of the excavation. Three temporary shoring conditions will exist at the site as discussed below and depicted on Figure 2. They are:

- Case 1 – Shoring for the 60-foot deep excavation
- Case 2 – Shoring for the 25-foot deep excavation
- Case 3 – Shoring for the 35-foot high wall between the tower and podium excavations

We have developed three lateral earth pressure diagrams for the three different shoring wall conditions listed above and they are presented on Figures 11 through 13. The surcharge pressure presented on Figure 13 is based on foundation pressure from the tower constructed to 33 floors. This is based on our discussion with Webcor Builders regarding the construction schedule. According to Webcor's schedule, the permanent podium basement wall next to the tower will be constructed to the level of the tower mat foundation when the tower is constructed to the 33rd floor. The permanent basement wall will be designed to resist the surcharge of the fully constructed tower. If this sequence changes, the surcharge pressure should be re-evaluated. In addition, we understand this interior shoring wall will be constructed below the proposed eastern edge of the tower mat foundation. The top of the shoring should be separated from the bottom of the mat by a minimum of 12 inches to prevent the shoring from influencing the mat behavior.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The contractor or his designer should be responsible for determining the type and size of bracing and struts required to resist the given pressures.

Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. Internal bracing should be installed as close to the time of excavation as possible. Excavation should not proceed below a level of bracing until the all bracing at that level has been installed. Jacking (preloading) of the bracing against the sides of the excavation can reduce movement of the shoring.

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

The shoring system should be designed by a licensed engineer, experienced in the design of shoring. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements.

We recommend both Treadwell & Rollo and DCE Engineers review shoring plans. In addition, we recommend a representative from our office observe the installation of the shoring system.

### 9.9 Dewatering

The groundwater should be drawn down so that the piezometric level in the soil layers below the base of the two excavations is at least three feet below the bottom of the respective excavation. These levels should be maintained until sufficient building weight and/or uplift capacity is available to resist the hydrostatic uplift pressure of the groundwater once it is allowed to rise to its normal elevation. The structural engineer should evaluate and provide recommendations when the dewatering system can be turned off. The number and depth of dewatering wells should be determined by a specialty dewatering contractor. The volume of water discharged should be monitored and a record of the amount should be submitted to the owner.

## 9.10 Construction Monitoring

To monitor ground movements, groundwater levels, and shoring movements, we recommend installing the instrumentation listed below:

**Slope indicators**: We recommend installing at least six slope indicators. A slope indicator should be installed behind each of the exterior walls. The remaining two slope indicators should be embedded in the shoring walls along the north and south sides of the site.

**Piezometers:** One piezometer should be installed behind each exterior shoring wall. The piezometers should each have two casings, one to measure groundwater level in the sand and the other in the bedrock. The upper portions of the piezometers should be properly sealed with cement-bentonite mix to reduce surface water infiltration.

**Survey points:** Survey points should be installed on the adjacent buildings and streets that are within 100 feet of the site.

The instrumentation should be read regularly and the results should be reviewed in a timely manner. Initially, the instrumentation should be read weekly. The frequency of readings may, in the later stage of construction, be modified as appropriate. In addition, the conditions of existing buildings within 100 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction.

## 10.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Treadwell & Rollo, Inc. can provide review of the project plans and specifications as required by the City and County of San Francisco for building permit approval. This will allow us to check conformance with the intent of our recommendations.

During construction, an engineer from our office should observe installation of groundwater wells, the shoring system, indicator and production piles, placement and compaction of any backfill and the excavation for the mat foundation. These observations will allow us to compare actual with anticipated soil conditions and verify that the contractors work conforms to the geotechnical aspects of the plans and specifications.

## 11.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited subsurface 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 so that supplemental recommendations can be made.

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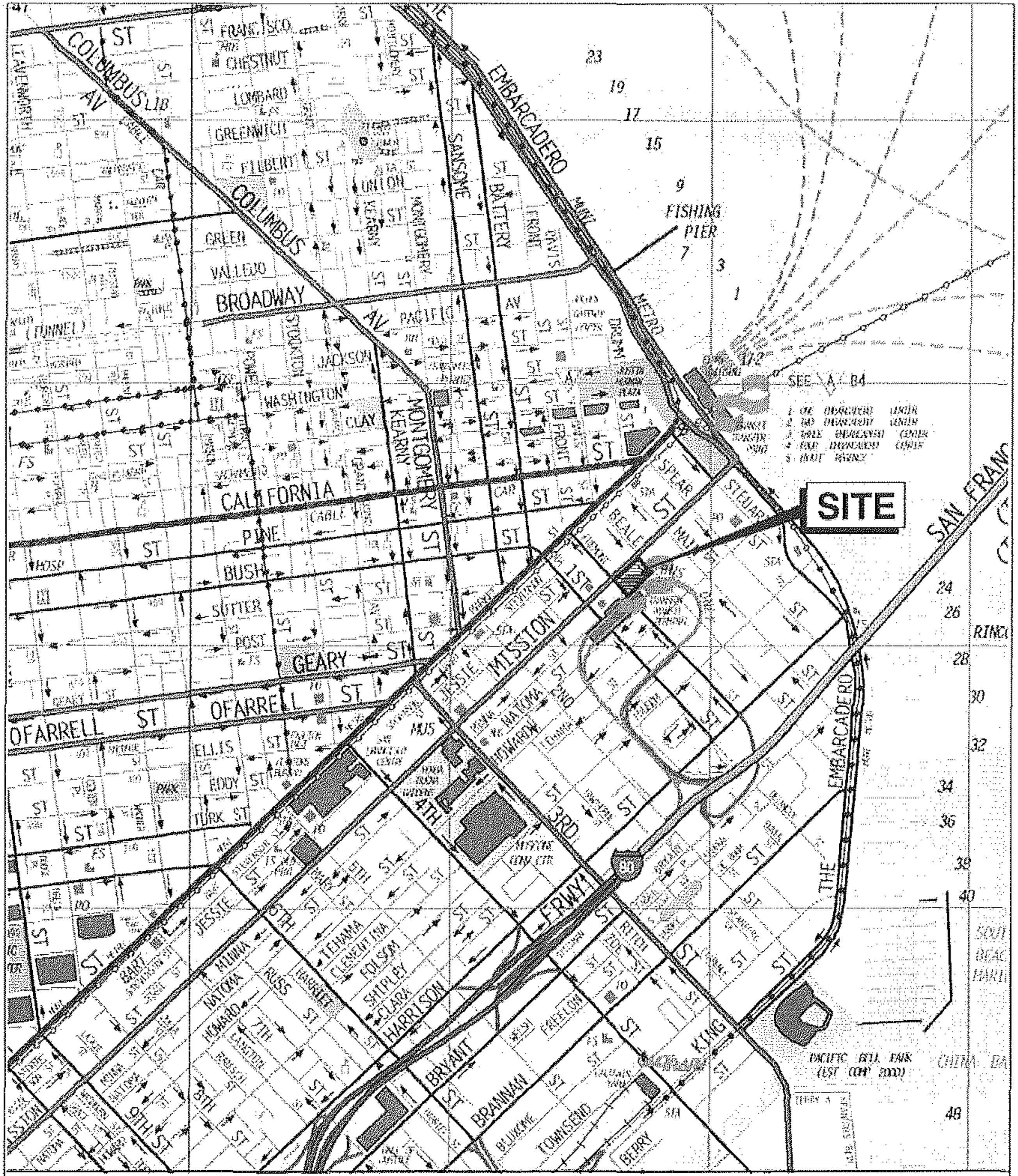
# FIGURES

# FIGURES



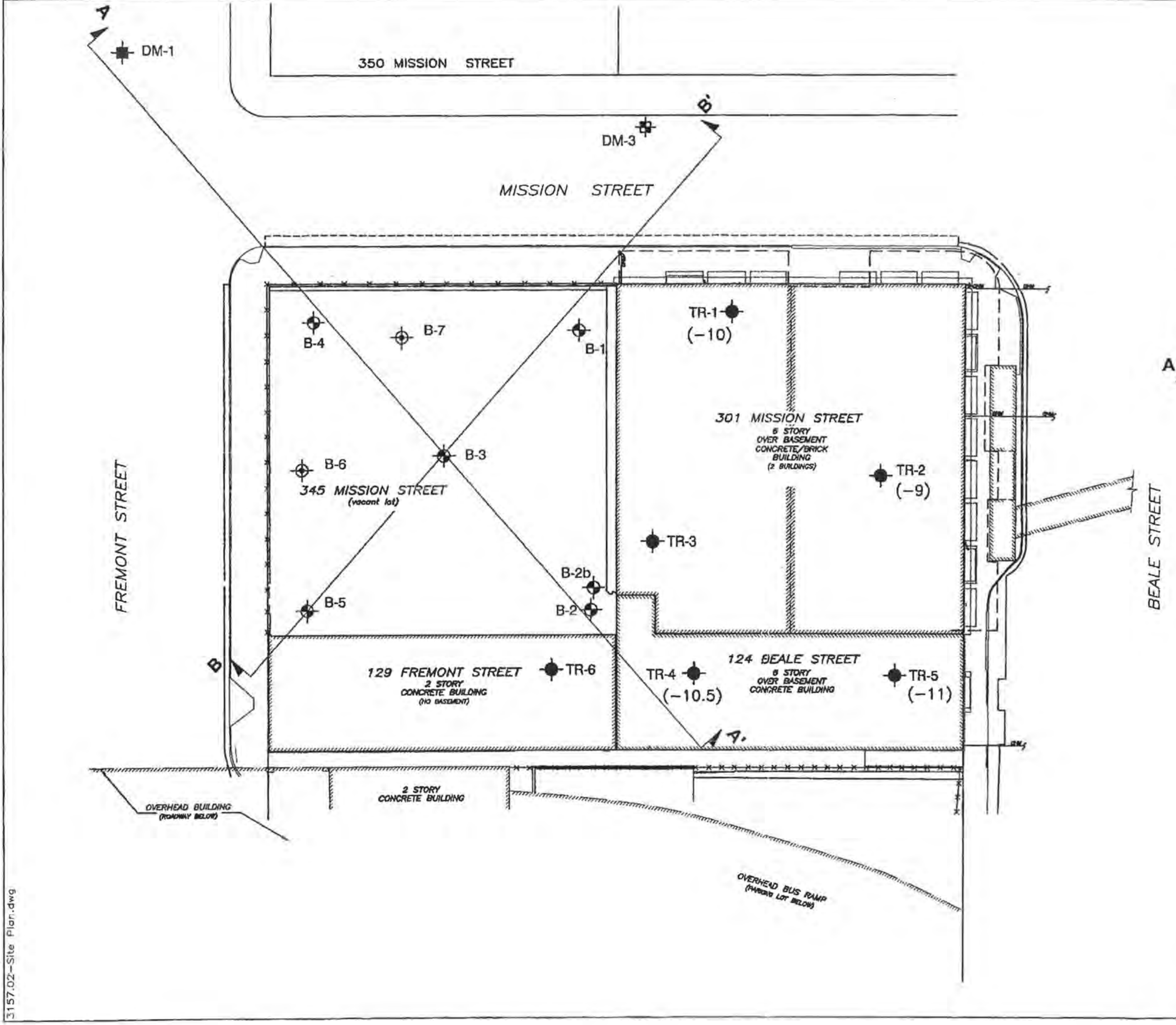
Base map: The Thomas Guide
San Francisco County
1999

**301 MISSION STREET**
San Francisco, California

**Treadwell&Rollo**

## SITE LOCATION MAP

| Date 12/29/04 | Project No. 3157.02 | Figure 1 |
|---|---|---|



EXPLANATION

| | | |
|---|---|---|
| B-1 | ⊕ | Approximate location of soil boring by Treadwell & Rollo, Inc., June to July 2001 |
| B-7 | ⊕ | Approximate location of soil boring by Treadwell & Rollo, Inc., May 2004 |
| TR-1 | ● | Approximate location of environmental soil boring by Treadwell & Rollo, Inc., 5 June 2001 |
| (-9) | | Approximate elevation at which groundwater was encountered, feet (San Francisco City datum) |
| DM-1 | ■ | Approximate location of soil boring by Dames & Moore, November 1980 |
| DM-3 | ⊞ | Approximate location of soil boring by Dames & Moore, February 1966 |
| A A' | | Approximate location of idealized subsurface profile |

0 40 feet
Approximate scale

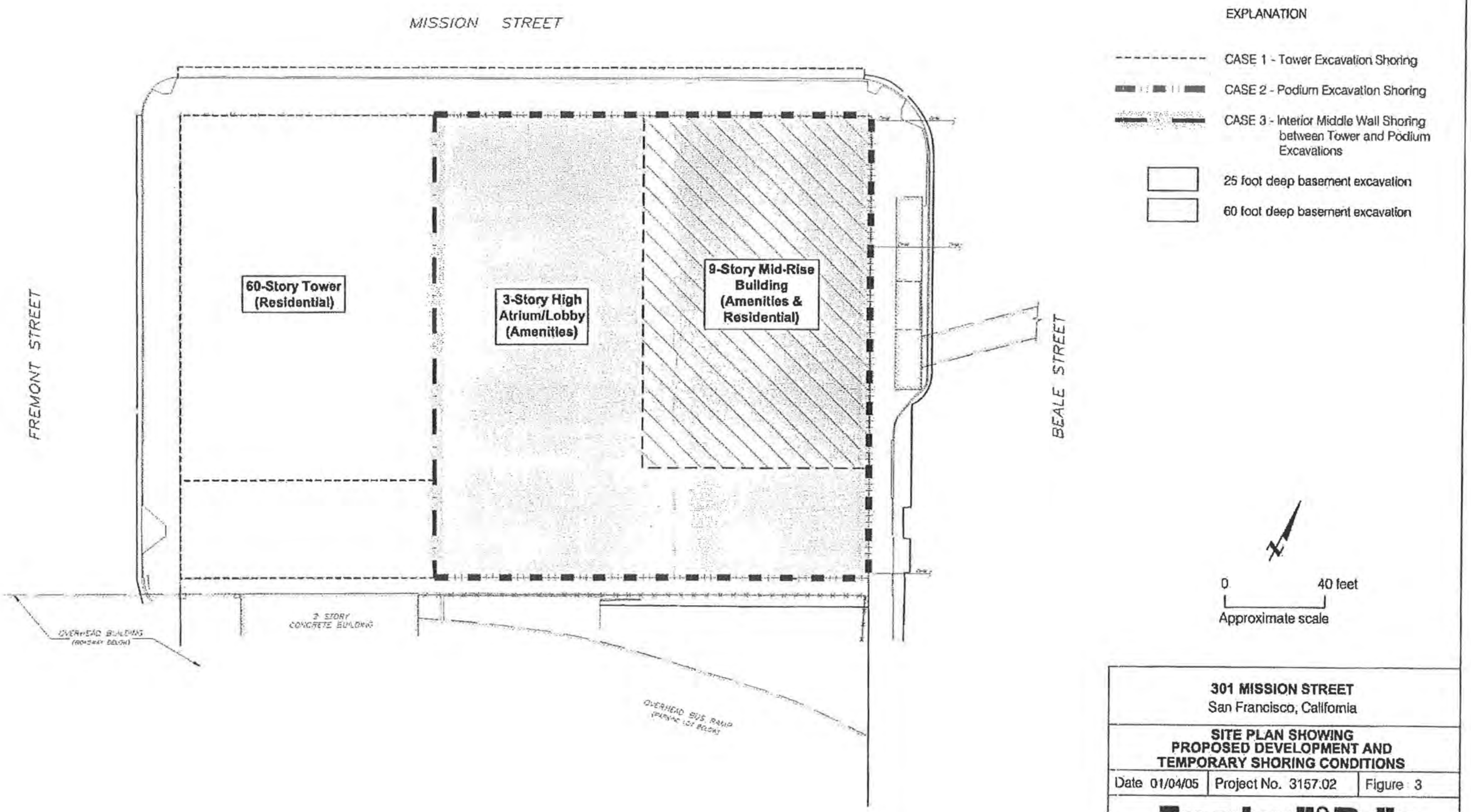
**301 MISSION STREET**
San Francisco, California

**SITE PLAN**

Date 01/11/05 | Project No. 3157.02 | Figure 2

**Treadwell&Rollo**

3157.02-Site Plan.dwg



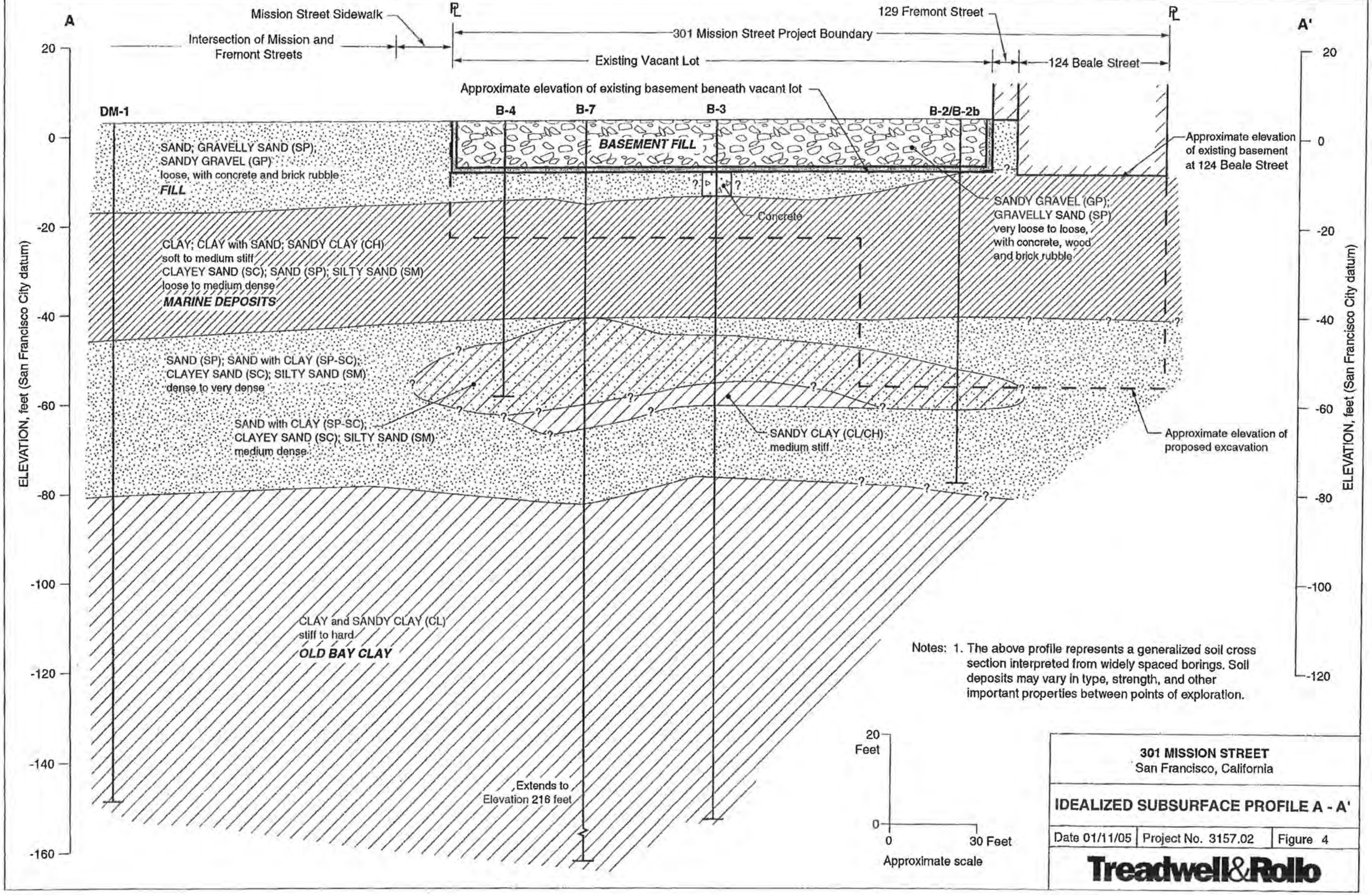
**301 MISSION STREET**
San Francisco, California

**SITE PLAN SHOWING PROPOSED DEVELOPMENT AND TEMPORARY SHORING CONDITIONS**

| | | |
|---|---|---|
| Date 01/04/05 | Project No. 3157.02 | Figure 3 |

**Treadwell&Rollo**

3157.02 PROPOSED DEVELOPMENT.DWG



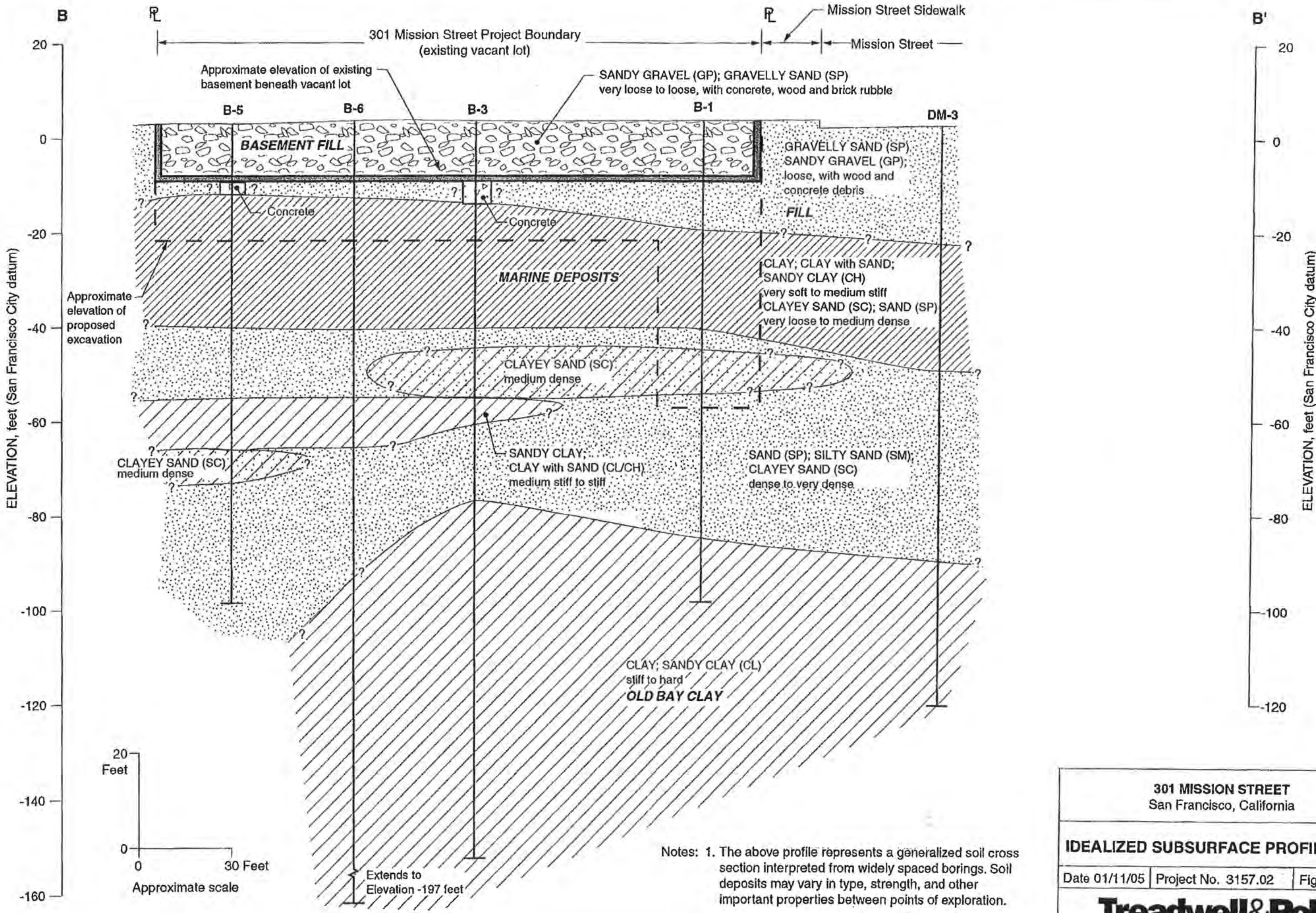
Notes: 1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

**301 MISSION STREET**
San Francisco, California

## IDEALIZED SUBSURFACE PROFILE A - A'

| | | |
|---|---|---|
| Date 01/11/05 | Project No. 3157.02 | Figure 4 |

**Treadwell&Rollo**



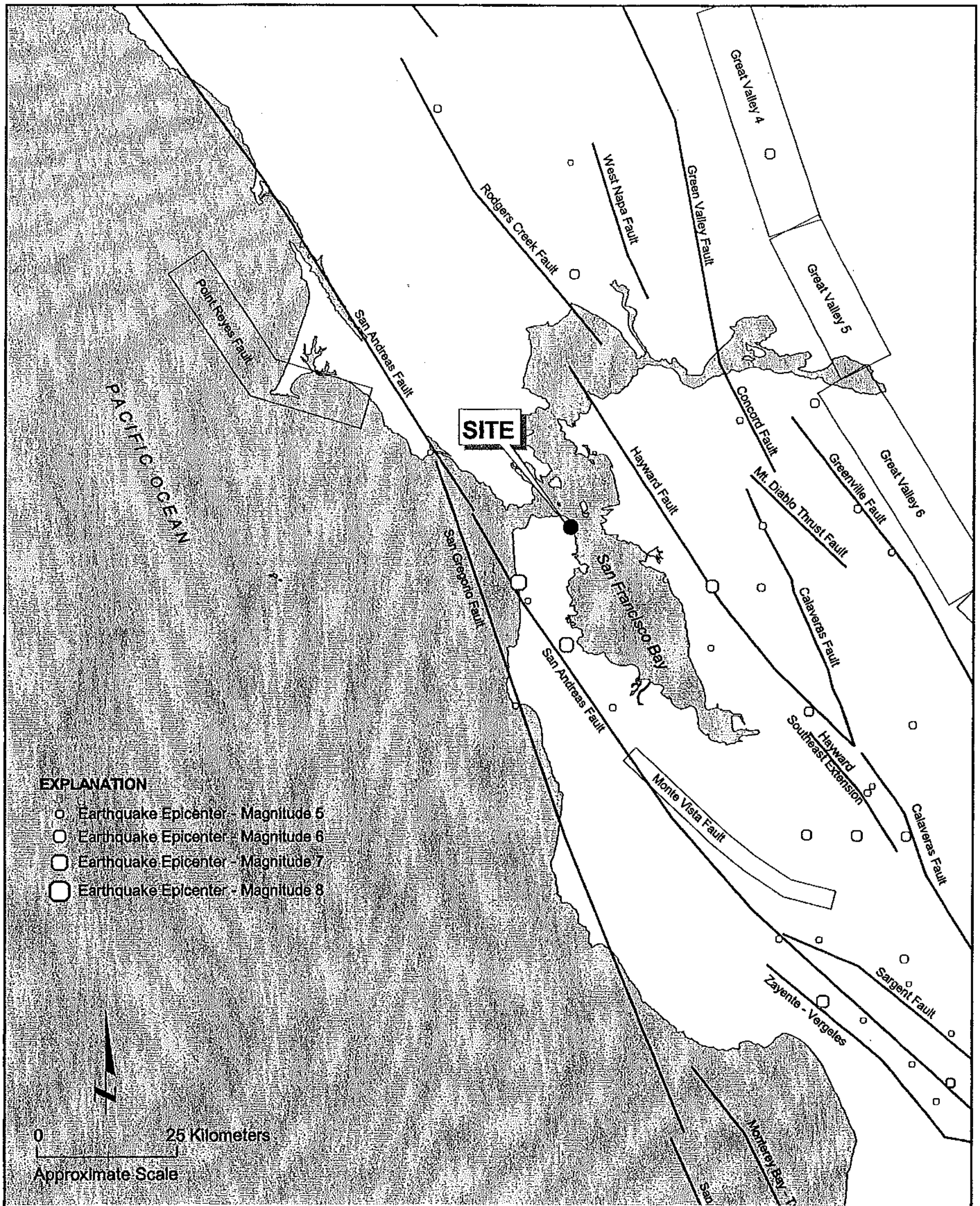
Notes: 1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

**301 MISSION STREET**
San Francisco, California

**IDEALIZED SUBSURFACE PROFILE B - B'**

Date 01/11/05 | Project No. 3157.02 | Figure 5

**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.

| | |
|---|---|
| **301 MISSION STREET**<br>San Francisco, California | **MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA** |
| **Treadwell&Rollo** | Date: 01/04/05 \| Project No. 3157.02 \| Figure: 6 |

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.

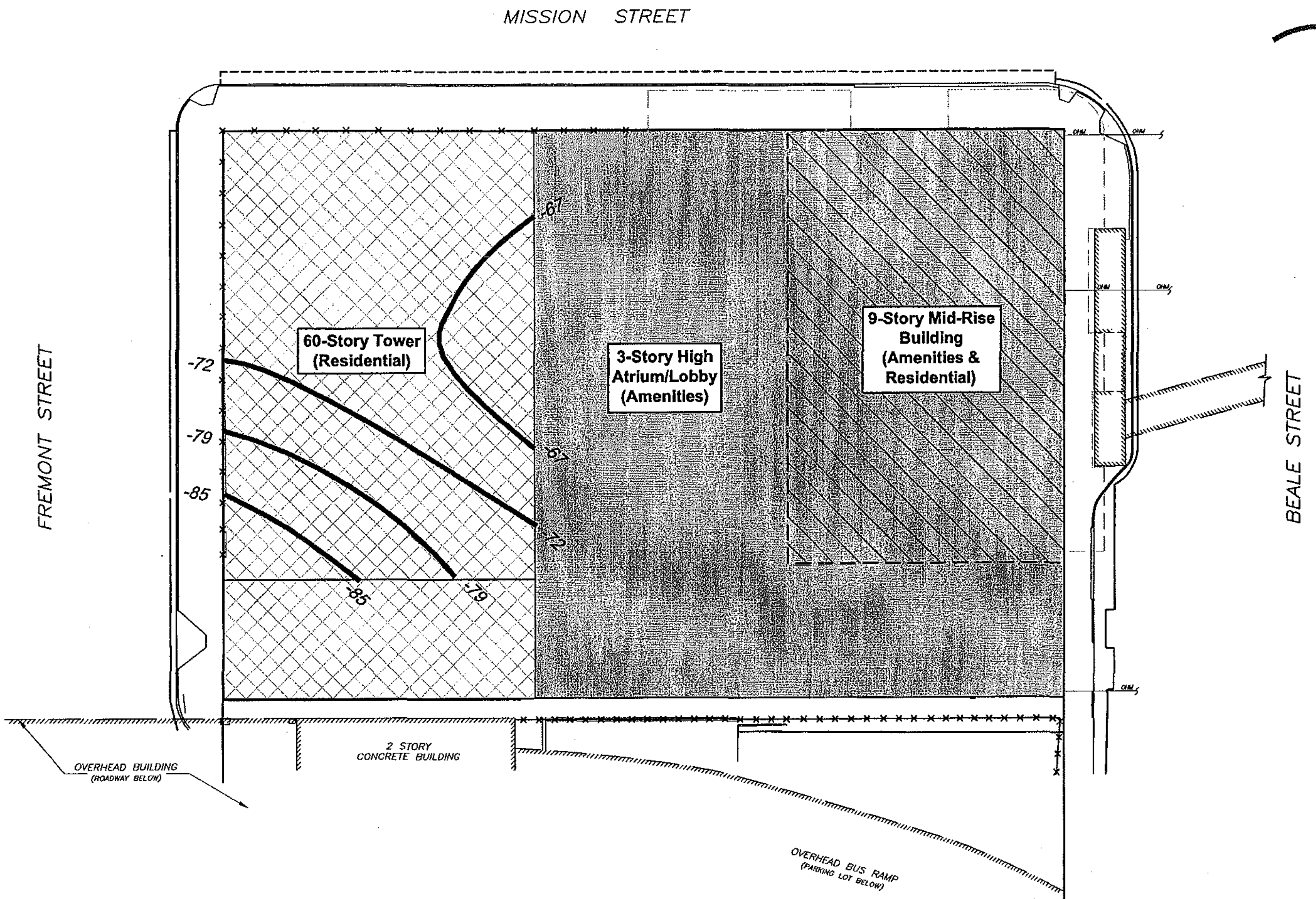
XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses 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.

**301 MISSION STREET**
San Francisco, California

**MODIFIED MERCALLI INTENSITY SCALE**

**Treadwell&Rollo**

Date 01/04/05
Project No. 3157.02
Figure 7



EXPLANATION

-67 Approximate elevation at the top of bearing layer, feet (San Francisco City datum)

Note:

Piles should pentrate the bearing layer a distance of at least ten feet. To be verified by the indicator pile program.

0 40 feet
Approximate scale

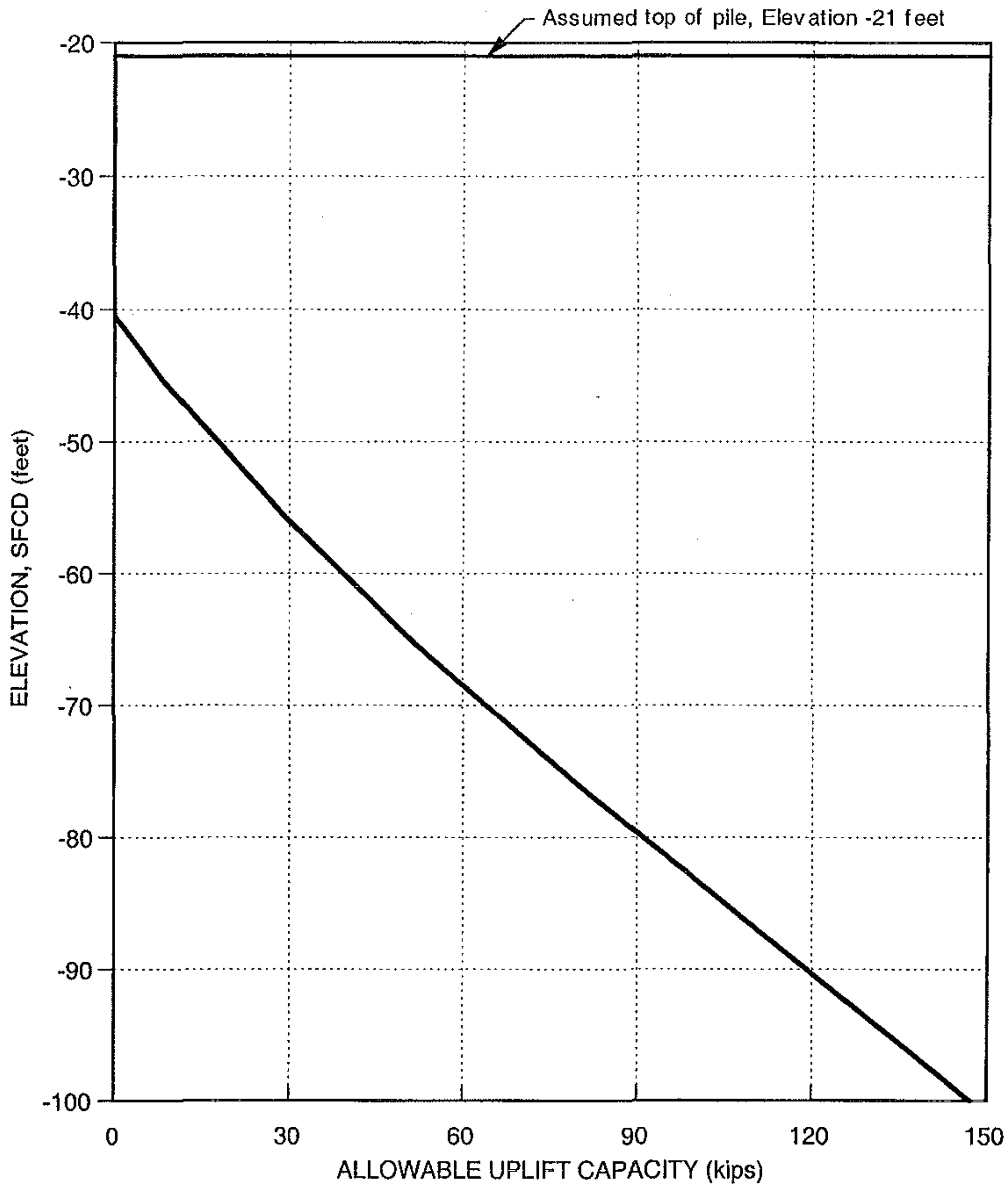
**301 MISSION STREET**
San Francisco, California

## TOP OF BEARING LAYER CONTOURS

Date 01/11/05 | Project No. 3157.02 | Figure 8

**Treadwell&Rollo**

3157.02 TOP OF BEARING-CONTOUR.DWG



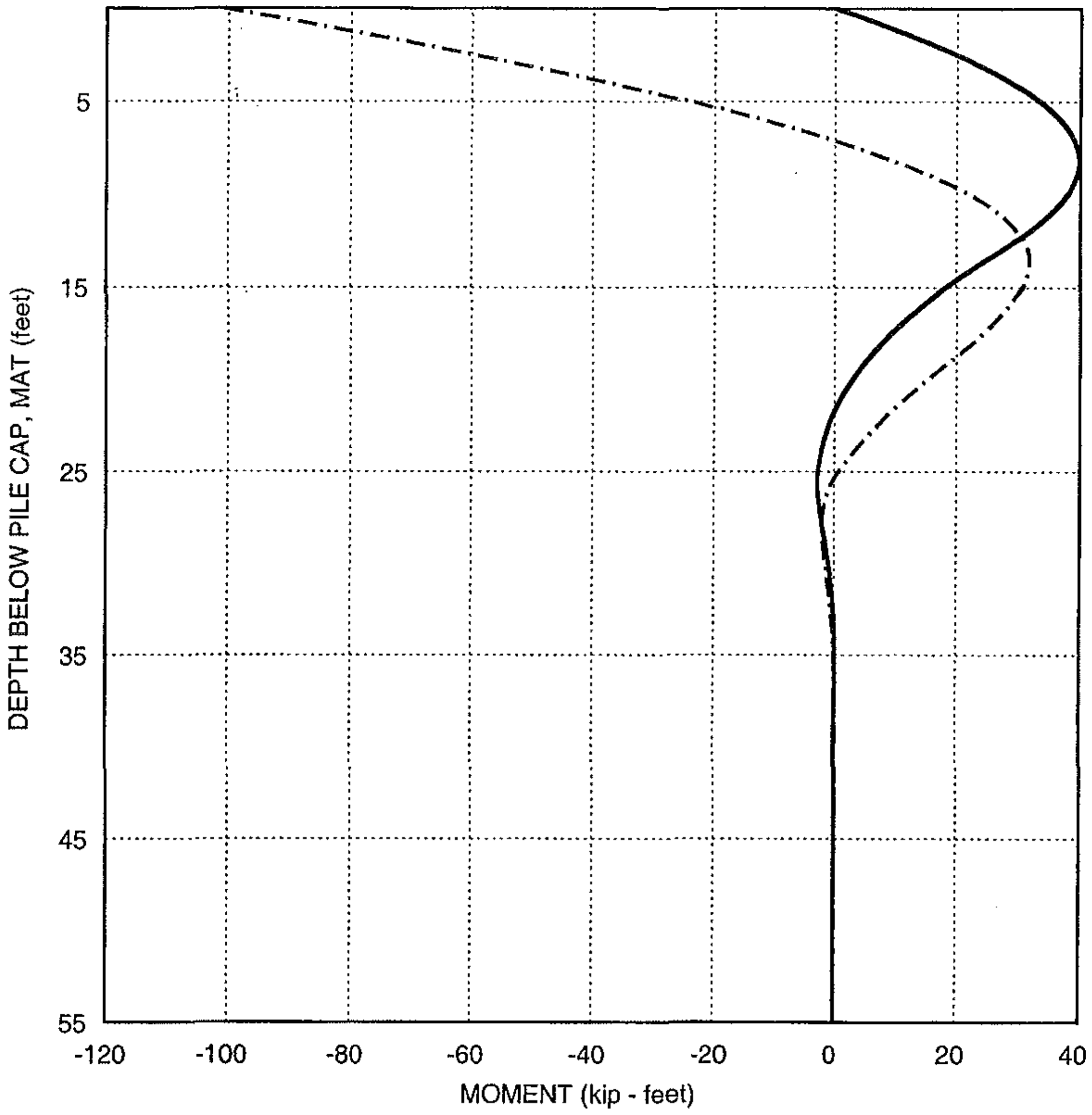
Notes: 1. The indicated capacities are based on the shear strength of the supporting soil; the structural capacity of the pile may govern.
2. Piles should be spaced no closer than three pile widths center to center.
3. City and County of San Francisco datum.

**301 MISSION STREET**
San Francisco, California

**Treadwell&Rollo**

**PILE UPLIFT CAPACITY FOR SUSTAINED LOADS 14-INCH SQUARE PRECASE-PRESTRESSED CONCRETE PILE**

Date 01/12/05 | Project No. 3157.02 | Figure 9



| Curve | Condition | Lateral Load, H (kips) |
|---|---|---|
| ——— | 14-inch free head | 7.8 |
| —·—·—· | 14-inch fixed head | 17.3 |

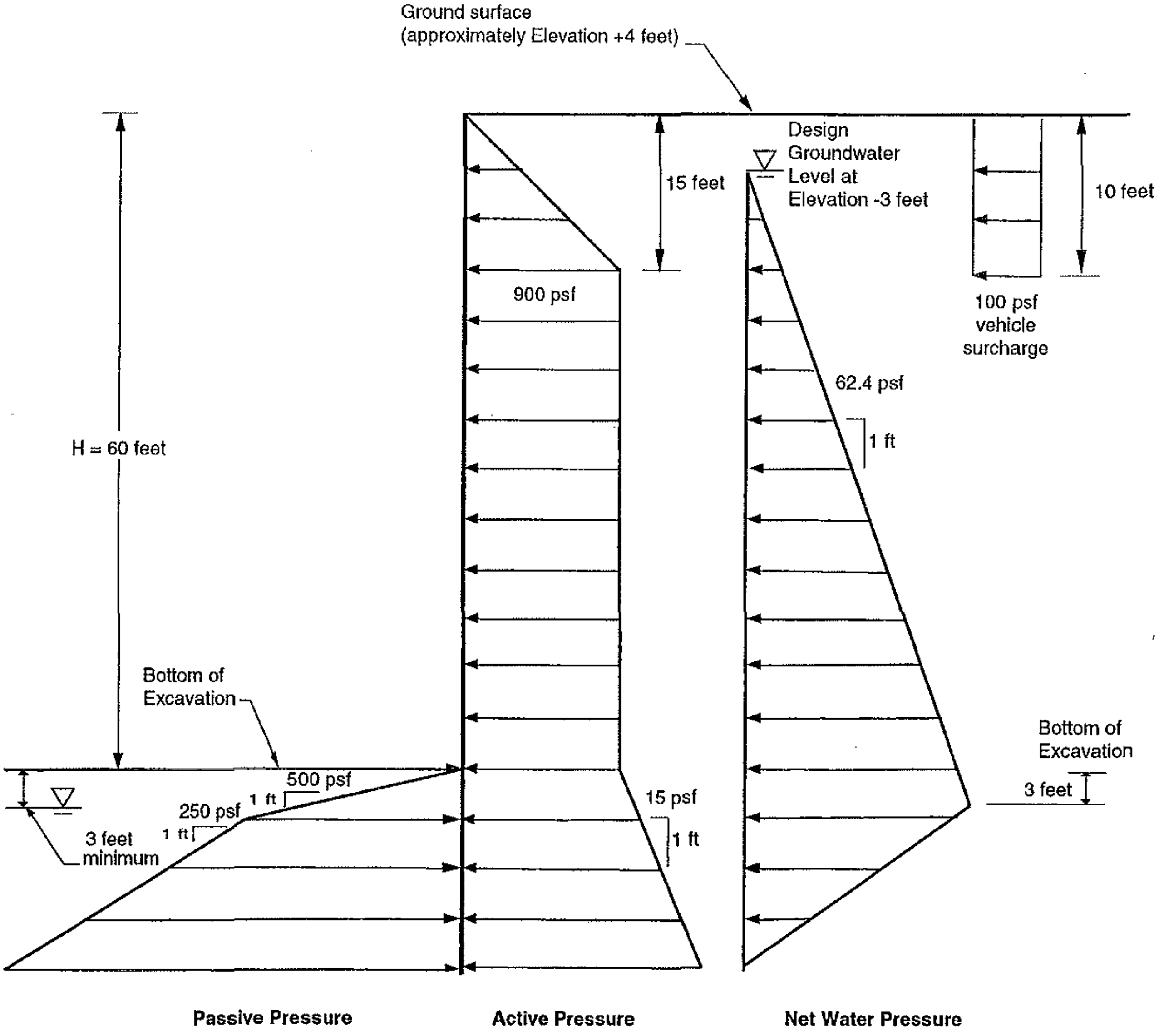
Notes: 1. The moment profiles are for 14-inch square, precast-prestressed concrete piles,at least 30 feet long.
2. Assumes maximum deflection of 0.5 inch at top of pile.
3. Assumes center to center spacing of piles is at least 8 times the pile width; for spacing less than 8 widths, see Section 9.2.1.2 of report.
4. Assumes there is no applied moment at the pile head.

**301 MISSION STREET**
San Francisco, California

**Treadwell&Rollo**

**BENDING MOMENT PROFILE FOR**
**14-INCH SQUARE**
**PRECAST-PRESTRESSED CONCRETE PILES**

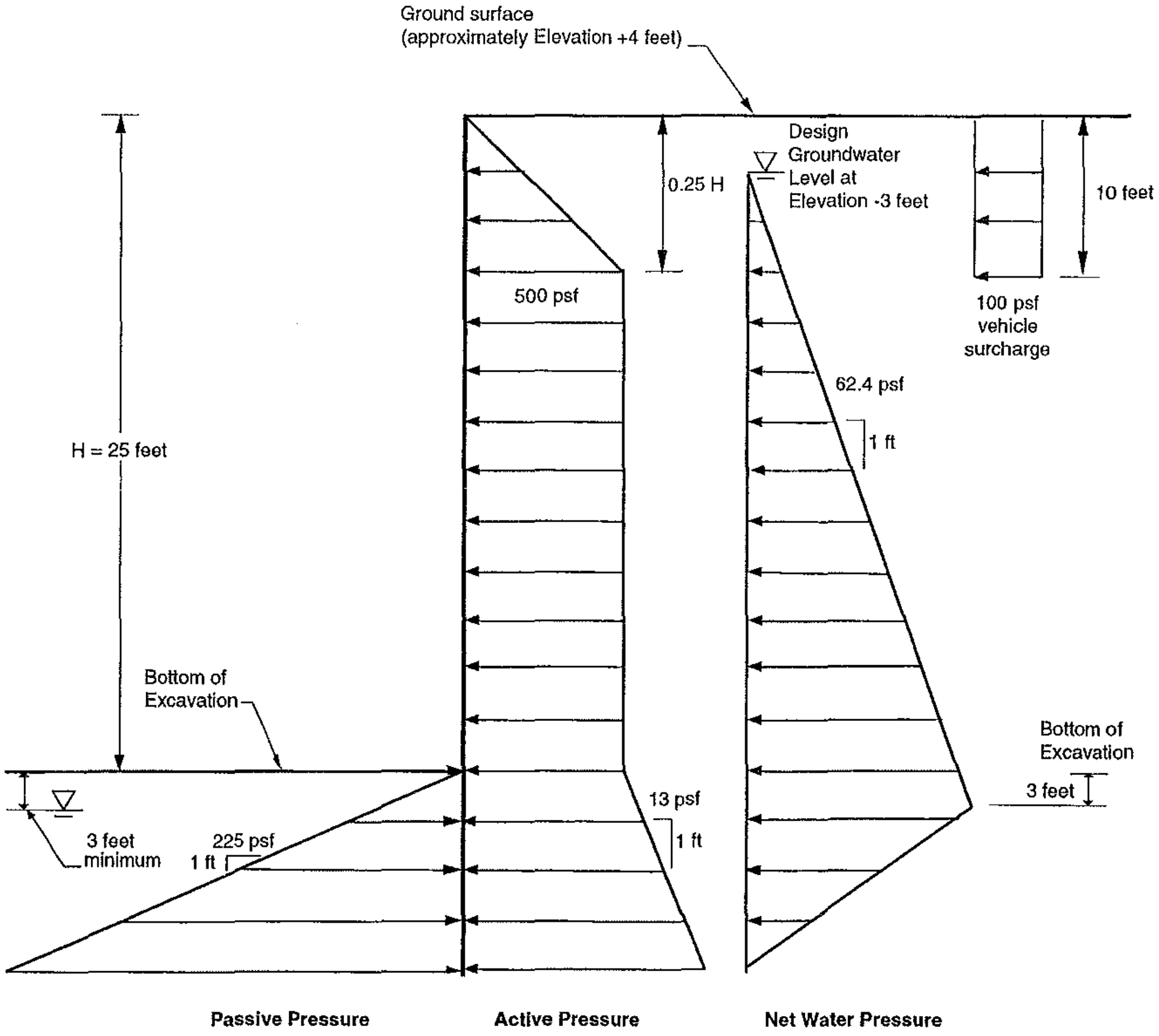
Date 01/12/05 | Project No. 3157.02 | Figure 10



**CASE 1 (see Figure 3)**

Notes:
1. The groundwater within the site will be lowered to at least 3 feet below the base of the excavation.
2. Passive pressure values do not include a factor of safety.
3. All elevations refer to San Francisco City Datum.

| | |
|---|---|
| **301 MISSION STREET**<br>San Francisco, California | **LATERAL EARTH PRESSURES FOR SOIL CEMENT WALL SHORING SYSTEM WITH INTERNAL BRACING FOR THE 60 FOOT DEEP EXCAVATION** |
| **Treadwell&Rollo** | Date 01/11/05 · Project No. 3157.02 · Figure 11 |



**CASE 2 (see Figure 3)**

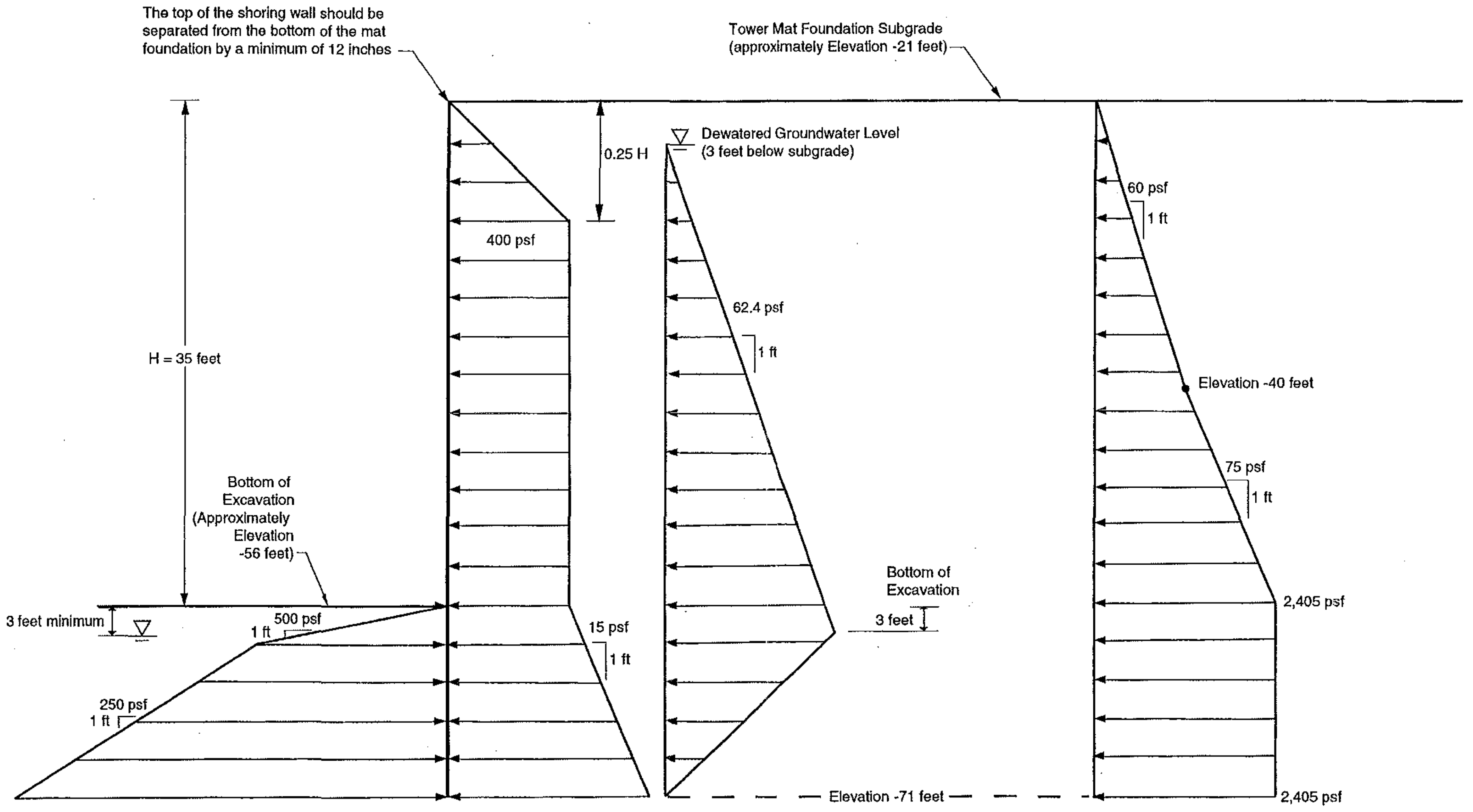
Notes:
1. The groundwater within the site will be lowered to at least 3 feet below the base of the excavation.
2. Passive pressure values do not include a factor of safety.
3. All elevations refer to San Francisco City Datum.

**301 MISSION STREET**
San Francisco, California

**Treadwell&Rollo**

**LATERAL EARTH PRESSURES FOR SOIL CEMENT WALL SHORING SYSTEM WITH INTERNAL BRACING FOR THE 25 FOOT DEEP EXCAVATION**

| | | |
|---|---|---|
| Date 01/11/05 | Project No. 3157.02 | Figure 12 |



**Passive Pressure** **Active Pressure** **Net Water Pressure** **Surcharge From Tower Foundation With Tower Constructed To 33 Floors**

CASE 3 (see Figure 3)

Notes:
1. The groundwater within the site will be lowered to at least 3 feet below the base of the excavation.
2. Passive pressure values do not include a factor of safety.
3. All elevations refer to San Francisco City Datum.
4. We assumed a 15 foot penetration of soil-mix wall below bottom of excavation. this should be checked by the shoring designer.

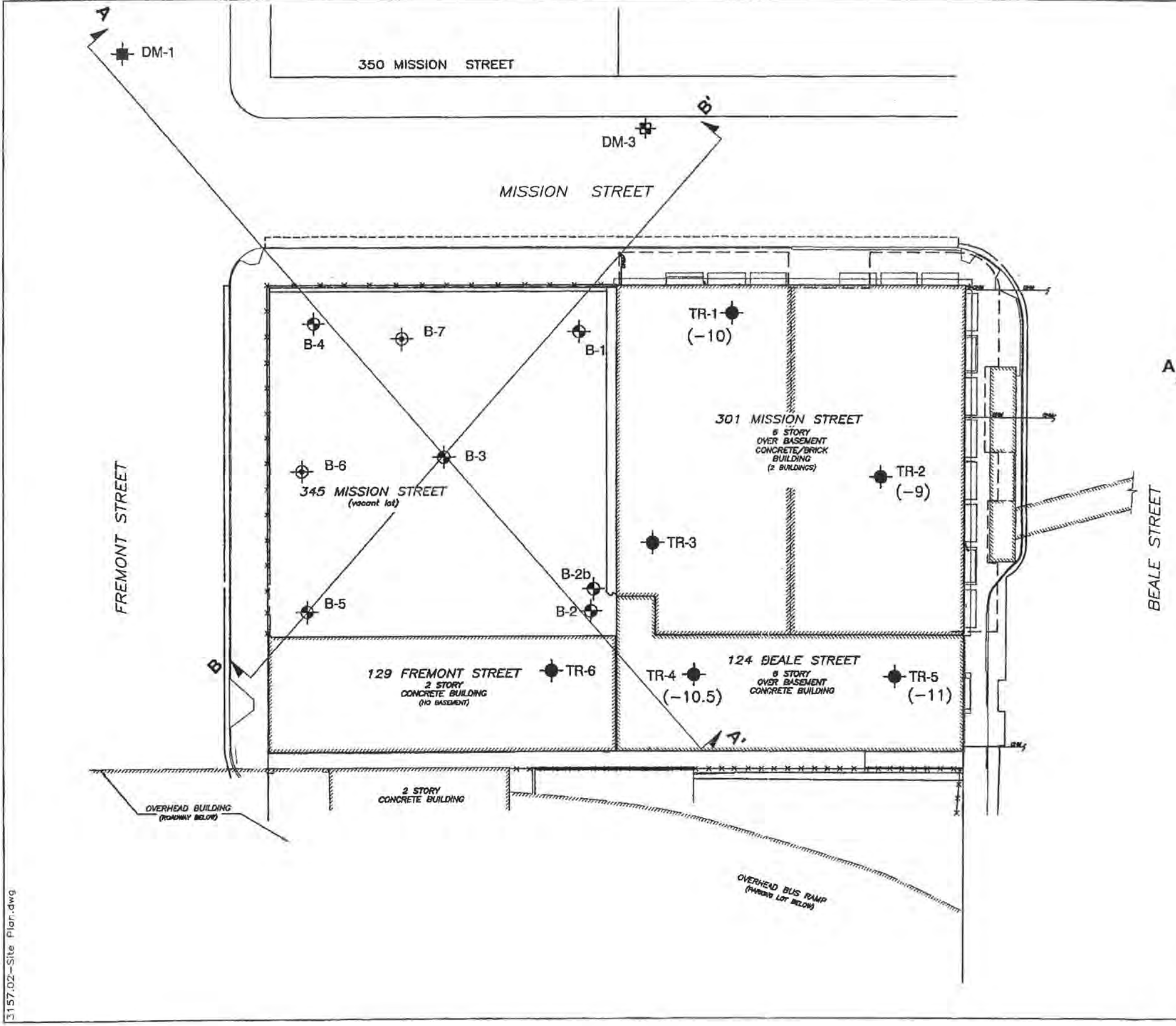
LATERAL EARTH PRESSURES
FOR SOIL CEMENT WALL SHORING SYSTEM WITH
INTERNAL BRACING FOR WALL BETWEEN THE
TOWER AND PODIUM EXCAVATIONS

301 MISSION STREET
San Francisco, California

Date 01/11/05 | Project No. 3157.02 | Figure 13

Treadwell&Rollo

# APPENDIX A



EXPLANATION

| | |
|---|---|
| B-1 ⊕ | Approximate location of soil boring by Treadwell & Rollo, Inc., June to July 2001 |
| B-7 ⊕ | Approximate location of soil boring by Treadwell & Rollo, Inc., May 2004 |
| TR-1 ● | Approximate location of environmental soil boring by Treadwell & Rollo, Inc., 5 June 2001 |
| (−9) | Approximate elevation at which groundwater was encountered, feet (San Francisco City datum) |
| DM-1 ■ | Approximate location of soil boring by Dames & Moore, November 1980 |
| DM-3 ▣ | Approximate location of soil boring by Dames & Moore, February 1966 |
| A A' | Approximate location of idealized subsurface profile |

0 40 feet

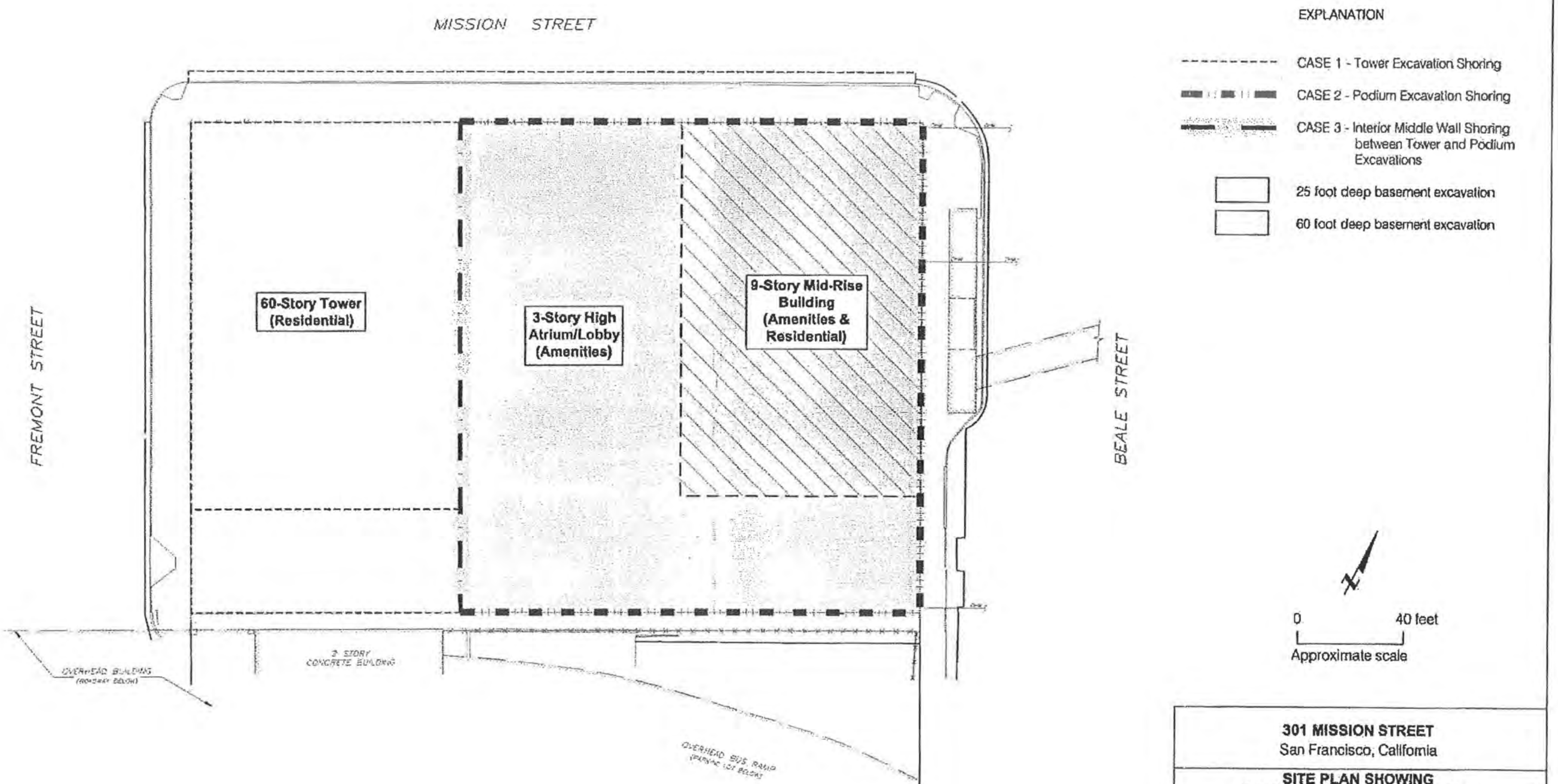
Approximate scale

**301 MISSION STREET**
San Francisco, California

**SITE PLAN**

Date 01/11/05 | Project No. 3157.02 | Figure 2

**Treadwell&Rollo**



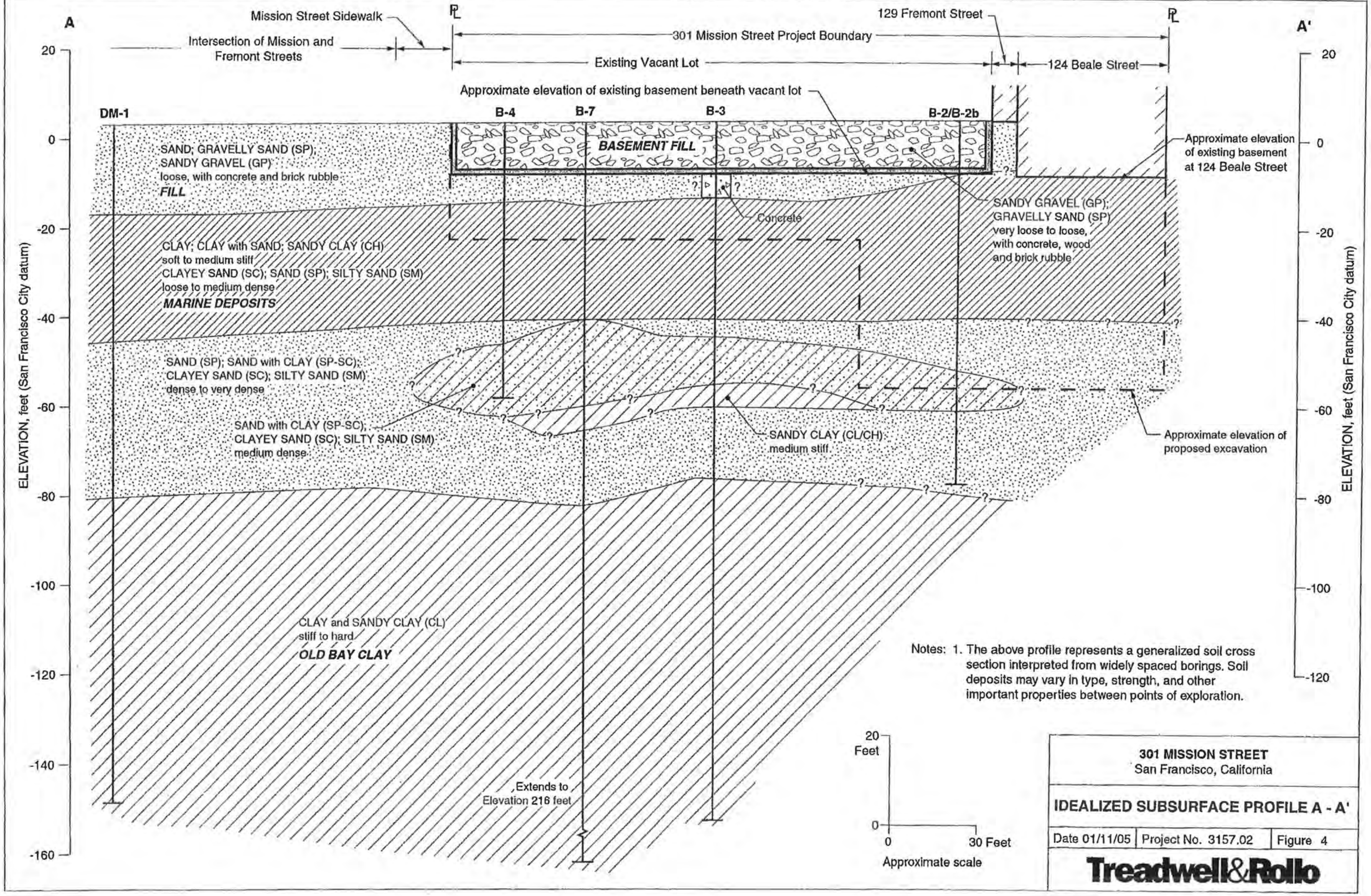
**301 MISSION STREET**
San Francisco, California

**SITE PLAN SHOWING PROPOSED DEVELOPMENT AND TEMPORARY SHORING CONDITIONS**

Date 01/04/05 | Project No. 3157.02 | Figure 3

**Treadwell&Rollo**

3157.02 PROPOSED DEVELOPMENT.DWG



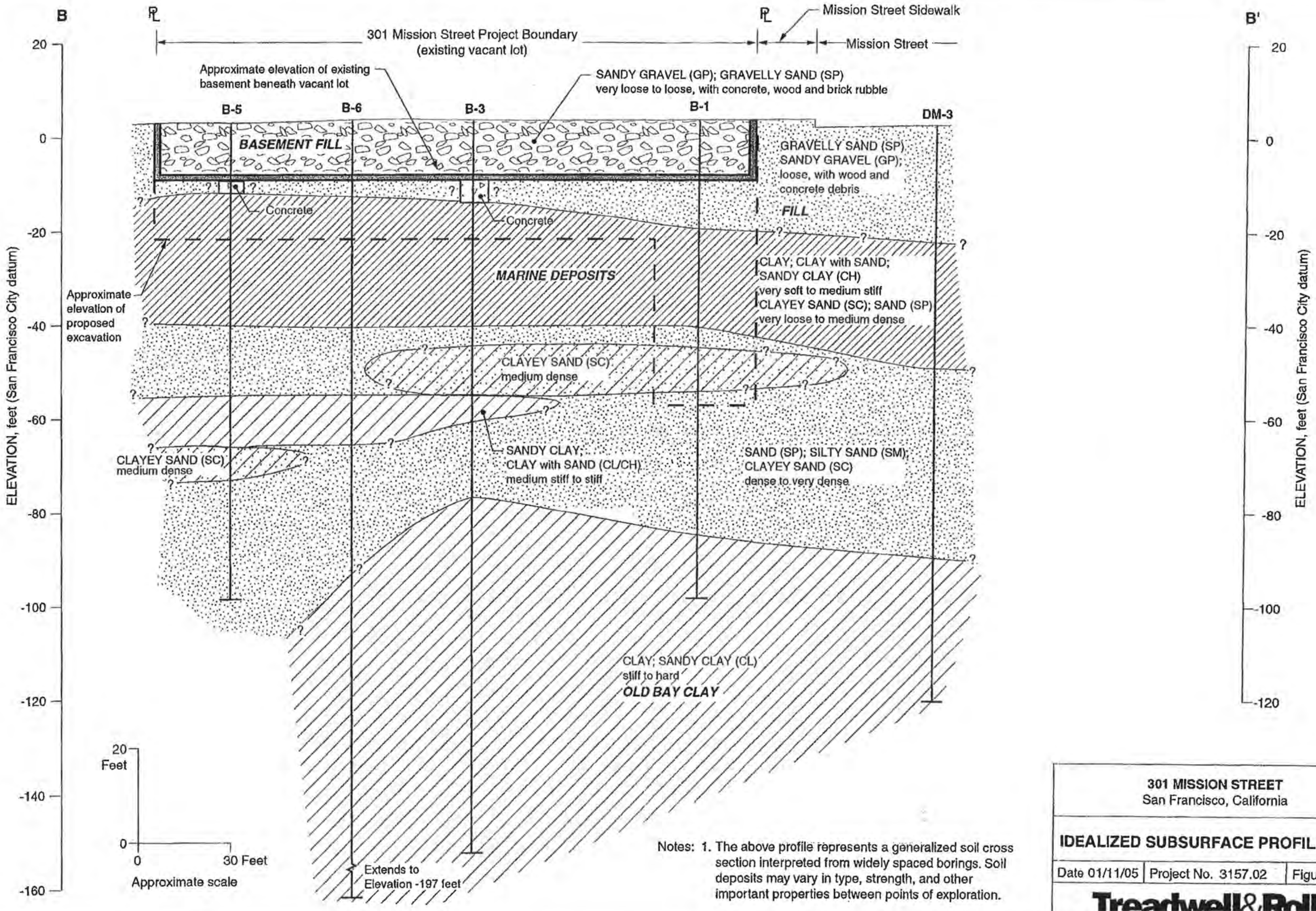
Notes: 1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

**301 MISSION STREET**
San Francisco, California

**IDEALIZED SUBSURFACE PROFILE A - A'**

| | | |
|---|---|---|
| Date 01/11/05 | Project No. 3157.02 | Figure 4 |

**Treadwell&Rollo**



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.

**301 MISSION STREET**
San Francisco, California

**IDEALIZED SUBSURFACE PROFILE B - B'**

| | | |
|---|---|---|
| Date 01/11/05 | Project No. 3157.02 | Figure 5 |

**Treadwell&Rollo**

# APPENDIX A

## Geotechnical Boring Logs

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-1

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

Date started: 6/28/01

Date finished: 6/29/01

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches

Hammer type: Safety, rope & pulley

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

LABORATORY TEST DATA

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION<br>Ground Surface Elevation: 3.5 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–11 | | | | GP | SANDY GRAVEL (GP)<br>light brown, loose, dry, with concrete and brick debris | FILL | | | | | | |
| 11 | | | | | CONCRETE SLAB 6-inches thick | | | | | | | |
| 11.5 | | | | GP | SANDY GRAVEL (GP)<br>light brown, loose, moist, with wood and concrete debris<br>∇ unstabilized groundwater level at 13 feet noted during drilling | | | | | | | |
| 13 | S&H | ▨ | 30/3" | | | | | | | | | |
| 21 | S&H | | 4 | | | | | | | | | |
| 23 | | | | CH | CLAY with SAND (CH)<br>gray, very soft to soft, wet, with shells | | | | | | | |
| 26 | O | ▨ | 50 psi | | | | | | | | 52.9 | 69 |
| 30 | | | | | | | | | | | | |

TEST GEOTECH LOG 315701_G.GPJ TR.GDT 1/12/05

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-1a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-1

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  | CLAY with SAND (CH) (continued) |  |  |  |  |  |  |
| 31 | S&H |  | 0 |  |  |  |  |  |  |  |  |
| 32 |  |  |  |  |  |  |  |  |  |  |  |
| 33 |  |  |  |  |  |  |  |  |  |  |  |
| 34 |  |  |  |  |  |  |  |  |  |  |  |
| 35 |  |  |  |  |  |  |  |  |  |  |  |
| 36 | O |  | 50 psi |  |  | TxUU | 1,400 | 685 |  | 37.1 | 85 |
| 37 |  |  |  | CH | Consolidation Test, See Figure C-8 |  |  |  |  | 35.0 | 87 |
| 38 |  |  |  |  |  |  |  |  |  |  |  |
| 39 |  |  |  |  |  |  |  |  |  |  |  |
| 40 |  |  |  |  |  |  |  |  |  |  |  |
| 41 | S&H |  | 2 |  |  |  |  |  |  | 48.4 | 72 |
| 42 |  |  |  |  |  |  |  |  |  |  |  |
| 43 |  |  |  |  |  |  |  |  |  |  |  |
| 44 |  |  |  |  | SAND (SP)<br>gray, very dense, wet |  |  |  |  |  |  |
| 45 |  |  |  |  |  |  |  |  |  |  |  |
| 46 | SPT |  | 51 | SP |  |  |  |  |  |  |  |
| 47 |  |  |  |  |  |  |  |  |  |  |  |
| 48 |  |  |  |  | CLAYEY SAND (SC)<br>gray, medium dense, wet |  |  |  |  |  |  |
| 49 |  |  |  |  |  |  |  |  |  |  |  |
| 50 |  |  |  |  |  |  |  |  |  |  |  |
| 51 | SPT |  | 13 |  | LL=17, PI=9, See Figure C-1 |  |  |  | 19 | 24.1 |  |
| 52 |  |  |  |  |  |  |  |  |  |  |  |
| 53 |  |  |  | SC |  |  |  |  |  |  |  |
| 54 |  |  |  |  |  |  |  |  |  |  |  |
| 55 |  |  |  |  |  |  |  |  |  |  |  |
| 56 |  |  |  |  |  |  |  |  |  |  |  |
| 57 |  |  |  |  |  |  |  |  |  |  |  |
| 58 |  |  |  |  | CLAYEY SAND (SC)<br>olive-gray, dense, wet |  |  |  |  |  |  |
| 59 |  |  |  | SC |  |  |  |  |  |  |  |
| 60 |  |  |  |  |  |  |  |  |  |  |  |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-1b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-1

PAGE 3 OF 4

| DEPTH (feet) | SAMPLES | | | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sampler Type | Sample | SPT N-Value[1] | | | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
| | S&H | ▨ | 43 | | CLAYEY SAND (SC) (continued) | | | | | 16.2 | 119 |
| 61 | | | | | | | | | | | |
| 62 | | | | | | | | | | | |
| 63 | | | | SC | | | | | | | |
| 64 | | | | | | | | | | | |
| 65 | | | | | | | | | | | |
| 66 | | | | | | | | | | | |
| 67 | | | | | CLAYEY SAND (SC)<br>yellow-brown, very dense, wet | | | | | | |
| 68 | | | | | | | | | | | |
| 69 | | | | | | | | | | | |
| 70 | S&H | ▨ | 30/5" | | | | | | | | |
| 71 | | | | | | | | | | | |
| 72 | | | | SC | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | | | | | | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | | SILTY SAND (SM)<br>olive-brown, very dense, wet | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | S&H | ▨ | 30/5" | | | | | | | 18.7 | 116 |
| 81 | | | | | | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | SM | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | | | | | | | | | | | |
| 86 | | | | | | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | | CLAY (CL) **[OLD BAY CLAY]**<br>gray, very stiff, wet | | | | | | |
| 89 | | | | CL | | | | | | | |
| 90 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-1c

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-1

PAGE 4 OF 4

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA: Fines % | LABORATORY TEST DATA: Natural Moisture Content, % | LABORATORY TEST DATA: Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CLAY (CL) (continued) | | | | | | |
| 91 | O | | 100 psi | | | TxUU | 3,000 | 1,910 | | 20.5 | 110 |
| 92 | | | | | | | | | | | |
| 93 | | | | | | | | | | | |
| 94 | | | | | | | | | | | |
| 95 | | | | | | | | | | | |
| 96 | | | | CL | | | | | | | |
| 97 | | | | | | | | | | | |
| 98 | | | | | | | | | | | |
| 99 | | | | | | | | | | | |
| 100 | | | | | green-gray, hard | | | | | | |
| 101 | S&H | | 34 | | | | | | | | |
| 102 | | | | | | | | | | | |
| 103 | | | | | | | | | | | |
| 104 | | | | | | | | | | | |
| 105 | | | | | | | | | | | |
| 106 | | | | | | | | | | | |
| 107 | | | | | | | | | | | |
| 108 | | | | | | | | | | | |
| 109 | | | | | | | | | | | |
| 110 | | | | | | | | | | | |
| 111 | | | | | | | | | | | |
| 112 | | | | | | | | | | | |
| 113 | | | | | | | | | | | |
| 114 | | | | | | | | | | | |
| 115 | | | | | | | | | | | |
| 116 | | | | | | | | | | | |
| 117 | | | | | | | | | | | |
| 118 | | | | | | | | | | | |
| 119 | | | | | | | | | | | |
| 120 | | | | | | | | | | | |

Boring terminated at 101.5 feet below ground surface.
Boring backfilled with cement grout.
Unstabilized groundwater encountered at 13 feet during drilling.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-1d

TEST GEOTECH LOG 315701_G.GPJ TR.GDT 1/12/05

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

Date started: 6/29/01

Date finished: 6/29/01

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches

Hammer type: Safety, rope & pulley

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

LABORATORY TEST DATA

| DEPTH (feet) | SAMPLES Sampler Type | SAMPLES Sample | SAMPLES SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION<br>Ground Surface Elevation: 3.5 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SANDY GRAVEL with RUBBLE (GP)<br>light brown, loose, dry, with concrete and brick debris | ↑ | | | | | | |
| 1 | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | |
| 3 | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | |
| 5 | | | | GP | | FILL | | | | | | |
| 6 | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | CONCRETE SLAB, 5 to 6-inches thick | ↓ | | | | | | |
| | | | | | CLAYEY SAND (SC)<br>dark gray, very loose, wet, with shells | | | | | | | |
| 12 | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | |
| 15 | S&H | ■ | 2 | SC | | | | | | | | |
| 16 | | | | | | | | | | | | |
| 17 | | | | | | | | | | | | |
| 18 | | | | | | | | | | | | |
| 19 | | | | | | | | | | | | |
| 20 | | | | | CLAY with SAND (CH)<br>gray, very soft to soft, wet, with shells | | | | | | | |
| 21 | O | ■ | 50 psi | | | | | | | | 39.0 | 85 |
| 22 | | | | | | | | | | | | |
| 23 | | | | | | | | | | | | |
| 24 | | | | | | | | | | | | |
| 25 | S&H | ■ | 0 | CH | | | | | | | | |
| 26 | | | | | | | | | | | | |
| 27 | | | | | | | | | | | | |
| 28 | | | | | | | | | | | | |
| 29 | | | | | | | | | | | | |
| 30 | | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-2a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-2

PAGE 2 OF 2

| | SAMPLES | | | | | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |
| | | | | | CLAY with SAND (CH) (continued) | | | | | | |
| 31 | O | ● | | CH | | | | | | | |
| 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 | | | | | | | | | | | |

Boring terminated at 32.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.

[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-2b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-2b

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

Date started: 7/3/01

Date finished: 7/3/01

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches

Hammer type: Safety, rope & pulley

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

LABORATORY TEST DATA

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION<br>Ground Surface Elevation: 3.5 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | | | SANDY GRAVEL with RUBBLE (GP)<br>light brown, loose, dry, with concrete, brick and metal debris | | | | | | | |
| 2 | | | | | | | | | | | | |
| 3 | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | |
| 5 | | | | | | | | | | | | |
| 6 | | | | GP | | FILL | | | | | | |
| 7 | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | CONCRETE SLAB 5- to 6-inches thick | | | | | | | |
| 12 | | | | | SANDY CLAY (CH)<br>black, very soft, wet | | | | | | | |
| 13 | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | |
| 15 | | ▨ | | | | | | | | | | |
| 16 | S&H | ▨ | 1 | | | | | | | | | |
| 17 | | | | | | | | | | | | |
| 18 | | | | | | | | | | | | |
| 19 | | | | | | | | | | | | |
| 20 | | | | | | | | | | | | |
| 21 | | | | CH | | | | | | | | |
| 22 | | | | | | | | | | | | |
| 23 | | | | | | | | | | | | |
| 24 | | | | | | | | | | | | |
| 25 | | | | | | | | | | | | |
| 26 | | | | | | | | | | | | |
| 27 | | | | | | | | | | | | |
| 28 | | | | | | | | | | | | |
| 29 | | | | | | | | | | | | |
| 30 | | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-3a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-2b

PAGE 2 OF 3

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 31 | | | | | CLAYEY SAND (SC)<br>gray, loose, wet | | | | | | |
| 32 | | | | | | | | | | | |
| 33 | | | | | | | | | | | |
| 34 | | | | SC | | | | | | | |
| 35 | | | | | | | | | | | |
| 36 | S&H | ■ | 7 | | Particle Size Analysis, See Figure C-2 | | | | 24 | 22.6 | 104 |
| 37 | | | | | | | | | | | |
| 38 | | | | | CLAY with SAND (CH)<br>gray, soft to medium stiff, wet, with shells | | | | | | |
| 39 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |
| 41 | S&H | ■ | 4 | CH | | | | | | | |
| 42 | | | | | | | | | | | |
| 43 | | | | | | | | | | | |
| 44 | | | | | SAND with CLAY (SP-SC)<br>dark gray, very dense, wet | | | | | | |
| 45 | | | | | | | | | | | |
| 46 | S&H | ■ | 49/9" | | | | | | | | |
| 47 | | | | | | | | | | | |
| 48 | | | | | | | | | | | |
| 49 | | | | | | | | | | | |
| 50 | | | | SP-SC | | | | | | | |
| 51 | SPT | ◪ | 58 | | | | | | | | |
| 52 | | | | | | | | | | | |
| 53 | | | | | | | | | | | |
| 54 | | | | | | | | | | | |
| 55 | | | | | | | | | | | |
| 56 | | | | | SAND with CLAY (SP-SC)<br>gray, medium dense to dense, wet | | | | | | |
| 57 | | | | | | | | | | | |
| 58 | | | | SP-SC | | | | | | | |
| 59 | | | | | | | | | | | |
| 60 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-3b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-2b

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA: Fines % | LABORATORY TEST DATA: Natural Moisture Content, % | LABORATORY TEST DATA: Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SAND with CLAY (SP-SC) (continued) | | | | | | |
| 61 | SPT | ◩ | 30 | | | | | | | | |
| 62 | | | | SP-SC | | | | | | | |
| 63 | | | | | | | | | | | |
| 64 | | | | | | | | | | | |
| 65 | | | | | SAND with CLAY (SP-SC)<br>green-gray and gray, very dense, wet | | | | | | |
| 66 | | | | | | | | | | | |
| 67 | | | | | | | | | | | |
| 68 | | | | | | | | | | | |
| 69 | | | | | | | | | | | |
| 70 | S&H | ■ | 30/6" | SP-SC | | | | | | | |
| 71 | | | | | | | | | | | |
| 72 | | | | | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | | | | | CLAYEY SAND (SC)<br>light gray-brown, very dense, wet | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | SC | | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | S&H | ■ | 30/4" | | | | | | | | |
| 81 | | | | | | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | | | | | | | | | | | |
| 86 | | | | | | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | | | | | | | | |
| 89 | | | | | | | | | | | |
| 90 | | | | | | | | | | | |

Boring terminated at 80.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-3c

TEST GEOTECH LOG 315701 G.GPJ TR.GDT 1/12/05

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-3

PAGE 1 OF 6

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

Date started: 6/26/01

Date finished: 6/27/01

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches

Hammer type: Safety, rope & pulley

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

| DEPTH (feet) | SAMPLES Sampler Type | SAMPLES Sample | SAMPLES SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION<br>Ground Surface Elevation: 3.5 feet² | | LABORATORY TEST DATA Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | GRAVELLY SAND (SP)<br>gray-brown, dense, dry, with concrete and brick debris | FILL | | | | | | |
| 6 | SPT | ▨ | 46 | SP | | | | | | | | |
| 10 | | | | | ∇ 6-27-01 | | | | | | | |
| 11 | | | | | ∇ 6-26-01<br>CONCRETE SLAB 7-inches thick | | | | | | | |
| 12 | | | | | WOOD | | | | | | | |
| 13 | S&H | ▨ | 24/5" | | | | | | | | | |
| 14 | | | | | CONCRETE | | | | | | | |
| 17 | | | | | SAND (SP)<br>dark gray, loose, wet | | | | | | | |
| 19 | S&H | ▨ | 5 | SP | | | | | | | | |
| 22 | | | | | CLAY (CH)<br>gray, soft, wet, with shells and some fine sand | | | | | | | |
| 26 | O | ▨ | 50 psi | CH | | | | | | | 28.9 | 95 |
| 30 | | | | | | | | | | | | |

TEST GEOTECH LOG 315701_G.GPJ TR.GDT 1/12/05

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-4a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-3

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ▨ | | | CLAY (CH) (continued) | | | | | | |
| 31 | O | ▨ | 50 psi | | | | | | | 51.2 | 72 |
| 32 | | ▨ | | CH | | | | | | | |
| 33 | | | | | | | | | | | |
| 34 | | | | | | | | | | | |
| 35 | | ▨ | | | SANDY CLAY (CH)<br>gray, soft, wet, with silty sand lenses | | | | | | |
| 36 | O | ▨ | 50 psi | | | | | | | 37.6 | 65 |
| 37 | | ▨ | | CH | Consolidation Test, See Figure C-9 | | | | | 44.6 | 75 |
| 38 | | | | | | | | | | | |
| 39 | | | | | CLAYEY SAND (SC)<br>gray, medium dense, wet | | | | | | |
| 40 | | ▨ | | | | | | | | | |
| 41 | O | ▨ | 50 psi | SC | | TxUU | 1,500 | 595 | 39 | 32.0 | 91 |
| 42 | | ▨ | | | | | | | | | |
| 43 | | | | | | | | | | | |
| 44 | | | | | SILTY SAND (SM)<br>green-gray, very dense, wet | | | | | | |
| 45 | S&H | ▨ | 30/4" | SM | | | | | | | |
| 46 | | | | | | | | | | | |
| 47 | | | | | | | | | | | |
| 48 | | | | | CLAYEY SAND (SC)<br>green-gray, medium dense, wet | | | | | | |
| 49 | | | | | | | | | | | |
| 50 | | ◩ | | | | | | | | | |
| 51 | SPT | ◩ | 23 | | | | | | | | |
| 52 | | | | | | | | | | | |
| 53 | | | | SC | | | | | | | |
| 54 | | | | | | | | | | | |
| 55 | | | | | | | | | | | |
| 56 | | | | | | | | | | | |
| 57 | | | | | | | | | | | |
| 58 | | | | | | | | | | | |
| 59 | | | | | | | | | | | |
| 60 | | | | CH | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-4b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-3

| DEPTH (feet) | SAMPLES<br>Sampler Type | <br>Sample | <br>SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA<br>Type of Strength Test | <br>Confining Pressure Lbs/Sq Ft | <br>Shear Strength Lbs/Sq Ft | <br>Fines % | <br>Natural Moisture Content, % | <br>Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 61 | ST | ▨ | 25 psi | | SANDY CLAY (CH)<br>dark gray, medium stiff, wet, with shells | | | | | 47.1 | 75 |
| 62 | | | | CH | | | | | | | |
| 63 | | | | | | | | | | | |
| 64 | | | | | SILTY SAND (SM)<br>green-gray, very dense, wet | | | | | | |
| 65 | | | | | | | | | | | |
| 66 | | | | | | | | | | | |
| 67 | SPT | ◪ | 54 | | | | | | | | |
| 68 | | | | | | | | | | | |
| 69 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 71 | | | | | | | | | | | |
| 72 | | | | SM | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | SPT | ◪ | 50/6" | | | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | | | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | | | | | SANDY CLAY (CL)<br>orange-brown and olive, hard, wet | | | | | | |
| 81 | | | | | | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | S&H | ▨ | 38 | CL | | | | | | 20.1 | 112 |
| 86 | | | | | | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | | | | | | | | |
| 89 | | | | | | | | | | | |
| 90 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01
Figure: A-4c

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-3

| DEPTH (feet) | SAMPLES | | | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sampler Type | Sample | SPT N-Value$^1$ | | | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
| | | | | | SANDY CLAY (CL) (continued) | | | | | | |
| 91 | | | | | | | | | | | |
| 92 | | | | | | | | | | | |
| 93 | | | | | | | | | | | |
| 94 | | | | | | | | | | | |
| 95 | SPT | ◩ | 45 | | olive | | | | | | |
| 96 | | | | CL | | | | | | | |
| 97 | | | | | | | | | | | |
| 98 | | | | | | | | | | | |
| 99 | | | | | | | | | | | |
| 100 | | | | | | | | | | | |
| 101 | | | | | | | | | | | |
| 102 | | | | | | | | | | | |
| 103 | | | | | CLAY (CL) **[OLD BAY CLAY]**<br>gray, very stiff, wet, with trace fine sand | | | | | | |
| 104 | | | | | | | | | | | |
| 105 | S&H | ■ | 22 | | | TxUU | 3,500 | 1,865 | | 25.2 | 100 |
| 106 | | | | | | | | | | | |
| 107 | | | | | | | | | | | |
| 108 | | | | | | | | | | | |
| 109 | | | | | | | | | | | |
| 110 | | | | | | | | | | | |
| 111 | | | | CL | | | | | | | |
| 112 | | | | | | | | | | | |
| 113 | | | | | | | | | | | |
| 114 | | | | | | | | | | | |
| 115 | S&H | ■ | 11 | | stiff | | | | | | |
| 116 | | | | | | | | | | | |
| 117 | | | | | | | | | | | |
| 118 | | | | | | | | | | | |
| 119 | | | | | | | | | | | |
| 120 | | | | | | | | | | | |

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-3

PAGE 5 OF 6

| DEPTH (feet) | SAMPLES Sampler Type | SAMPLES Sample | SAMPLES SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA Type of Strength Test | LABORATORY TEST DATA Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA Fines % | LABORATORY TEST DATA Natural Moisture Content, % | LABORATORY TEST DATA Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CLAY (CL) (continued) | | | | | | |
| 121 | | | | | | | | | | | |
| 122 | | | | | | | | | | | |
| 123 | | | | | | | | | | | |
| 124 | | | | | | | | | | | |
| 125 | ST | ▨ | 50 psi | | Consolidation Test, See Figure C-10 | | | | | 44.6 | 76 |
| 126 | | | | | | | | | | | |
| 127 | | | | | | | | | | | |
| 128 | | | | | | | | | | | |
| 129 | | | | | | | | | | | |
| 130 | | | | | | | | | | | |
| 131 | | | | | | | | | | | |
| 132 | | | | | | | | | | | |
| 133 | | | | | | | | | | | |
| 134 | | | | | | | | | | | |
| 135 | S&H | ▨ | 17 | | very stiff | | | | | | |
| 136 | | | | | | | | | | | |
| 137 | | | | | | | | | | | |
| 138 | | | | | | | | | | | |
| 139 | | | | | | | | | | | |
| 140 | | | | | | | | | | | |
| 141 | | | | | | | | | | | |
| 142 | | | | | | | | | | | |
| 143 | | | | | | | | | | | |
| 144 | | | | | | | | | | | |
| 145 | ST | ▨ | 50 psi | | | | | | | | |
| 146 | | | | | | | | | | | |
| 147 | | | | | | | | | | | |
| 148 | | | | | | | | | | | |
| 149 | | | | | | | | | | | |
| 150 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-4e

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-3

PAGE 6 OF 6

| DEPTH (feet) | SAMPLES<br>Sampler Type | <br>Sample | <br>SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA<br>Type of Strength Test | <br>Confining Pressure Lbs/Sq Ft | <br>Shear Strength Lbs/Sq Ft | <br>Fines % | <br>Natural Moisture Content, % | <br>Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CLAY (CL) (continued) | | | | | | |
| 151 | | | | | | | | | | | |
| 152 | | | | | | | | | | | |
| 153 | | | | | | | | | | | |
| 154 | | | | | | | | | | | |
| 155 | S&H | ■ | 20 | | | | | | | | |
| 156 | | | | | | | | | | | |
| 157 | | | | | | | | | | | |
| 158 | | | | | | | | | | | |
| 159 | | | | | | | | | | | |
| 160 | | | | | | | | | | | |
| 161 | | | | | | | | | | | |
| 162 | | | | | | | | | | | |
| 163 | | | | | | | | | | | |
| 164 | | | | | | | | | | | |
| 165 | | | | | | | | | | | |
| 166 | | | | | | | | | | | |
| 167 | | | | | | | | | | | |
| 168 | | | | | | | | | | | |
| 169 | | | | | | | | | | | |
| 170 | | | | | | | | | | | |
| 171 | | | | | | | | | | | |
| 172 | | | | | | | | | | | |
| 173 | | | | | | | | | | | |
| 174 | | | | | | | | | | | |
| 175 | | | | | | | | | | | |
| 176 | | | | | | | | | | | |
| 177 | | | | | | | | | | | |
| 178 | | | | | | | | | | | |
| 179 | | | | | | | | | | | |
| 180 | | | | | | | | | | | |

Boring terminated at 155.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 10 to 11 feet during drilling.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-4f

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-4

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

Date started: 6/27/01 | Date finished: 6/28/01

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches | Hammer type: Safety, rope & pulley

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

LABORATORY TEST DATA

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION<br>Ground Surface Elevation: 3.5 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SANDY GRAVEL (GP)<br>gray-brown, dry, with concrete and brick debris | FILL | | | | | | |
| 1 | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | |
| 3 | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | |
| 5 | | | | GP | | | | | | | | |
| 6 | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | CONCRETE SLAB 7.5-inches thick | | | | | | | |
| | | | | | RUBBLE<br>loose, concrete, brick | | | | | | | |
| 12 | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | |
| 16 | S&H | ⊡ | 5 | | | | | | | | | |
| 17 | | | | | | | | | | | | |
| 18 | | | | | SANDY CLAY (CH)<br>dark gray, soft, wet | | | | | | | |
| 19 | | | | | | | | | | | | |
| 20 | | | | | | | | | | | | |
| 21 | O | ▨ | 50 psi | CH | | | | | | | | |
| 22 | | | | | | | | | | | | |
| 23 | | | | | | | | | | | | |
| 24 | | | | | CLAY with SAND (CH)<br>gray, soft, wet, with shells | | | | | | | |
| 25 | | | | | | | | | | | | |
| 26 | O | ▨ | 50 psi | | | | | | | | 47.0 | 71 |
| 27 | | | | CH | | | | | | | | |
| 28 | | | | | | | | | | | | |
| 29 | | | | | | | | | | | | |
| 30 | | | | | | | | | | | | |

TEST GEOTECH LOG 315701_G.GPJ TR.GDT 1/12/05

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-5a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-4

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  | CLAY with SAND (CH) (continued) |  |  |  |  |  |  |
| 31 | O |  | 50 psi |  |  |  |  |  |  | 33.2 | 86 |
| 32 |  |  |  | CH |  |  |  |  |  |  |  |
| 33 |  |  |  |  |  |  |  |  |  |  |  |
| 34 |  |  |  |  | CLAYEY SAND (SC)<br>gray, medium dense, wet |  |  |  |  |  |  |
| 35 |  |  |  |  |  |  |  |  |  |  |  |
| 36 | O |  | 75 psi |  |  | TxUU | 1,400 | 980 | 19 | 24.0 | 103 |
| 37 |  |  |  |  |  |  |  |  |  |  |  |
| 38 |  |  |  |  |  |  |  |  |  |  |  |
| 39 |  |  |  | SC |  |  |  |  |  |  |  |
| 40 |  |  |  |  |  |  |  |  |  |  |  |
| 41 | S&H |  | 19 |  |  |  |  |  | 24 | 25.4 | 101 |
| 42 |  |  |  |  |  |  |  |  |  |  |  |
| 43 |  |  |  |  |  |  |  |  |  |  |  |
| 44 |  |  |  |  | SAND (SP)<br>green-gray, very dense, wet |  |  |  |  |  |  |
| 45 | S&H |  | 30/5" |  |  |  |  |  |  |  |  |
| 46 |  |  |  | SP |  |  |  |  |  |  |  |
| 47 |  |  |  |  |  |  |  |  |  |  |  |
| 48 |  |  |  |  |  |  |  |  |  |  |  |
| 49 |  |  |  |  | SILTY SAND (SM)<br>gray, medium dense, wet<br>LL=17, PI=4, See Figure C-1 |  |  |  |  |  |  |
| 50 |  |  |  |  |  |  |  |  |  |  |  |
| 51 | SPT |  | 12 |  |  |  |  |  | 21 | 27.7 |  |
| 52 |  |  |  |  |  |  |  |  |  |  |  |
| 53 |  |  |  | SM |  |  |  |  |  |  |  |
| 54 |  |  |  |  |  |  |  |  |  |  |  |
| 55 |  |  |  |  |  |  |  |  |  |  |  |
| 56 |  |  |  |  |  |  |  |  |  |  |  |
| 57 |  |  |  |  |  |  |  |  |  |  |  |
| 58 |  |  |  |  | CLAYEY SAND (SC)<br>green-gray, medium dense, wet |  |  |  |  |  |  |
| 59 |  |  |  | SC |  |  |  |  |  |  |  |
| 60 |  |  |  |  |  |  |  |  |  |  |  |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-5b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-4

PAGE 3 OF 3

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 61 | S&H | ■ | 28 | SC | CLAYEY SAND (SC) (continued) | | | | 14 | 20.2 | 111 |
| 62 | | | | | | | | | | | |
| 63 | | | | | | | | | | | |
| 64 | | | | | | | | | | | |
| 65 | | | | | | | | | | | |
| 66 | | | | | | | | | | | |
| 67 | | | | | | | | | | | |
| 68 | | | | | | | | | | | |
| 69 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 71 | | | | | | | | | | | |
| 72 | | | | | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | | | | | | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | | | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | | | | | | | | | | | |
| 81 | | | | | | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | | | | | | | | | | | |
| 86 | | | | | | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | | | | | | | | |
| 89 | | | | | | | | | | | |
| 90 | | | | | | | | | | | |

Boring terminated at 61.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.01
Figure: A-5c

TEST GEOTECH LOG 315701_G.GPJ TR.GDT 1/12/05

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-5

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 6/29/01 Date finished: 7/1/01
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30-inches Hammer type: Safety, rope & pulley
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Osterberg (O)
Logged by: R. Nelson

Ground Surface Elevation: 3 feet[2]

| DEPTH (feet) | SAMPLES |  |  | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  | Sampler Type | Sample | SPT N-Value[1] |  |  | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|  |  |  |  |  | SANDY GRAVEL with RUBBLE (GP)<br>brown, loose, dry, with concrete and brick debris |  |  |  |  |  |  |
| 1 |  |  |  |  |  |  |  |  |  |  |  |
| 2 |  |  |  |  |  |  |  |  |  |  |  |
| 3 |  |  |  |  |  |  |  |  |  |  |  |
| 4 |  |  |  |  |  |  |  |  |  |  |  |
| 5 |  |  |  | GP |  |  |  |  |  |  |  |
| 6 |  |  |  |  |  |  |  |  |  |  |  |
| 7 |  |  |  |  | FILL |  |  |  |  |  |  |
| 8 |  |  |  |  |  |  |  |  |  |  |  |
| 9 |  |  |  |  |  |  |  |  |  |  |  |
| 10 |  |  |  |  |  |  |  |  |  |  |  |
| 11 |  |  |  |  | CONCRETE SLAB ~11-inches thick |  |  |  |  |  |  |
| 12 |  |  |  |  | CONCRETE |  |  |  |  |  |  |
| 13 |  |  |  |  |  |  |  |  |  |  |  |
| 14 |  |  |  |  |  |  |  |  |  |  |  |
| 15 |  |  |  |  | CLAYEY SAND/SANDY CLAY (SC/CH)<br>dark-gray, very loose/very soft to soft, wet, with shells |  |  |  |  |  |  |
| 16 |  |  |  |  |  |  |  |  |  |  |  |
| 17 |  |  |  |  |  |  |  |  |  |  |  |
| 18 |  |  |  |  |  |  |  |  |  |  |  |
| 19 |  |  |  |  |  |  |  |  |  |  |  |
| 20 |  |  |  |  |  |  |  |  |  |  |  |
| 21 | S&H | ▨ | 2 |  |  |  |  |  |  |  |  |
| 22 |  |  |  | SC-<br>CH |  |  |  |  |  |  |  |
| 23 |  |  |  |  |  |  |  |  |  |  |  |
| 24 |  |  |  |  |  |  |  |  |  |  |  |
| 25 |  |  |  |  |  |  |  |  |  |  |  |
| 26 |  |  |  |  |  |  |  |  |  |  |  |
| 27 |  |  |  |  |  |  |  |  |  |  |  |
| 28 |  |  |  |  |  |  |  |  |  |  |  |
| 29 |  |  |  |  |  |  |  |  |  |  |  |
| 30 |  |  |  |  |  |  |  |  |  |  |  |

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-5

PAGE 2 OF 4

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value$^1$ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA: Fines % | LABORATORY TEST DATA: Natural Moisture Content, % | LABORATORY TEST DATA: Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 31 | S&H | ▨ | 5 |  | CLAYEY SAND/SANDY CLAY (SC/CH) (continued)<br>loose/medium stiff |  |  |  |  | 23.6 | 101 |
| 32 |  |  |  |  |  |  |  |  |  |  |  |
| 33 |  |  |  |  |  |  |  |  |  |  |  |
| 34 |  |  |  | SC-CH |  |  |  |  |  |  |  |
| 35 |  |  |  |  |  |  |  |  |  |  |  |
| 36 |  |  |  |  |  |  |  |  |  |  |  |
| 37 |  |  |  |  |  |  |  |  |  |  |  |
| 38 |  |  |  |  | CLAYEY SAND (SC)<br>dark gray, medium dense, wet, with some fine gravel |  |  |  |  |  |  |
| 39 |  |  |  |  |  |  |  |  |  |  |  |
| 40 |  |  |  |  |  |  |  |  |  |  |  |
| 41 | S&H | ▨ | 25 | SC |  |  |  |  |  | 22.0 | 101 |
| 42 |  |  |  |  |  |  |  |  |  |  |  |
| 43 |  |  |  |  | SAND (SP)<br>green-gray, very dense, wet |  |  |  |  |  |  |
| 44 |  |  |  |  |  |  |  |  |  |  |  |
| 45 | S&H | ▨ | 30/4" |  |  |  |  |  |  |  |  |
| 46 |  |  |  |  |  |  |  |  |  |  |  |
| 47 |  |  |  |  |  |  |  |  |  |  |  |
| 48 |  |  |  |  |  |  |  |  |  |  |  |
| 49 |  |  |  |  |  |  |  |  |  |  |  |
| 50 |  |  |  |  | dense |  |  |  |  |  |  |
| 51 | SPT | ◩ | 42 | SP |  |  |  |  |  | 16.7 |  |
| 52 |  |  |  |  |  |  |  |  |  |  |  |
| 53 |  |  |  |  |  |  |  |  |  |  |  |
| 54 |  |  |  |  |  |  |  |  |  |  |  |
| 55 |  |  |  |  |  |  |  |  |  |  |  |
| 56 |  |  |  |  |  |  |  |  |  |  |  |
| 57 |  |  |  |  |  |  |  |  |  |  |  |
| 58 |  |  |  |  | CLAY with SAND (CL)<br>gray, medium stiff to stiff, wet |  |  |  |  |  |  |
| 59 |  |  |  | CL |  |  |  |  |  |  |  |
| 60 |  |  |  |  |  |  |  |  |  |  |  |

TEST GEOTECH LOG 315701 G.GPJ TR.GDT 1/12/05

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-6b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-5

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CLAY with SAND (CL) (continued) | | | | | | |
| 61 | SPT | ◪ | 8 | | | | | | | | |
| 62 | | | | | | | | | | | |
| 63 | | | | | | | | | | | |
| 64 | | | | CL | | | | | | | |
| 65 | | | | | | | | | | | |
| 66 | | | | | | | | | | | |
| 67 | | | | | | | | | | | |
| 68 | | | | | | | | | | | |
| 69 | | | | | CLAYEY SAND (SC)<br>green-gray, medium dense, wet | | | | | | |
| 70 | | | | | | | | | | | |
| 71 | S&H | ■ | 19 | | | | | | | 32.2 | 87 |
| 72 | | | | SC | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | | | | | SILTY SAND (SM)<br>yellow-brown, dense, wet | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | | | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | | | | | | | | | | | |
| 81 | S&H | ■ | 37 | SP | | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | | | | | | | | | | | |
| 86 | | | | | SAND (SP)<br>gray, very dense, wet | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | SP | | | | | | | |
| 89 | | | | | | | | | | | |
| 90 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-6c

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-5

| DEPTH (feet) | SAMPLES Sampler Type | SAMPLES Sample | SAMPLES SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA Type of Strength Test | LABORATORY TEST DATA Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA Fines % | LABORATORY TEST DATA Natural Moisture Content, % | LABORATORY TEST DATA Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  | SPT | ▨ | 50/6" |  | SAND (SP) (continued) |  |  |  |  |  |  |
| 91 |  |  |  |  |  |  |  |  |  |  |  |
| 92 |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  | SP |  |  |  |  |  |  |  |
| 93 |  |  |  |  |  |  |  |  |  |  |  |
| 94 |  |  |  |  |  |  |  |  |  |  |  |
| 95 |  |  |  |  | CLAYEY SAND (SC)<br>green-gray, very dense, wet |  |  |  |  |  |  |
| 96 |  |  |  |  |  |  |  |  |  |  |  |
| 97 |  |  |  |  |  |  |  |  |  |  |  |
| 98 |  |  |  | SC |  |  |  |  |  |  |  |
| 99 |  |  |  |  |  |  |  |  |  |  |  |
| 100 |  |  |  |  |  |  |  |  |  |  |  |
|  | SPT | ▨ | 50/3" |  |  |  |  |  |  |  |  |
| 101 |  |  |  |  |  |  |  |  |  |  |  |
| 102 |  |  |  |  |  |  |  |  |  |  |  |
| 103 |  |  |  |  |  |  |  |  |  |  |  |
| 104 |  |  |  |  |  |  |  |  |  |  |  |
| 105 |  |  |  |  |  |  |  |  |  |  |  |
| 106 |  |  |  |  |  |  |  |  |  |  |  |
| 107 |  |  |  |  |  |  |  |  |  |  |  |
| 108 |  |  |  |  |  |  |  |  |  |  |  |
| 109 |  |  |  |  |  |  |  |  |  |  |  |
| 110 |  |  |  |  |  |  |  |  |  |  |  |
| 111 |  |  |  |  |  |  |  |  |  |  |  |
| 112 |  |  |  |  |  |  |  |  |  |  |  |
| 113 |  |  |  |  |  |  |  |  |  |  |  |
| 114 |  |  |  |  |  |  |  |  |  |  |  |
| 115 |  |  |  |  |  |  |  |  |  |  |  |
| 116 |  |  |  |  |  |  |  |  |  |  |  |
| 117 |  |  |  |  |  |  |  |  |  |  |  |
| 118 |  |  |  |  |  |  |  |  |  |  |  |
| 119 |  |  |  |  |  |  |  |  |  |  |  |
| 120 |  |  |  |  |  |  |  |  |  |  |  |

Boring terminated at 101.0 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.

[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.01

Figure: A-6d

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-6

Boring location: See Site Plan, Figure 2
Logged by: R. Reindl
Date started: 5/12/04
Date finished: 5/13/04
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30-inches
Hammer type: Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | | LABORATORY TEST DATA Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Ground Surface Elevation: +4 feet[2] | | | | | | | |
| | | | | | GRAVEL with SAND (GP)<br>gray brown, dry, with concrete and brick debris | | | | | | | |
| 1 | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | |
| 3 | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | |
| 5 | | | | GP | | | | | | | | |
| 6 | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | |
| 8 | | | | | | FILL | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | 6-feet-thick Concrete Slab | | | | | | | |
| 12 | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | |
| 16 | | | | | | | | | | | | |
| 17 | | | | | CLAY (CH)<br>gray, soft, wet, with shells, sand and silt | | | | | | | |
| 18 | | | | | | | | | | | | |
| 19 | | | | | | | | | | | | |
| 20 | | | | | | | | | | | | |
| 21 | | | | | | | | | | | | |
| 22 | | | | | | | | | | | | |
| 23 | | | | CH | | | | | | | | |
| 24 | | | | | | | | | | | | |
| 25 | | | | | | | | | | | | |
| 26 | S&H | ▨ | 2 | | | | | | | | | |
| 27 | | | | | | | | | | | | |
| 28 | | | | | | | | | | | | |
| 29 | | | | | | | | | | | | |
| 30 | | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.02
Figure: A-7a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-6

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CLAY (CH) (continued) | | | | | | |
| 31 | | | | | | | | | | | |
| 32 | | | | | | | | | | | |
| 33 | | | | | | | | | | | |
| 34 | | | | | | | | | | | |
| 35 | | | | | | | | | | | |
| 36 | | | | | | | | | | | |
| 37 | | | | CH | | | | | | | |
| 38 | | | | | sandy, gravelly cuttings from 38 to 42 feet | | | | | | |
| 39 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |
| 41 | | | | | | | | | | | |
| 42 | | | | | less sand and gravel in cuttings from 42 to 45 feet | | | | | | |
| 43 | | | | | | | | | | | |
| 44 | | | | | | | | | | | |
| 45 | SPT | ◩ | 50/ 6" | | SAND (SP)<br>gray, very dense, wet, fine grained | | | | | | |
| 46 | | | | | | | | | | | |
| 47 | | | | | | | | | | | |
| 48 | | | | | | | | | | | |
| 49 | | | | | | | | | | | |
| 50 | | | | | | | | | | | |
| 51 | | | | | | | | | | | |
| 52 | | | | SP | | | | | | | |
| 53 | | | | | | | | | | | |
| 54 | | | | | | | | | | | |
| 55 | | | | | | | | | | | |
| 56 | | | | | | | | | | | |
| 57 | | | | | | | | | | | |
| 58 | | | | | | | | | | | |
| 59 | | | | CH | CLAY (CH) | | | | | | |
| 60 | | | | | | | | | | | |

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA: Fines % | LABORATORY TEST DATA: Natural Moisture Content, % | LABORATORY TEST DATA: Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 61 | S&H | ■ | 2 | | CLAY (CH) (continued)<br>gray, soft, wet | | | | | | |
| 62 | | | | | | | | | | | |
| 63 | | | | | | | | | | | |
| 64 | | | | | | | | | | | |
| 65 | | | | CH | | | | | | | |
| 66 | | | | | | | | | | | |
| 67 | | | | | | | | | | | |
| 68 | | | | | | | | | | | |
| 69 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 71 | | | | | SAND (SP)<br>gray, very dense, moist, fine grained | | | | | | |
| 72 | | | | | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | SPT | ◪ | 65 | | | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | | | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | | | | SP | | | | | | | |
| 81 | | | | | | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | | | | | | | | | | | |
| 86 | | | | | | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | | | | | | | | |
| 89 | | | | | | | | | | | |
| 90 | | | | | | | | | | | |

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-6

| DEPTH (feet) | SAMPLES Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SAND (SP) (continued) | | | | | | |
| 91 | | | | | | | | | | | |
| 92 | | | | SP | grades with clay and dark brown organics | | | | | | |
| 93 | | | | | | | | | | | |
| 94 | | ▨ | | | SAND (SP)<br>gray, dense, wet, fine grained | | | | | | |
| 95 | S&H | ▨ | 33 | SP | | | | | | | |
| 96 | | | | | CLAY (CH)<br>dark gray, stiff, wet | | | | | | |
| 97 | | | | | | | | | | | |
| 98 | | | | CH | | | | | | | |
| 99 | | | | | | | | | | | |
| 100 | | | | | CLAY with SAND (CH)<br>greenish-gray, stiff, wet, with trace of sand | | | | | | |
| 101 | | | | | | | | | | | |
| 102 | | | | | | | | | | | |
| 103 | | | | | | | | | | | |
| 104 | | ▨ | 100 psi to 350 psi | | | | | | | | |
| 105 | ST | ▨ | | | | TV | | 400 | | | |
| 106 | | ▨ | | | | | | | | | |
| 107 | | | | | | | | | | | |
| 108 | | | | | OLD BAY CLAY | | | | | | |
| 109 | | | | | | | | | | | |
| 110 | | ▨ | 100 to 200 psi | CH | | | | | | | |
| 111 | ST | ▨ | | | | TV | | 1,500 | | 40.3 | 82 |
| 112 | | ▨ | | | | | | | | | |
| 113 | | | | | | | | | | | |
| 114 | | | | | | | | | | | |
| 115 | | | | | | | | | | | |
| 116 | | | | | | | | | | | |
| 117 | | | | | | | | | | | |
| 118 | | | | | | | | | | | |
| 119 | | | | | | | | | | | |
| 120 | | | | | | | | | | | |

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-6

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA: Fines % | LABORATORY TEST DATA: Natural Moisture Content, % | LABORATORY TEST DATA: Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 100 to 200 psi | | CLAY with SAND (CH) (continued) | | | | | | |
| 121 | ST | | | | | | | | | 42.9 | 78 |
| 122 | | | | | | TV | | 1,400 | | | |
| 123 | | | | | dark gray, trace organics, no sand | | | | | | |
| 124 | | | | | | | | | | | |
| 125 | | | | | | | | | | | |
| 126 | | | | | | | | | | | |
| 127 | | | | | | | | | | | |
| 128 | | | | | | | | | | | |
| 129 | | | | | | | | | | | |
| 130 | | | 100 to 200 psi | | | | | | | | |
| 131 | ST | | | | Consolidation Test, See Figure B-1 | | | | | 42.3 | 79 |
| 132 | | | | | | TV | | 1,400 | | | |
| 133 | | | | | | | | | | | |
| 134 | | | | | OLD BAY CLAY | | | | | | |
| 135 | | | | CH | | | | | | | |
| 136 | | | | | | | | | | | |
| 137 | | | | | | | | | | | |
| 138 | | | | | | | | | | | |
| 139 | | | | | | | | | | | |
| 140 | | | 100 to 200 psi | | | | | | | | |
| 141 | ST | | | | | | | | | 41.6 | 82 |
| 142 | | | | | | TV | | 1,600 | | | |
| 143 | | | | | | | | | | | |
| 144 | | | | | | | | | | | |
| 145 | | | | | | | | | | | |
| 146 | | | | | | | | | | | |
| 147 | | | | | | | | | | | |
| 148 | | | | | | | | | | | |
| 149 | | | | | | | | | | | |
| 150 | | | | | | | | | | | |

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-6

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Sample | SAMPLES: SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft | LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft | LABORATORY TEST DATA: Fines % | LABORATORY TEST DATA: Natural Moisture Content, % | LABORATORY TEST DATA: Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ▨ | 100 to 225 psi | | CLAY with SAND (CH) (continued) | | | | | | |
| 151 | ST | ▨ | | | | | | | | | |
| 152 | | ▨ | | | | TV | | 1,700 | | | |
| 153 | | | | | | | | | | | |
| 154 | | | | | | | | | | | |
| 155 | | | | | | | | | | | |
| 156 | | | | | | | | | | | |
| 157 | | | | | | | | | | | |
| 158 | | | | | | | | | | | |
| 159 | | | | | | | | | | | |
| 160 | | ▨ | 100 to 200 psi | | | | | | | | |
| 161 | ST | ▨ | | | | | | | | | |
| 162 | | ▨ | | | | TV | | 2,000 | | | |
| 163 | | | | | | | | | | | |
| 164 | | | | | | | | | | | |
| 165 | | | | CH | OLD BAY CLAY | | | | | | |
| 166 | | | | | | | | | | | |
| 167 | | | | | | | | | | | |
| 168 | | | | | | | | | | | |
| 169 | | | | | | | | | | | |
| 170 | | ▨ | 100 to 250 psi | | | | | | | | |
| 171 | ST | ▨ | | | Consolidatation Test, See Figure B-2 | | | | | 45.3 | 76 |
| 172 | | ▨ | | | | TV | | 2,700 | | | |
| 173 | | | | | | | | | | | |
| 174 | | | | | sand lense | | | | | | |
| 175 | | | | | | | | | | | |
| 176 | | | | | | | | | | | |
| 177 | | | | | | | | | | | |
| 178 | | | | | green gray, hard, wet, trace sand and organics | | | | | | |
| 179 | | | | | | | | | | | |
| 180 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-7f

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-6

PAGE 7 OF 7

| | SAMPLES | | | | | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |
| | ST | | 100 to 400 psi | | CLAY (CL) (continued) | | | | | 36.9 | 87 |
| 181 | | | | | very stiff | TV | | 2,250 | | | |
| 182 | | | | | | | | | | | |
| 183 | | | | | | | | | | | |
| 184 | | | | | | | | | | | |
| 185 | | | | | | | | | | | |
| 186 | | | | | | | | | | | |
| 187 | | | | | | | | | | | |
| 188 | | | | | | | | | | | |
| 189 | | | | CH | OLD BAY CLAY | | | | | | |
| 190 | | | | | | | | | | | |
| 191 | S&H | | 36 | | hard | | | | | 33.3 | 89 |
| 192 | | | | | | | | | | | |
| 193 | | | | | | | | | | | |
| 194 | | | | | | | | | | | |
| 195 | | | | | | | | | | | |
| 196 | | | | | | | | | | | |
| 197 | | | | | | | | | | | |
| 198 | | | | | | | | | | | |
| 199 | | | | CL | CLAY (CL)<br>dark brown, hard, wet | | | | | | |
| 200 | S&H | | 30/ 3.5" | SP | SAND (SP)<br>dark brown, very dense, wet | | | | | | |
| 201 | | | | | | | | | | | |
| 202 | | | | | | | | | | | |
| 203 | | | | | | | | | | | |
| 204 | | | | | | | | | | | |
| 205 | | | | | | | | | | | |
| 206 | | | | | | | | | | | |
| 207 | | | | | | | | | | | |
| 208 | | | | | | | | | | | |
| 209 | | | | | | | | | | | |
| 210 | | | | | | | | | | | |

Boring terminated at 200.75 feet below ground surface.
Boring backfilled with cement grout under the observation of the SFDPH.
Groundwater level was obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.02
Figure: A-7g

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

PAGE 1 OF 8

Boring location: See Site Plan, Figure 2

Logged by: L. Bedolla

Date started: 5/14/04

Date finished: 5/17/04

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches

Hammer type: Safety

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

LABORATORY TEST DATA

| DEPTH (feet) | SAMPLES Sampler Type | SAMPLES Sample | SAMPLES SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION<br>Ground Surface Elevation: +4 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  | SAND with GRAVEL (SP)<br>gray brown, loose, dry, with brick and concrete |  |  |  |  |  |  |  |
| 1 |  |  |  |  |  |  |  |  |  |  |  |  |
| 2 |  |  |  |  |  |  |  |  |  |  |  |  |
| 3 |  |  |  |  |  |  |  |  |  |  |  |  |
| 4 |  |  |  |  |  |  |  |  |  |  |  |  |
| 5 |  |  |  | SP |  |  |  |  |  |  |  |  |
| 6 |  |  |  |  |  |  |  |  |  |  |  |  |
| 7 |  |  |  |  |  |  |  |  |  |  |  |  |
| 8 |  |  |  |  |  |  |  |  |  |  |  |  |
| 9 |  |  |  |  |  | FILL |  |  |  |  |  |  |
| 10 |  |  |  |  |  |  |  |  |  |  |  |  |
| 11 |  |  |  |  | 12-inches-thick Concrete Slab |  |  |  |  |  |  |  |
| 12 |  |  |  |  | SILTY SAND (SM)<br>dark gray, medium dense, wet, with brick |  |  |  |  |  |  |  |
| 13 |  |  |  |  |  |  |  |  |  |  |  |  |
| 14 |  |  |  | SM |  |  |  |  |  |  |  |  |
| 15 |  | ▨ |  |  |  |  |  |  |  |  |  |  |
| 16 | S&H |  | 11 |  |  |  |  |  |  |  |  |  |
| 17 |  |  |  |  | CLAY (CH)<br>black, soft to medium stiff, wet, with rubble and organics |  |  |  |  |  |  |  |
| 18 |  |  |  | CH |  |  |  |  |  |  |  |  |
| 19 |  |  |  |  | CLAY (CH)<br>gray, soft to medium stiff, wet, trace sand and shells |  |  |  |  |  |  |  |
| 20 |  |  |  |  |  |  |  |  |  |  |  |  |
| 21 |  |  |  |  |  |  |  |  |  |  |  |  |
| 22 |  |  |  |  |  |  |  |  |  |  |  |  |
| 23 |  |  |  |  |  |  |  |  |  |  |  |  |
| 24 |  |  |  | CH |  |  |  |  |  |  |  |  |
| 25 |  |  |  |  |  |  |  |  |  |  |  |  |
| 26 |  |  |  |  |  |  |  |  |  |  |  |  |
| 27 |  |  |  |  |  |  |  |  |  |  |  |  |
| 28 |  |  |  |  |  |  |  |  |  |  |  |  |
| 29 |  |  |  |  |  |  |  |  |  |  |  |  |
| 30 |  |  |  |  |  |  |  |  |  |  |  |  |

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-8a

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

| | SAMPLES | | | | | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |
| | | | | | CLAY (CH) (continued) | | | | | | |
| 31 | | | | | | | | | | | |
| 32 | | | | | | | | | | | |
| 33 | | | | CH | | | | | | | |
| 34 | | | | | | | | | | | |
| 35 | | | | | | | | | | | |
| 36 | ST | | 100 to 200 psi | | SILTY SAND (SM)<br>gray, medium dense, wet | | | | | | |
| 37 | | | | | | | | | | | |
| 38 | | | | SM | | | | | | | |
| 39 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |
| 41 | | | | | CLAY with SAND (CH)<br>gray, medium stiff, wet, trace shells | | | | | | |
| 42 | | | | | | | | | | | |
| 43 | | | | | | | | | | | |
| 44 | | | | | | | | | | | |
| 45 | | | | | no sand | | | | | | |
| 46 | | | | | | | | | | | |
| 47 | | | | | | | | | | | |
| 48 | | | | | with sand | | | | | | |
| 49 | | | | | | | | | | | |
| 50 | | | | CH | | | | | | | |
| 51 | | | | | | | | | | | |
| 52 | | | | | | | | | | | |
| 53 | | | | | | | | | | | |
| 54 | | | | | | | | | | | |
| 55 | | | | | | | | | | | |
| 56 | S&H | | 4 | | | TV | | 800 | | | |
| 57 | | | | | | | | | | | |
| 58 | | | | | | | | | | | |
| 59 | | | | | | | | | | | |
| 60 | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-8b

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

| DEPTH (feet) | SAMPLES<br>Sampler Type | Sample | SPT N-Value¹ | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA<br>Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SANDY CLAY (CH)<br>dark gray to black, medium stiff to stiff, wet | | | | | | |
| 61 | | | | | | | | | | | |
| 62 | | | | | | | | | | | |
| 63 | | | | CH | | | | | | | |
| 64 | | | | | | | | | | | |
| 65 | | | | | SANDY CLAY (CL)<br>green gray, stiff, wet | | | | | | |
| 66 | | | | CL | | | | | | | |
| 67 | | | | | | | | | | | |
| 68 | | | | | SILTY SAND (SM)<br>yellow brown, dense, wet, pockets of clayey sand and cemented sand | | | | | | |
| 69 | | | | | | | | | | | |
| 70 | S&H | ■ | 33 | | gray<br>yellow brown | | | | | | |
| 71 | | | | | | | | | | | |
| 72 | | | | | | | | | | | |
| 73 | | | | | | | | | | | |
| 74 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 76 | | | | | | | | | | | |
| 77 | | | | | | | | | | | |
| 78 | | | | SM | | | | | | | |
| 79 | | | | | | | | | | | |
| 80 | S&H | ■ | 30/ 6" | | | | | | | | |
| 81 | | | | | very dense | | | | | | |
| 82 | | | | | | | | | | | |
| 83 | | | | | | | | | | | |
| 84 | | | | | | | | | | | |
| 85 | | | | | | | | | | | |
| 86 | | | | | | | | | | | |
| 87 | | | | | | | | | | | |
| 88 | | | | | CLAY with SAND (CL)<br>olive gray, medium stiff to stiff, wet<br>OLD BAY CLAY | | | | | | |
| 89 | | | | CL | | | | | | | |
| 90 | | | | | | | | | | | |

TEST GEOTECH LOG 315702.GPJ TR.GDT 1/12/05

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-8c

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

| | SAMPLES | | | | | | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (feet) | Sampler Type | Sample | SPT N-Value¹ | 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 | ST | | 100 to 400 psi | | CLAY with SAND (CL) | | | | | | | |
| 92 | | | | | SILTY SAND (SM)<br>dark gray, medium dense to dense, wet | | | | | | | |
| 93 | | | | SM | | | | | | | | |
| 94 | | | | | SANDY CLAY (CL)<br>olive gray, stiff, wet | | | | | | | |
| 95 | | | | CL | | | | | | | | |
| 96 | | | | | | | | | | | | |
| 97 | | | | | CLAY (CL)<br>dark gray, stiff, wet | | | | | | | |
| 98 | | | | | | | | | | | | |
| 99 | | | | | | | | | | | | |
| 100 | | | | | | | | | | | | |
| 101 | ST | | 100 to 200 psi | | with silt and fine sand | | | | | | 33.7 | 90 |
| 102 | | | | | | | TV | | 950 | | | |
| 103 | | | | | | | | | | | | |
| 104 | | | | | | | | | | | | |
| 105 | | | | | | OLD BAY CLAY | | | | | | |
| 106 | | | | | | | | | | | | |
| 107 | | | | | | | | | | | | |
| 108 | | | | CL | | | | | | | | |
| 109 | | | | | | | | | | | | |
| 110 | | | | | | | | | | | | |
| 111 | ST | | 100 to 180 psi | | less silt and no fine sand | | | | | | 40.2 | 80 |
| 112 | | | | | Consolidation Test, See Figure B-3 | | TV | | 800 | | | |
| 113 | | | | | | | | | | | | |
| 114 | | | | | | | | | | | | |
| 115 | | | | | | | | | | | | |
| 116 | | | | | | | | | | | | |
| 117 | | | | | | | | | | | | |
| 118 | | | | | | | | | | | | |
| 119 | | | | | | | | | | | | |
| 120 | | | | | | | | | | | | |

TEST GEOTECH LOG 315702.GPJ TR.GDT 1/12/05

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

PAGE 5 OF 8

| DEPTH (feet) | SAMPLES<br>Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA<br>Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CLAY (CL) (continued)<br>OLD BAY CLAY | | | | | | |
| 121 | ST | | 100 to 200 psi | | | | | | | 41.3 | 81 |
| 122 | | | | | | TV | | 900 | | | |
| 123 | | | | | | | | | | | |
| 124 | | | | | | | | | | | |
| 125 | | | | | | | | | | | |
| 126 | | | | | | | | | | | |
| 127 | | | | | | | | | | | |
| 128 | | | | | | | | | | | |
| 129 | | | | | | | | | | | |
| 130 | | | | | | | | | | | |
| 131 | ST | | 100 to 200 psi | | | | | | | 43.1 | 79 |
| 132 | | | | | | TV | | 1,200 | | | |
| 133 | | | | | | | | | | | |
| 134 | | | | | | | | | | | |
| 135 | | | | | | | | | | | |
| 136 | | | | | | | | | | | |
| 137 | | | | | | | | | | | |
| 138 | | | | | | | | | | | |
| 139 | | | | | | | | | | | |
| 140 | | | | | | | | | | | |
| 141 | ST | | 100 to 200 psi | | | | | | | | |
| 142 | | | | | | TV | | 1,300 | | | |
| 143 | | | | | | | | | | | |
| 144 | | | | | | | | | | | |
| 145 | | | | | | | | | | | |
| 146 | | | | | | | | | | | |
| 147 | | | | | | | | | | | |
| 148 | | | | | | | | | | | |
| 149 | | | | | | | | | | | |
| 150 | | | | | | | | | | | |

TEST GEOTECH LOG 315702.GPJ TR.GDT 1/12/05

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-8e

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

| DEPTH (feet) | SAMPLES: Sampler Type | Sample | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  | ▨ | 100 to 225 psi |  | CLAY (CL) (continued) |  |  |  |  |  |  |
| 151 | ST | ▨ |  |  | Consolidation Test, See Figure B-4 |  |  |  |  | 42.4 | 78 |
| 152 |  | ▨ |  |  |  | TV |  | 1,500 |  |  |  |
| 153 |  |  |  |  |  |  |  |  |  |  |  |
| 154 |  |  |  |  |  |  |  |  |  |  |  |
| 155 |  |  |  |  |  |  |  |  |  |  |  |
| 156 |  |  |  |  |  |  |  |  |  |  |  |
| 157 |  |  |  |  |  |  |  |  |  |  |  |
| 158 |  |  |  |  |  |  |  |  |  |  |  |
| 159 |  |  |  |  |  |  |  |  |  |  |  |
| 160 |  | ▨ | 100 to 200 psi |  |  |  |  |  |  |  |  |
| 161 | ST | ▨ |  |  |  |  |  |  |  | 44.1 | 77 |
| 162 |  | ▨ |  |  |  | TV |  | 1,900 |  |  |  |
| 163 |  |  |  |  |  |  |  |  |  |  |  |
| 164 |  |  |  |  |  |  |  |  |  |  |  |
| 165 |  |  |  | CL | OLD BAY CLAY |  |  |  |  |  |  |
| 166 |  |  |  |  |  |  |  |  |  |  |  |
| 167 |  |  |  |  |  |  |  |  |  |  |  |
| 168 |  |  |  |  |  |  |  |  |  |  |  |
| 169 |  |  |  |  |  |  |  |  |  |  |  |
| 170 |  | ▨ | 100 to 200 psi |  | green gray, very stiff, wet, trace sand and organics |  |  |  |  |  |  |
| 171 | ST | ▨ |  |  |  |  |  |  |  |  |  |
| 172 |  | ▨ |  |  |  | TV |  | 2,200 |  |  |  |
| 173 |  |  |  |  |  |  |  |  |  |  |  |
| 174 |  |  |  |  |  |  |  |  |  |  |  |
| 175 |  |  |  |  |  |  |  |  |  |  |  |
| 176 |  |  |  |  |  |  |  |  |  |  |  |
| 177 |  |  |  |  |  |  |  |  |  |  |  |
| 178 |  |  |  |  |  |  |  |  |  |  |  |
| 179 |  |  |  |  |  |  |  |  |  |  |  |
| 180 |  |  |  |  |  |  |  |  |  |  |  |

TEST GEOTECH LOG 315702.GPJ TR.GDT 1/12/05

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

| DEPTH (feet) | SAMPLES | | | LITHOLOGY | MATERIAL DESCRIPTION | | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sampler Type | Sample | SPT N-Value$^1$ | | | | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
| 181 | ST | | 150 to 200 psi | | CLAY (CL) (continued)<br>gray | OLD BAY CLAY | | | | | | |
| 182 | | | | | | | TV | | 2,700 | | | |
| 183 | | | | | | | | | | | | |
| 184 | | | | | | | | | | | | |
| 185 | | | | | | | | | | | | |
| 186 | | | | | | | | | | | | |
| 187 | | | | | | | | | | | | |
| 188 | | | | | | | | | | | | |
| 189 | | | | | | | | | | | | |
| 190 | | | | CL | | | | | | | | |
| 191 | ST | | 0 to 150 psi | | Consolidation Test, See Figure B-5 | | | | | | 36.7 | 85 |
| 192 | | | | | | | TV | | 2,400 | | | |
| 193 | | | | | | | | | | | | |
| 194 | | | | | | | | | | | | |
| 195 | | | | | | | | | | | | |
| 196 | | | | | | | | | | | | |
| 197 | | | | | | | | | | | | |
| 198 | | | | | | | | | | | | |
| 199 | | | | | | | | | | | | |
| 200 | | | | | SANDY CLAY (CL)<br>gray brown, very stiff, wet, trace organics | | | | | | | |
| 201 | ST | | 0 to 300 psi | | | | | | | | | |
| 202 | | | | | | | TV | | 2,300 | | | |
| 203 | | | | | | | | | | | | |
| 204 | | | | CL | | | | | | | | |
| 205 | | | | | | | | | | | | |
| 206 | | | | | | | | | | | | |
| 207 | | | | | | | | | | | | |
| 208 | | | | | SILTY SAND (SM)<br>gray, very dense, wet, trace organics | | | | | | | |
| 209 | | | | SM | | | | | | | | |
| 210 | | | | | | | | | | | | |

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-8g

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring B-7

PAGE 8 OF 8

| | SAMPLES | | | | | LABORATORY TEST DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |
| | S&H | ▨ | 30/ 3" | | SILTY SAND (SM) (continued) | | | | | | |
| 211 | | | | | | | | | | | |
| 212 | | | | | | | | | | | |
| 213 | | | | SM | | | | | | | |
| 214 | | | | | | | | | | | |
| 215 | | | | | | | | | | | |
| 216 | | | | | | | | | | | |
| 217 | | | | | CLAY (CL)<br>gray, hard, wet | | | | | | |
| 218 | | | | CL | | | | | | | |
| 219 | | | | | | | | | | | |
| 220 | | | | | | | | | | | |
| 221 | | | | | | | | | | | |
| 222 | | | | | | | | | | | |
| 223 | | | | | | | | | | | |
| 224 | | | | | | | | | | | |
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| 226 | | | | | | | | | | | |
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| 228 | | | | | | | | | | | |
| 229 | | | | | | | | | | | |
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| 231 | | | | | | | | | | | |
| 232 | | | | | | | | | | | |
| 233 | | | | | | | | | | | |
| 234 | | | | | | | | | | | |
| 235 | | | | | | | | | | | |
| 236 | | | | | | | | | | | |
| 237 | | | | | | | | | | | |
| 238 | | | | | | | | | | | |
| 239 | | | | | | | | | | | |
| 240 | | | | | | | | | | | |

Boring terminated at 220 feet below ground surface.
Boring backfilled with cement grout under the observation of the SFDPH.
Groundwater level was obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.

[2] Elevations based on San Francisco City datum.

**Treadwell&Rollo**

Project No.: 3157.02

Figure: A-8h

## UNIFIED SOIL CLASSIFICATION SYSTEM

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

## GRAIN SIZE CHART

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

| | |
|---|---|
| ▽ | Unstabilized groundwater level |
| ▼ | Stabilized groundwater level |

## SAMPLE DESIGNATIONS/SYMBOLS

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

## 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 |

**301 MISSION STREET**
San Francisco, California

**Treadwell&Rollo**

## CLASSIFICATION CHART

Date 01/12/05 | Project No. 3157.02 | Figure A-9

# APPENDIX B

# APPENDIX B

## Environmental Boring Logs

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring TR-1

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: C. Keane

Date started: 7/5/01

Date finished: 7/5/01

Drilling method: Hand Auger

Hammer weight/drop: ---

Hammer type: ---

Sampler: ---

| DEPTH (feet) | SAMPLES Sample Number | Sample | Blow Count | Recovery (inches) | OVM (ppm) | LITHOLOGY | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|---|
| | | | | | | | CONCRETE SLAB |
| | | | | | | | Concrete core to 6-inches, rubber membrane 1/4" thick, second concrete slab to total of 13-1/2" |
| 1 | | | | | | SP | SILTY SAND<br>brown, moist, with brick fragments **FILL** |
| 2 | | | | | | SP | ∇ SAND<br>grey, wet<br>Groundwater encountered at 3 feet |
| 3 | TR-1-3.5 | ▨ | | | | | |
| 4 | TR-1-4.0 | ▨ | | | | | |
| 5 | | | | | | | |
| 6 | | | | | | | |
| 7 | | | | | | | |
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| 26 | | | | | | | |
| 27 | | | | | | | |
| 28 | | | | | | | |
| 29 | | | | | | | |
| 30 | | | | | | | |

Boring terminated at 4.0 feet.
Boring backfilled with bentonite grout mix.
Groundwater encountered at 3.0 feet.

**Treadwell&Rollo**

Project No.: 3157.01

Figure: B-1

TEST ENVIRONMENTAL 315701 E.GPJ T&R.GDT 1/11/05

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring TR-2

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Logged by: C. Keane
Date started: 7/5/01
Date finished: 7/5/01
Drilling method: Hand Auger
Hammer weight/drop: ---
Hammer type: ---
Sampler: ---

| DEPTH (feet) | SAMPLES | | | | OVM (ppm) | LITHOLOGY | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|---|
| | Sample Number | Sample | Blow Count | Recovery (inches) | | | |
| | | | | | | | CONCRETE SLAB |
| | | | | | | | 16-inch concrete slab |
| 1 | | | | | | | |
| 2 | | | | | | SP | ▽ SILTY SAND<br>brown, loose, with brick fragments **FILL** |
| 3 | TR-2-3.5 | ▨ | | | | SP | SAND<br>grey, loose, wet, fine-grained<br>groundwater encountered at 2 feet. |
| 4 | TR-2-4.0 | ▨ | | | | | |
| 5 | | | | | | | |
| 6 | | | | | | | |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
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| 27 | | | | | | | |
| 28 | | | | | | | |
| 29 | | | | | | | |
| 30 | | | | | | | |

Boring terminated at 4.0 feet.
Boring backfilled with bentonite grout mix.
Groundwater encountered at 2.0 feet.

**Treadwell&Rollo**

Project No.: 3157.01
Figure: B-2

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring TR-3

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 7/5/01
Date finished: 7/5/01
Drilling method: Hand Auger
Logged by: C. Keane
Hammer weight/drop: ---
Hammer type: ---
Sampler: ---

| DEPTH (feet) | SAMPLES<br>Sample Number | Sample | Blow Count | Recovery (inches) | OVM (ppm) | LITHOLOGY | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|---|
| | | | | | | | CONCRETE SLAB |
| | | | | | | | 10-inch layer of concrete |
| 1 | | | | | | SP | SILTY SAND<br>brown, moist, with brick fragments **FILL** |
| 2 | | | | | | SP | SAND<br>grey, dense, dry, trace of clayey sand |
| 3 | TR-3-3.5 | ▨ | | | | | |
| 4 | TR-3-4.0 | ▨ | | | | | |
| 5 | | | | | | | |
| 6 | | | | | | | |
| 7 | | | | | | | |
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| 29 | | | | | | | |
| 30 | | | | | | | |

Boring terminated at 4.0 feet.
Boring backfilled with bentonite grout.
Groundwater not encountered during drilling.

**Treadwell&Rollo**

Project No.: 3157.01
Figure: B-3

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring TR-4

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Logged by: C. Keane
Date started: 7/5/01
Date finished: 7/5/01
Drilling method: Hand Auger
Hammer weight/drop: ---
Hammer type: ---
Sampler: ---

| DEPTH (feet) | SAMPLES<br>Sample Number | Sample | Blow Count | Recovery (inches) | OVM (ppm) | LITHOLOGY | MATERIAL DESCRIPTION<br>CONCRETE SLAB |
|---|---|---|---|---|---|---|---|
| | | | | | | | 8-inch concrete slab |
| 1 | | | | | | | SAND<br>brown, then grey after 1-foot, loose, moist |
| 2 | | | | | | SP | |
| 3 | TR-4-3.0<br>TR-4-3.5 | ■ | | | | | ▽ |
| 4 | | | | | | | |
| 5 | | | | | | | |
| 6 | | | | | | | |
| 7 | | | | | | | |
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| 29 | | | | | | | |
| 30 | | | | | | | |

Boring terminated at 3.5 feet.
Boring backfilled with bentonite grout.
Groundwater encountered at 3.0 feet.

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Project No.: 3157.01
Figure: B-4

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring TR-5

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Logged by: C. Keane
Date started: 7/5/01
Date finished: 7/5/01
Drilling method: Hand Auger
Hammer weight/drop: ---
Hammer type: ---
Sampler: ---

| DEPTH (feet) | SAMPLES<br>Sample Number | Sample | Blow Count | Recovery (inches) | OVM (ppm) | LITHOLOGY | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|---|
| | | | | | | | CONCRETE SLAB |
| | | | | | | | 6-inch concrete slab |
| 1 | | | | | | SP | SILTY SAND<br>light-brown, moist, with brick fragments **FILL** |
| 2 | | | | | | | |
| 3 | TR-5-3.0<br>TR-5-3.5 | ▨<br>▨ | | | | SP | SAND<br>▽ grey, dense, wet, fine-grained, poorly-graded |
| 4 | | | | | | | |
| 5 | | | | | | | |
| 6 | | | | | | | |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
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| 29 | | | | | | | |
| 30 | | | | | | | |

Boring terminated at 3.5 feet.
Boring backfilled with bentonite grout.
Groundwater encountered at 3.5 feet.

**Treadwell&Rollo**

Project No.: 3157.01
Figure: B-5

PROJECT: **301 MISSION STREET**
San Francisco, California

# Log of Boring TR-6

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: C. Keane

Date started: 7/5/01

Date finished: 7/5/01

Drilling method: Hand Auger

Hammer weight/drop: ---

Hammer type: ---

Sampler: ---

| DEPTH (feet) | SAMPLES Sample Number | Sample | Blow Count | Recovery (inches) | OVM (ppm) | LITHOLOGY | MATERIAL DESCRIPTION | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | CONCRETE SLAB | |
| | | | | | | | 6-inch concrete slab | |
| 1 | | | | | | | SAND<br>dark brown, loose, dry, fine-grained, poorly-graded with red brick | |
| 2 | | | | | | | | |
| 3 | | | | | | | | |
| 4 | | | | | | SP | | FILL |
| 5 | | | | | | | black coal waste | |
| 6 | | | | | | | porcelain | |
| 7 | | | | | | | wood pieces | |
| 8 | TR-6-8.0 | ■ | | | | | | |
| 9 | | | | | | | | |
| 10 | | | | | | | | |
| 11 | | | | | | | | |
| 12 | | | | | | | | |
| 13 | | | | | | | | |
| 14 | | | | | | | | |
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| 28 | | | | | | | | |
| 29 | | | | | | | | |
| 30 | | | | | | | | |

Borehole keeps collapsing in itself. Further sampling is not possible.
Boring terminated at 8.0 feet.
Boring backfilled with bentonite grout mix.
Groundwater not encountered during drilling.

**Treadwell&Rollo**

Project No.: 3157.01

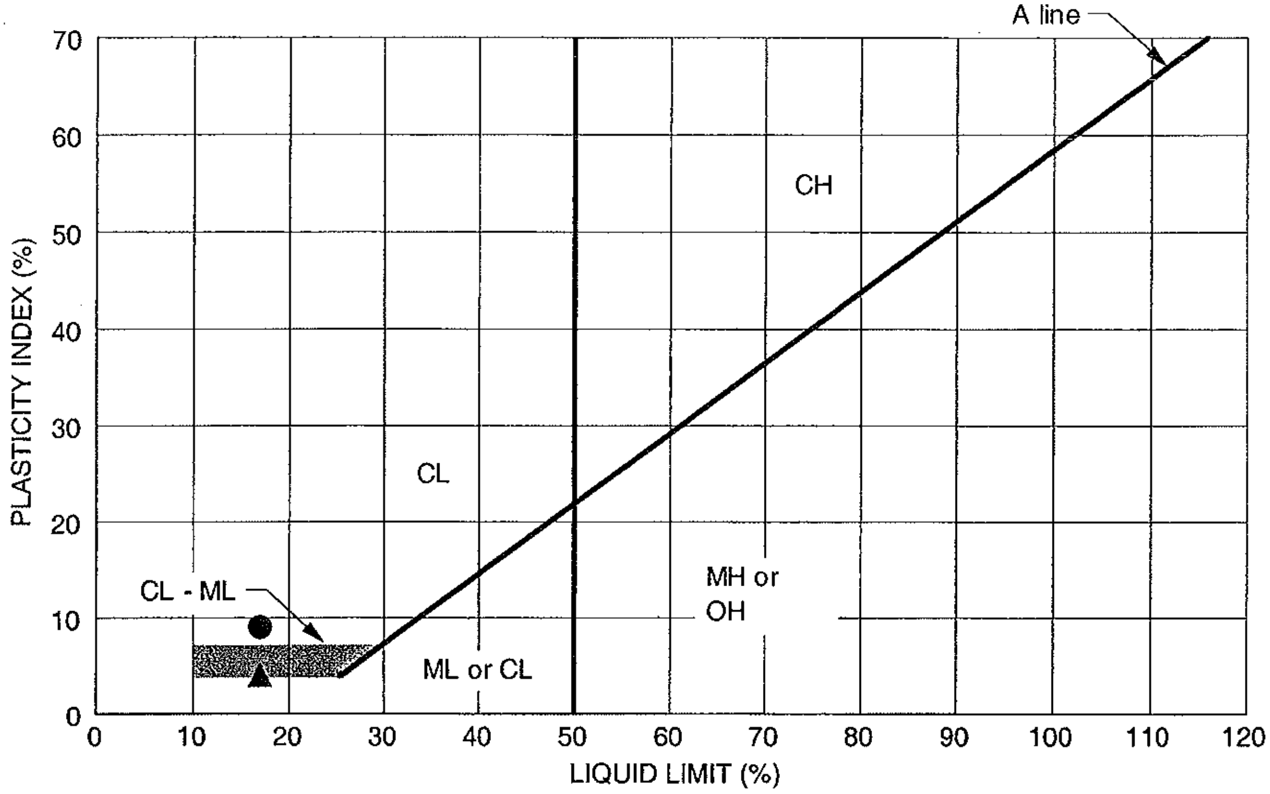
Figure: B-6

TEST ENVIRONMENTAL 315701_E.GPJ T&R.GDT 1/11/05

# APPENDIX C

# APPENDIX C

## Laboratory Test Data



| Symbol | Source | Description and Classification | Natural M.C. (%) | Liquid Limit (%) | Plasticity Index (%) | % Passing #200 Sieve |
|---|---|---|---|---|---|---|
| ● | B-1 at 50 feet | CLAYEY SAND (SC), gray | 24.1 | 17 | 9 | 19 |
| ▲ | B-4 at 50 feet | SILTY SAND (SM), gray | 27.7 | 17 | 4 | 21 |

**301 MISSION STREET**
San Francisco, California

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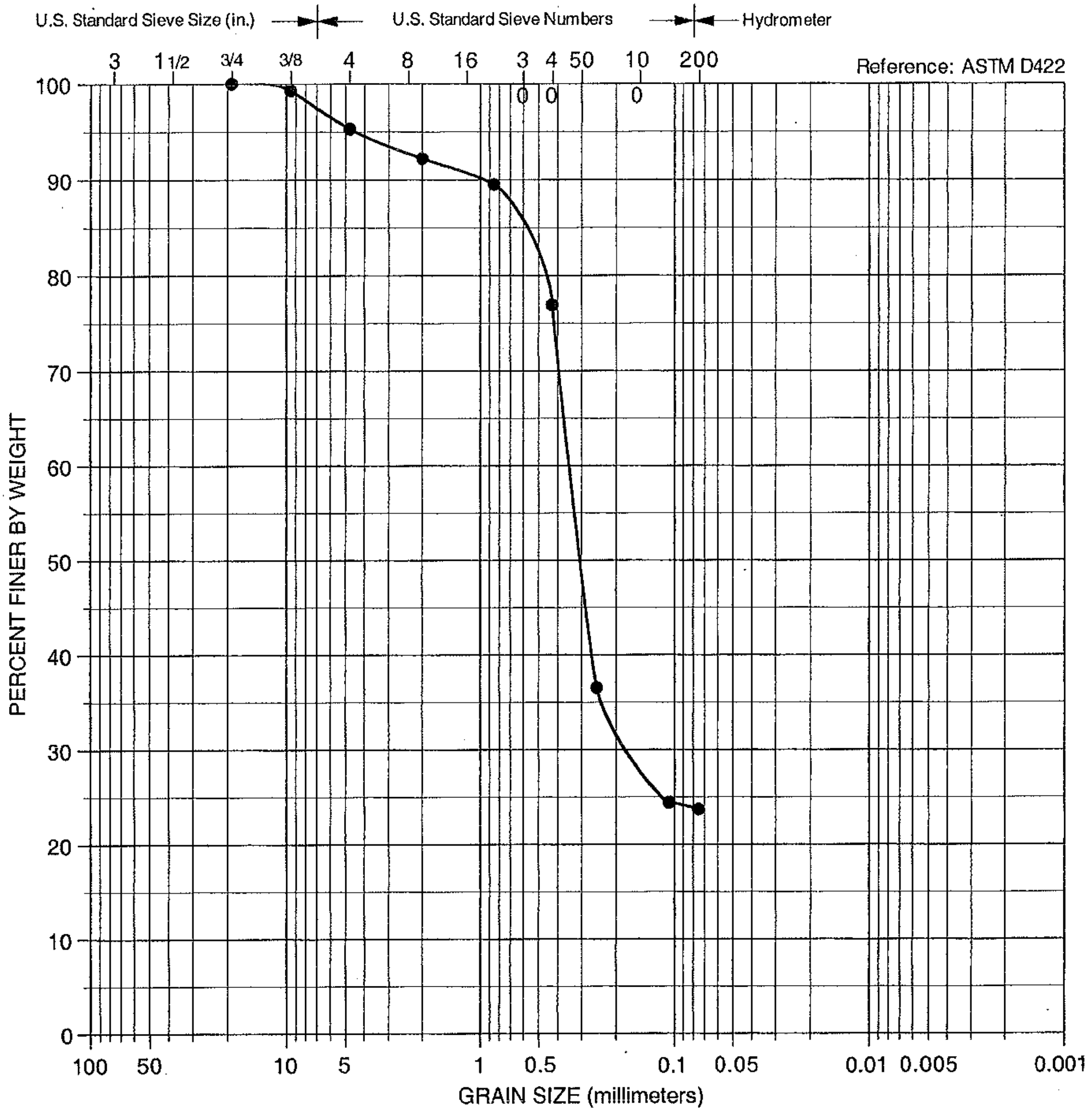
## PLASTICITY CHART

Date 08/08/01 | Project No. 3157.01 | Figure C-1



<table>
<tr><td rowspan="2">Cobbles</td><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td></tr>
</table>

| Symbol | Sample Source | Classification |
|---|---|---|
| ● | B-26 at 36 feet | CLAYEY SAND (SC), gray |

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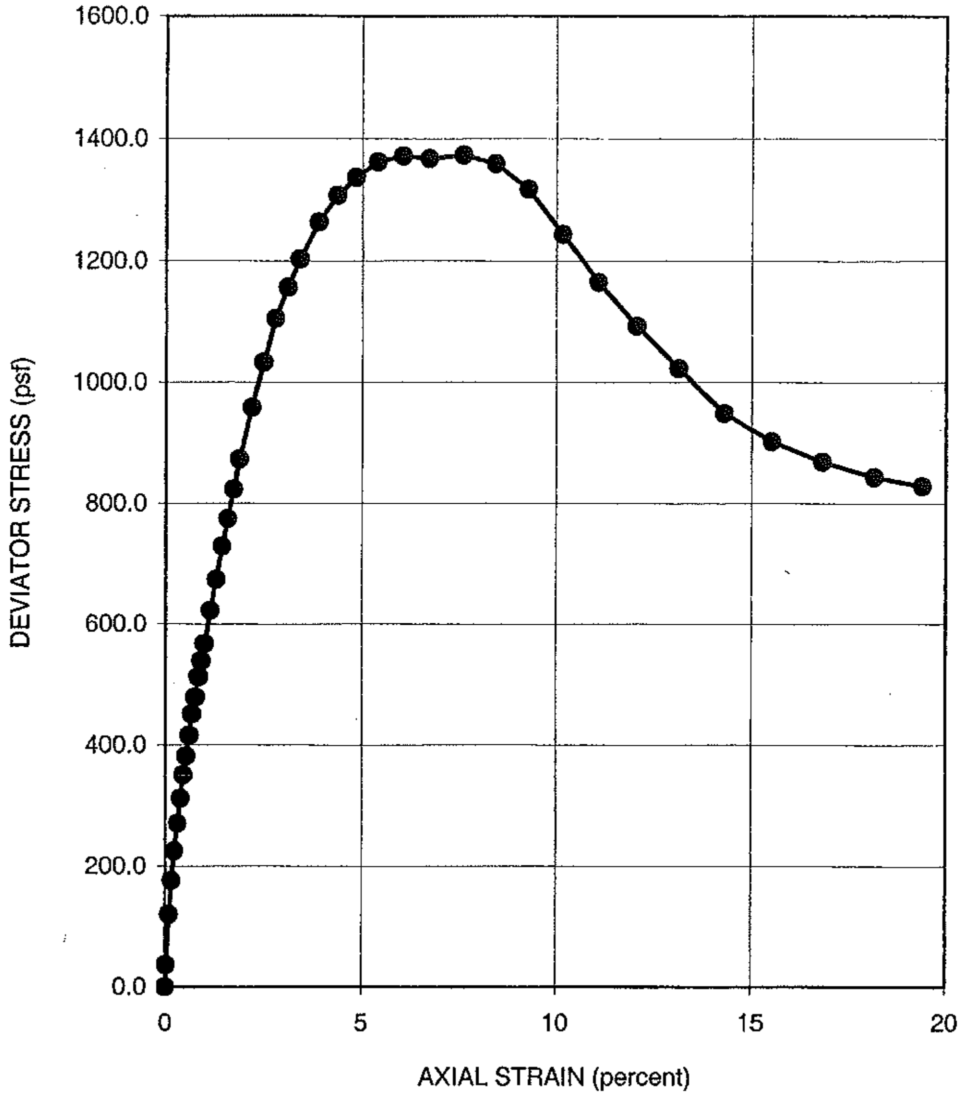
**PARTICLE SIZE ANALYSIS**

| Date 08/13/01 | Project No. 3157.01 | Figure C-2 |
|---|---|---|



| | | | | | |
|---|---|---|---|---|---|
| SAMPLER TYPE | Osterberg | | SHEAR STRENGTH | 685 | psf |
| DIAMETER (in.) 2.9 | HEIGHT (in.) 6.0 | | STRAIN AT FAILURE | 7.6 | % |
| MOISTURE CONTENT | 37.1 | % | CONFINING PRESSURE | 1400 | psf |
| DRY DENSITY | 85 | pcf | STRAIN RATE | 0.67 | % / min |
| DESCRIPTION | CLAY with SAND (CH), gray | | SOURCE B-1 at 35 Feet | | |

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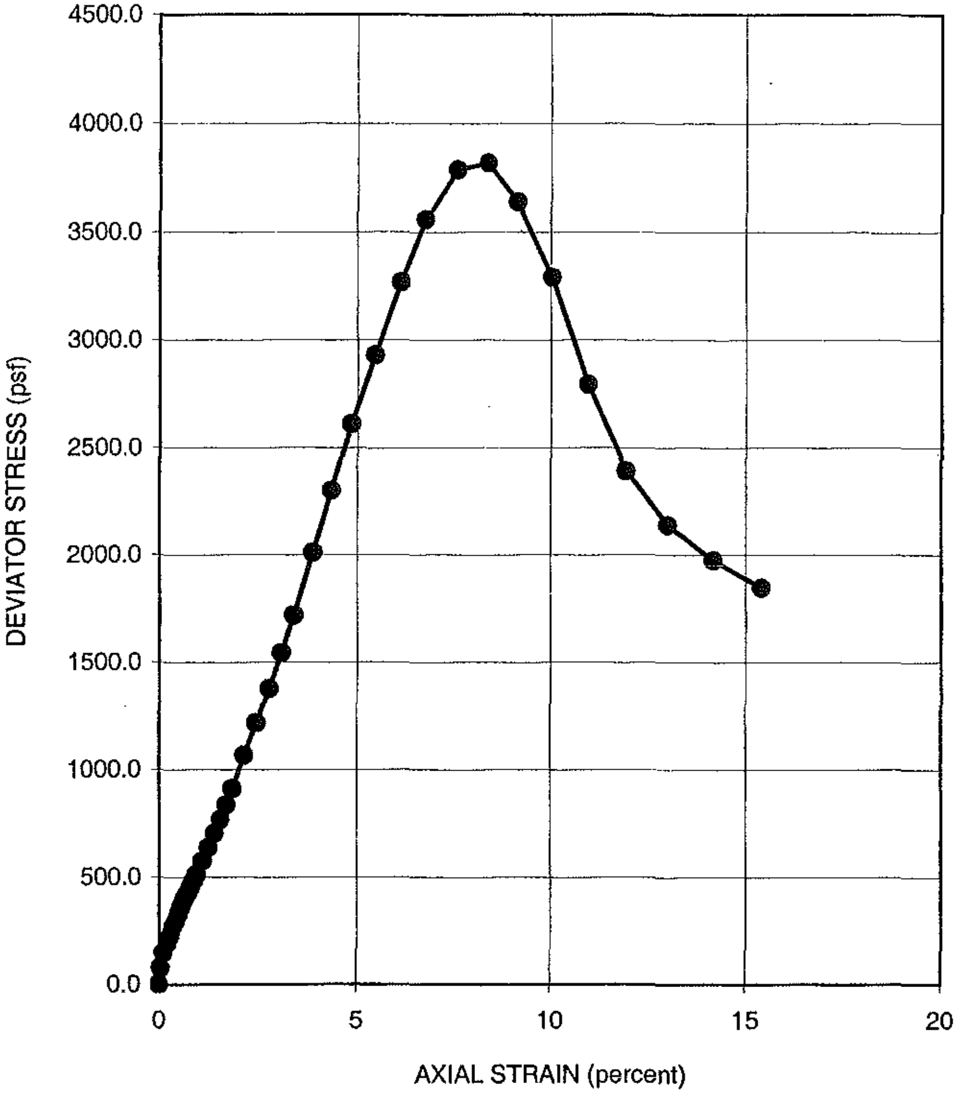
## UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 01/11/05 Project No. 3157.01 Figure C-3



| | | | | | | |
|---|---|---|---|---|---|---|
| SAMPLER TYPE | Osterberg | | | SHEAR STRENGTH | 1910 | psf |
| DIAMETER (in.) 2.9 | HEIGHT (in.) 6.0 | | | STRAIN AT FAILURE | 8.4 | % |
| MOISTURE CONTENT | | 20.5 | % | CONFINING PRESSURE | 3000 | psf |
| DRY DENSITY | | 110 | pcf | STRAIN RATE | 0.67 | % / min |
| DESCRIPTION | CLAY (CL), gray | | | SOURCE B-1 at 90 Feet | | |

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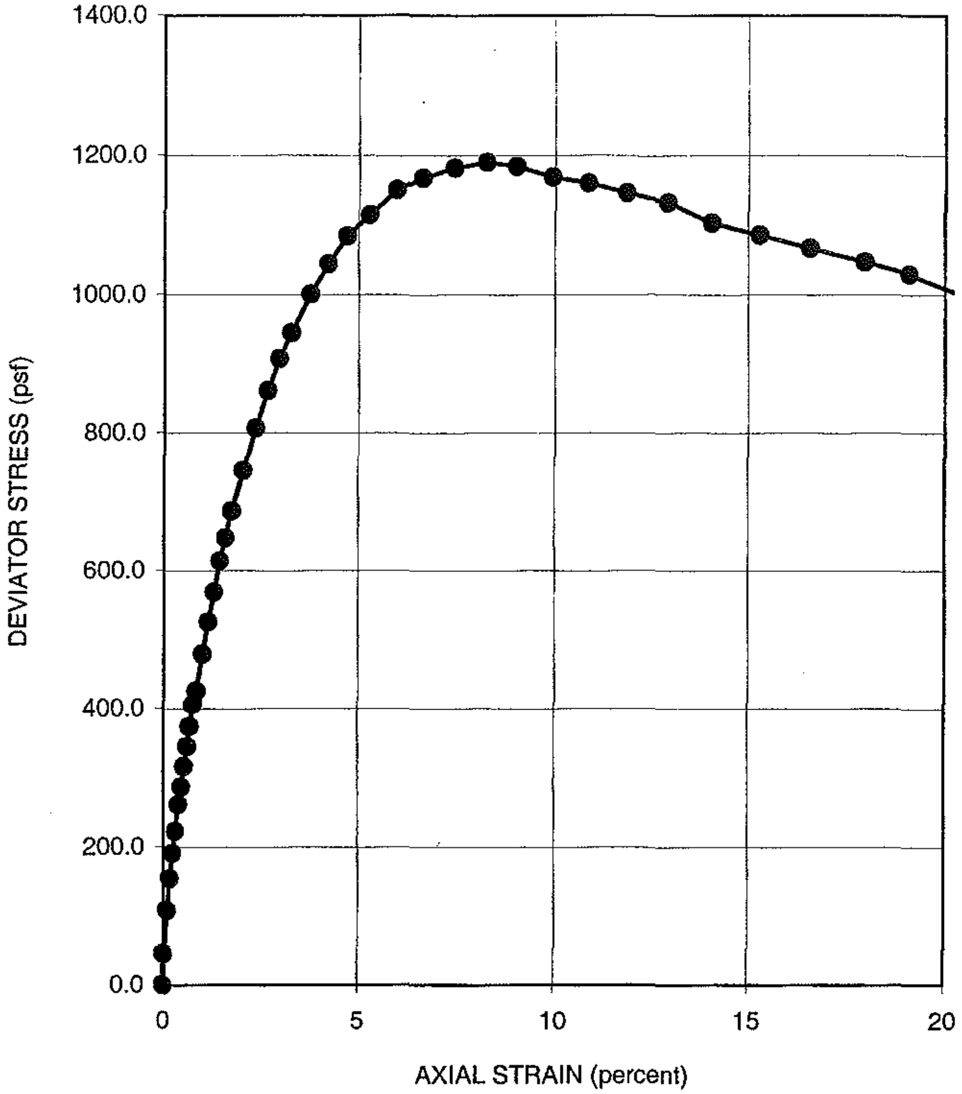
**UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST**

Date 01/11/05 | Project No. 3157.01 | Figure C-4



| | | | | | |
|---|---|---|---|---|---|
| SAMPLER TYPE Osterberg | | | SHEAR STRENGTH | 595 | psf |
| DIAMETER (in.) 2.9 | HEIGHT (in.) 6.0 | | STRAIN AT FAILURE | 8.2 | % |
| MOISTURE CONTENT | 32.0 | % | CONFINING PRESSURE | 1500 | psf |
| DRY DENSITY | 91 | pcf | STRAIN RATE | 0.67 | % / min |
| DESCRIPTION SANDY CLAY (CH), gray | | | SOURCE B-3 at 40 Feet | | |

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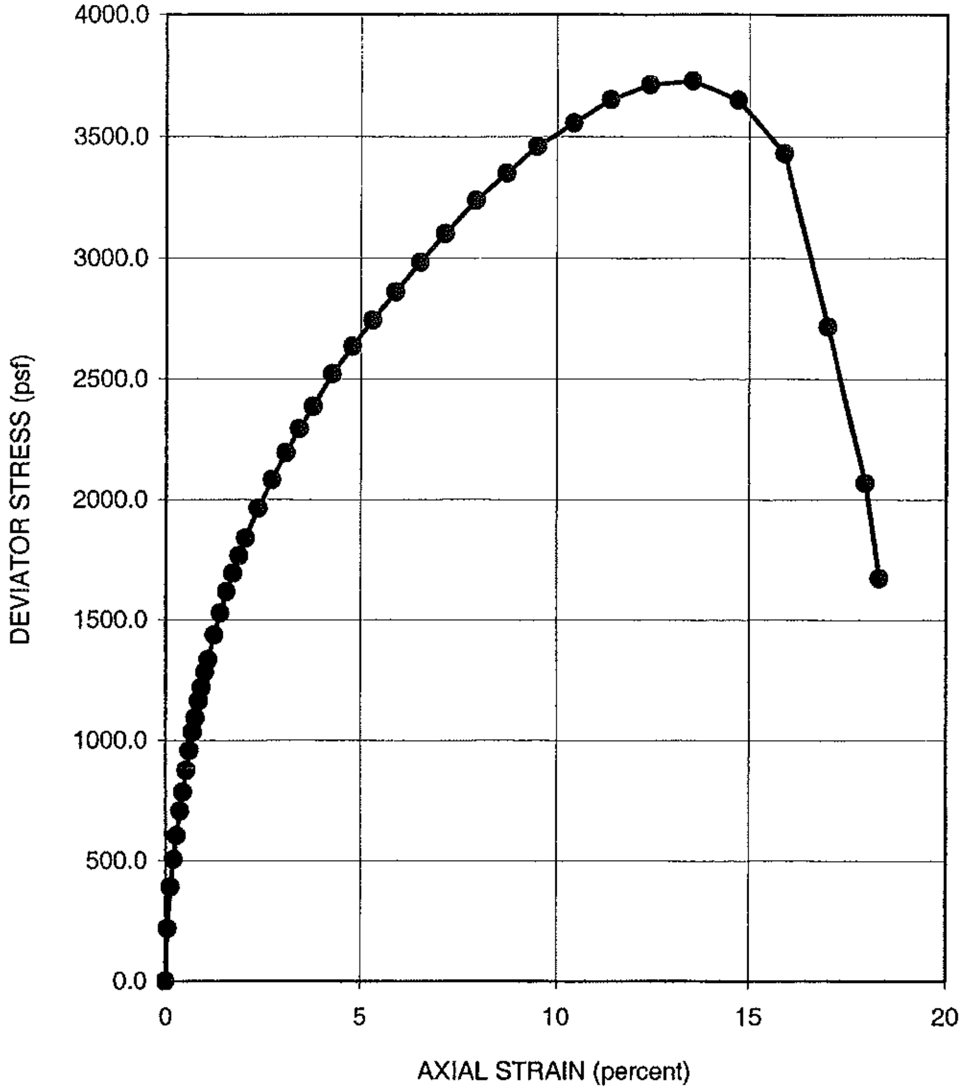
## UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 01/11/05 | Project No. 3157.01 | Figure C-5



| | | | | | | |
|---|---|---|---|---|---|---|
| SAMPLER TYPE | Sprague & Henwood | | | SHEAR STRENGTH | 1865 | psf |
| DIAMETER (in.) | 2.4 | HEIGHT (in.) | 5.8 | STRAIN AT FAILURE | 13.5 | % |
| MOISTURE CONTENT | | 25.2 | % | CONFINING PRESSURE | 3500 | psf |
| DRY DENSITY | | 100 | pcf | STRAIN RATE | 0.69 | % / min |
| DESCRIPTION | CLAY (CL), gray | | | SOURCE | B-3 at 105 Feet | |

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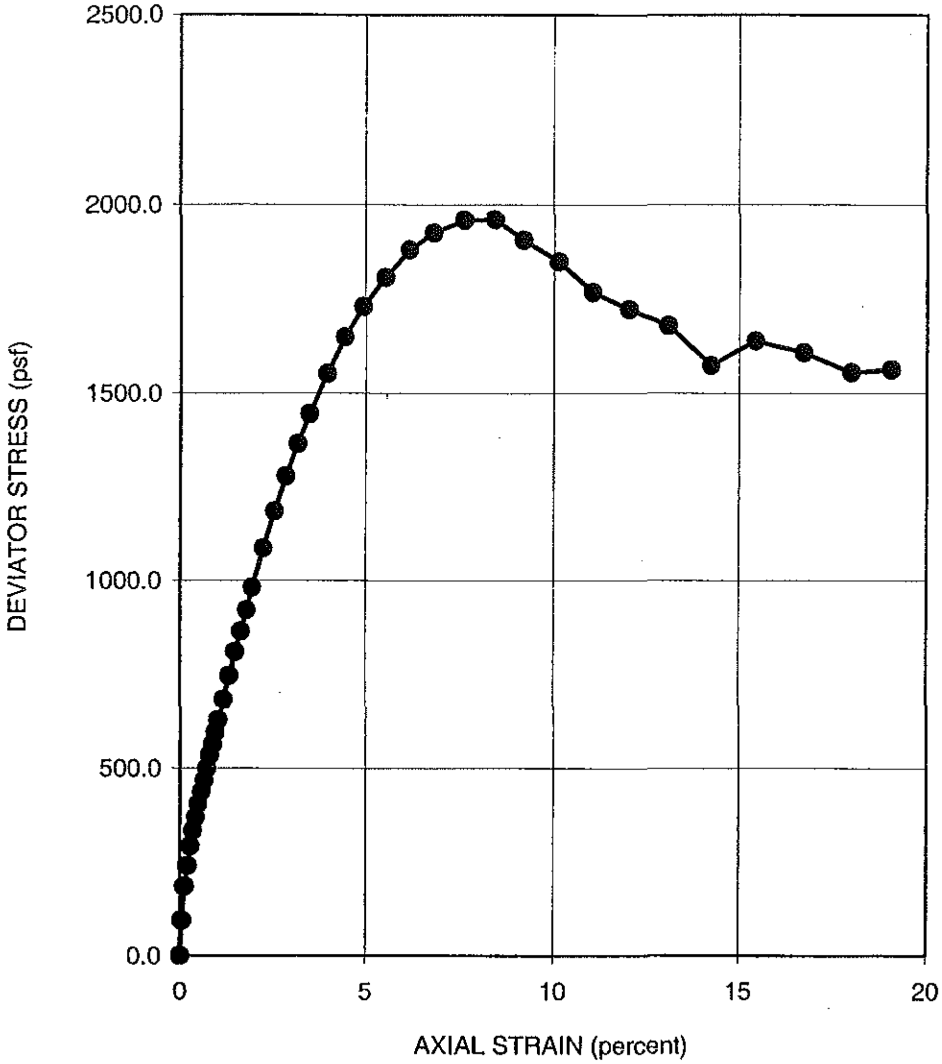
## UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

| | | | | | |
|---|---|---|---|---|---|
| Date | 01/11/05 | Project No. | 3157.01 | Figure | C-6 |



| | | | | | | |
|---|---|---|---|---|---|---|
| SAMPLER TYPE | Osterberg | | | SHEAR STRENGTH | 980 | psf |
| DIAMETER (in.) | 2.9 | HEIGHT (in.) | 6.0 | STRAIN AT FAILURE | 8.4 | % |
| MOISTURE CONTENT | | 24.0 | % | CONFINING PRESSURE | 1400 | psf |
| DRY DENSITY | | 103 | pcf | STRAIN RATE | 0.67 | % / min |
| DESCRIPTION | CLAYEY SAND (SC), gray | | | SOURCE | B-4 at 35 Feet | |

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San Francisco, California

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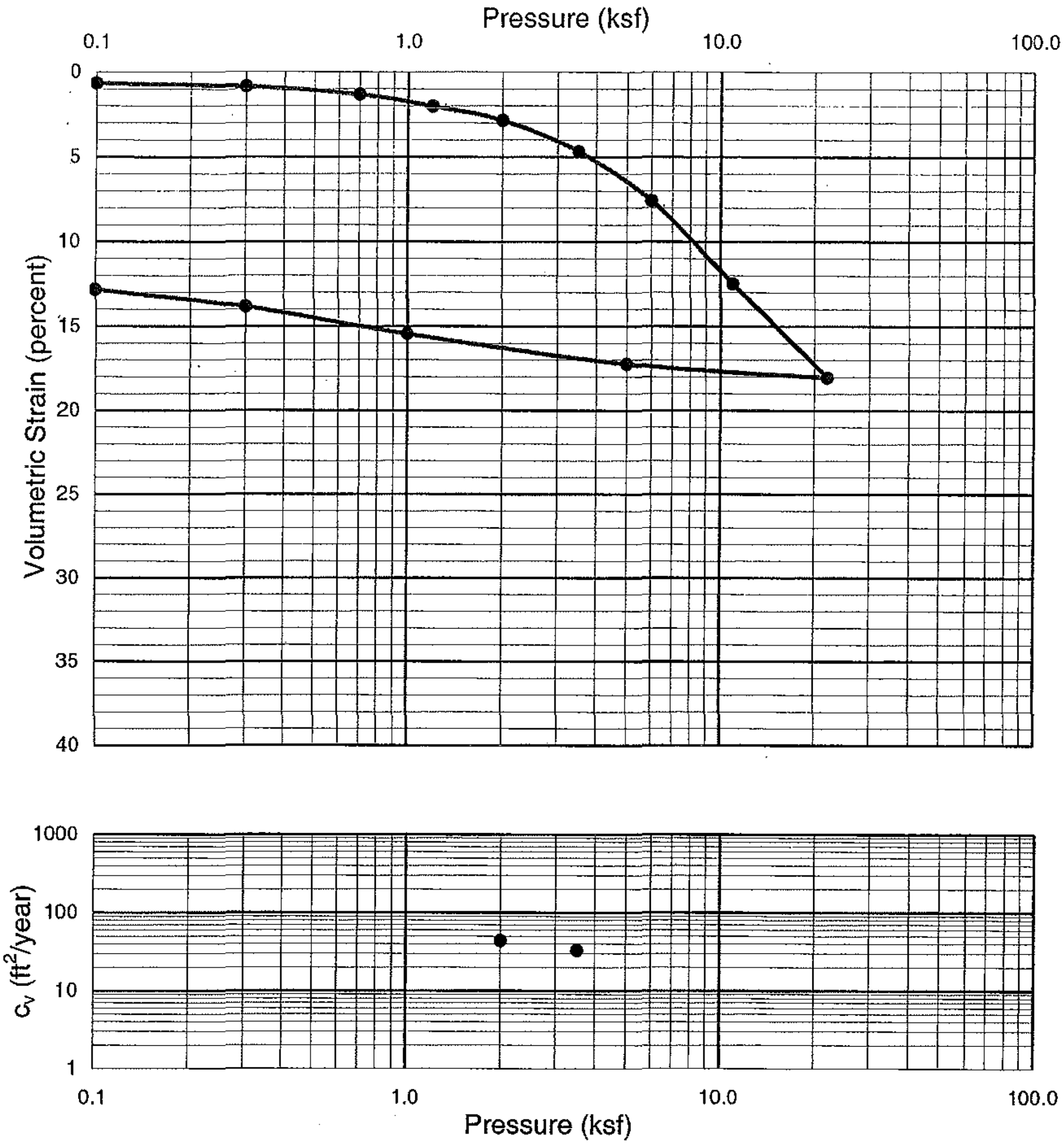
## UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 01/11/05 | Project No. 3157.01 | Figure C-7



| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Sampler Type: Osterberg | | | Condition | | Before Test | | After Test |
| Diameter (in) 2.42 | Height (in) | 1.00 | Water Content | $w_o$ | 35.0 % | $w_f$ | 27.2 % |
| Overburden Pressure, $p_o$ | 2,660 | psf | Void Ratio | $e_o$ | 0.93 | $e_f$ | 0.68 |
| Preconsol. Pressure, $p_c$ | 3,600 | psf | Saturation | $S_o$ | 100 % | $S_f$ | 100 % |
| Compression Ratio, $C_{\varepsilon c}$ | 0.22 | | Dry Density | $\gamma_d$ | 87 pcf | $\gamma_d$ | 100 pcf |
| LL --- | PL --- | | PI --- | | $G_s$ | 2.70 | (assumed) |
| Classification CLAY with SAND (CH), gray | | | Source | | B-1 at 35 Feet | | |

**301 MISSION STREET**
San Francisco, California

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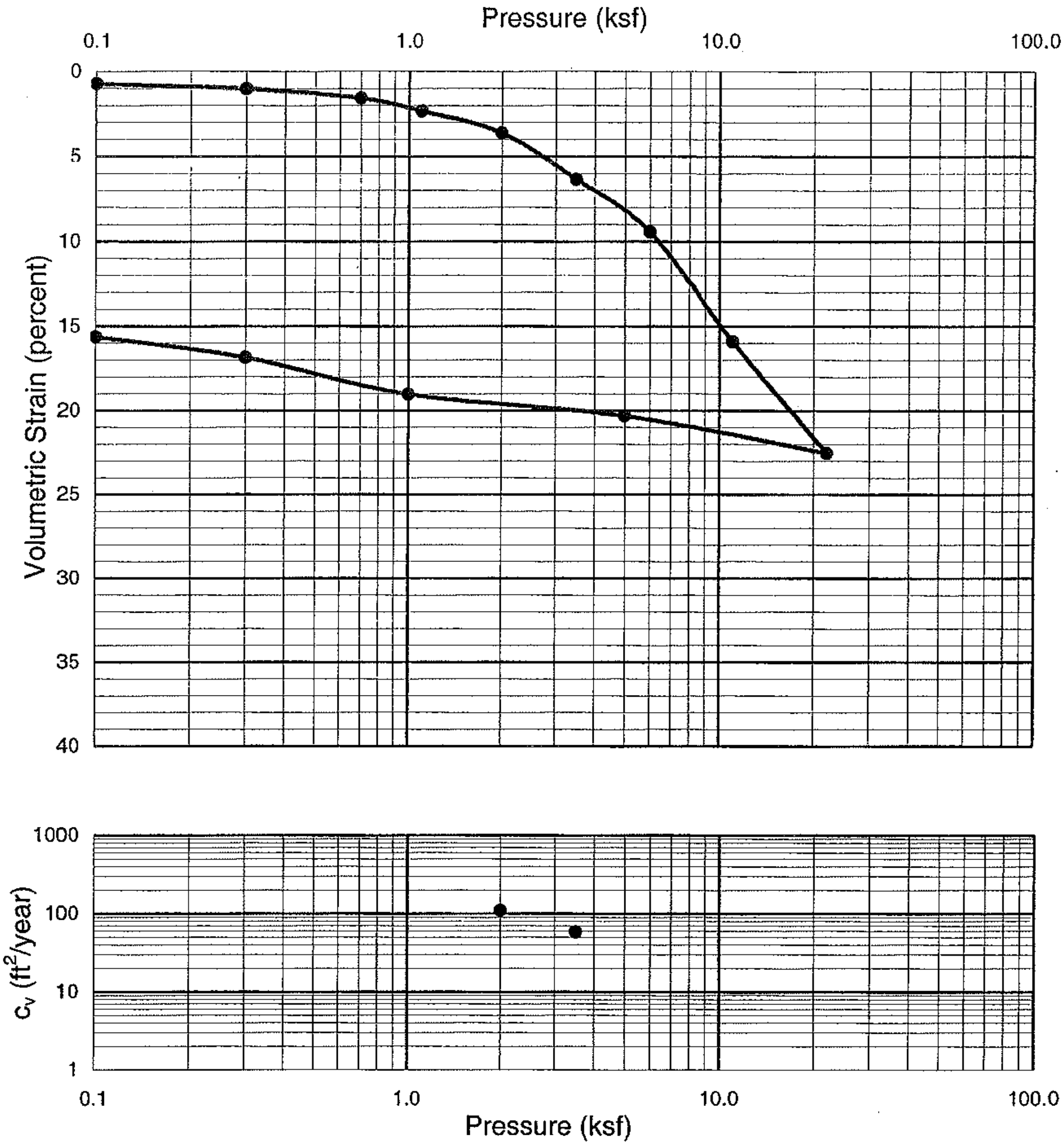
## CONSOLIDATION TEST REPORT

Date 01/11/05 | Project No. 3157.01 | Figure C-8



| | | | | | | |
|---|---|---|---|---|---|---|
| Sampler Type: Osterberg | | Condition | | Before Test | | After Test |
| Diameter (in) 2.42 | Height (in) 1.00 | Water Content | $w_o$ | 44.6 % | $w_f$ | 33.7 % |
| Overburden Pressure, $p_o$ 2,870 psf | | Void Ratio | $e_o$ | 1.24 | $e_f$ | 0.89 |
| Preconsol. Pressure, $p_c$ 3,600 psf | | Saturation | $S_o$ | 97.0 % | $S_f$ | 100 % |
| Compression Ratio, $C_{\epsilon c}$ 0.27 | | Dry Density | $\gamma_d$ | 75 pcf | $\gamma_d$ | 89 pcf |
| LL --- | PL --- | PI --- | | | $G_s$ 2.70 | (assumed) |

Classification SANDY CLAY (CH), gray Source B-3 at 35 Feet

**301 MISSION STREET**
San Francisco, California

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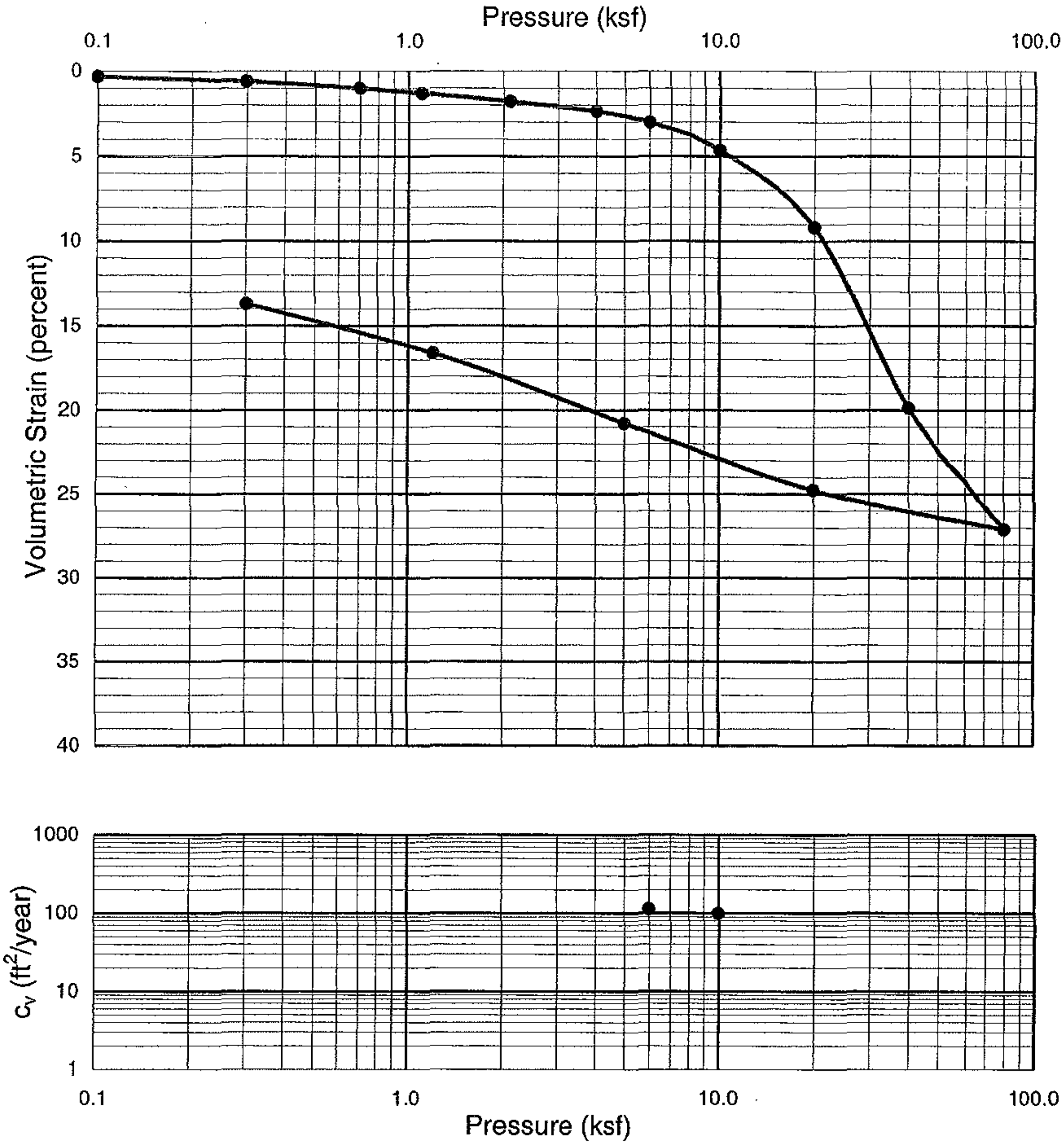
## CONSOLIDATION TEST REPORT

Date 01/11/05 Project No. 3157.01 Figure C-9



| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Sampler Type: Shelby Tube | | | Condition | | Before Test | | | After Test | |
| Diameter (in) | 2.42 | Height (in) 1.00 | Water Content | $w_o$ | 44.6 | % | $w_f$ | 35.2 | % |
| Overburden Pressure, $p_o$ | 8,280 | psf | Void Ratio | $e_o$ | 1.22 | | $e_f$ | 0.91 | |
| Preconsol. Pressure, $p_c$ | 13,000 | psf | Saturation | $S_o$ | 99.1 | % | $S_f$ | 100 | % |
| Compression Ratio, $C_{\varepsilon c}$ | 0.31 | | Dry Density | $\gamma_d$ | 76 | pcf | $\gamma_d$ | 88 | pcf |
| LL --- | | PL --- | | PI --- | | | $G_s$ | 2.70 | (assumed) |
| Classification CLAY (CL), gray | | | | Source | B-3 at 124 Feet | | | | |

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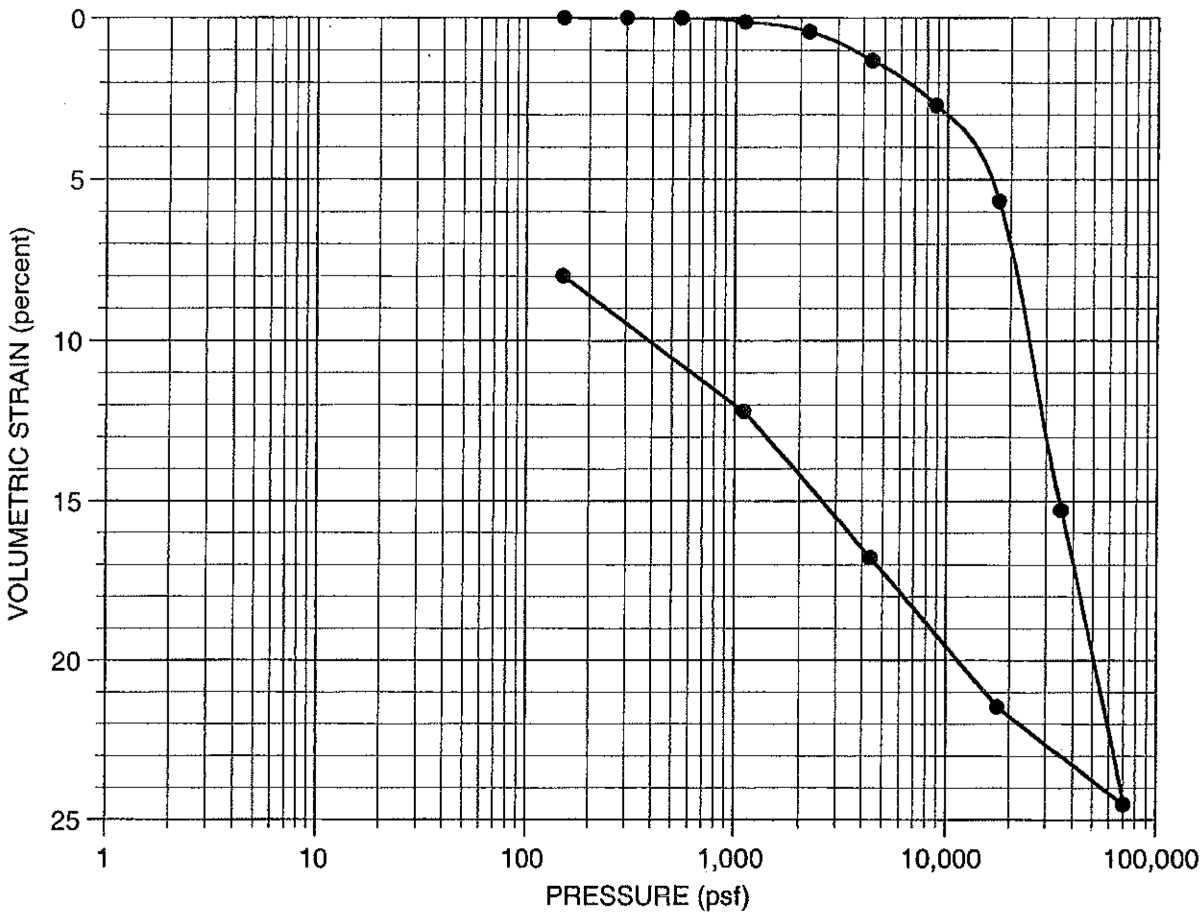
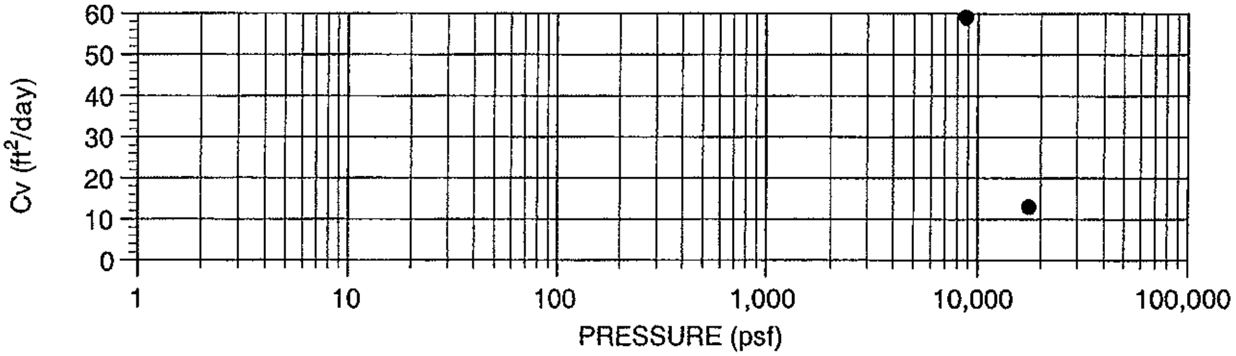
San Francisco, California

**Treadwell&Rollo**

## CONSOLIDATION TEST REPORT

Date 01/11/05 Project No. 3157.01 Figure C-10





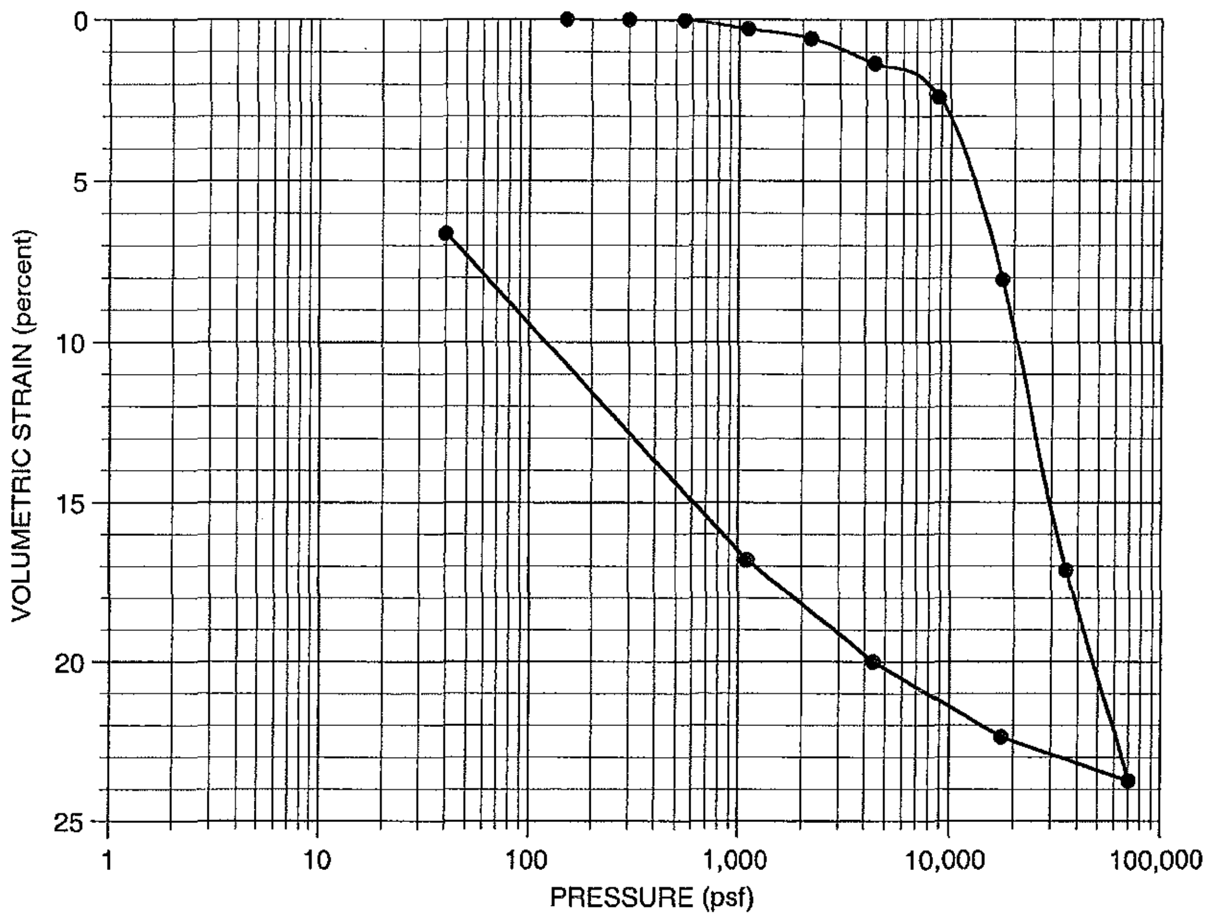
| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Type Shelby Tube | | Condition | | Before test | | After test |
| Diameter (in) 2.4 | Height (in) 1.0 | Water Content | $w_o$ | 45.3 % | $w_f$ | 41.2 % |
| Overburden Pressure, $P_o$ 8,000 psf | | Void Ratio | $e_o$ | 1.229 | $e_f$ | 1.110 |
| Preconsol. Pressure, $P_c$ 16,000 psf | | Saturation | $S_o$ | 99.6 % | $S_f$ | 100 % |
| Compression Ratio, $C_{ec}$ 0.32 | | Dry Density | $\gamma_d$ | 76 pcf | $\gamma_d$ | 80 pcf |
| LL -- | PL -- | PI -- | | $G_s$ 2.7 (assumed) | | |
| Classification CLAY and SAND (CH), dark gray | | | | Source B-6 at 170 feet | | |

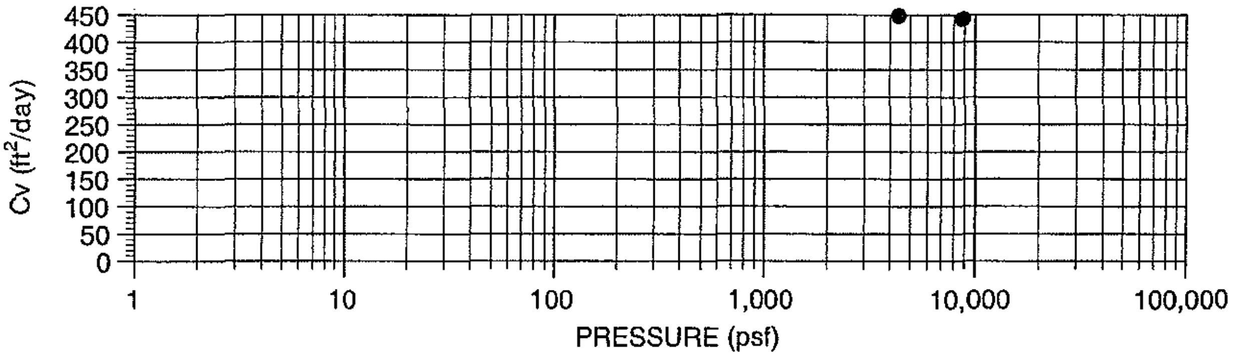
**301 MISSION STREET**
San Francisco, California

## CONSOLIDATION TEST REPORT

**Treadwell&Rollo**

Date 01/11/05 | Project No. 3157.02 | Figure C-12





| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Sample Type | Shelby Tube | | | Condition | | Before test | | After test |
| Diameter (in) | 2.7 | Height (in) | 1.0 | Water Content | $w_o$ | 40.2 % | $w_f$ | 34.0 % |
| Overburden Pressure, $P_o$ | 5,300 psf | | | Void Ratio | $e_o$ | 1.100 | $e_f$ | 0.915 |
| Preconsol. Pressure, $P_c$ | 11,000 psf | | | Saturation | $S_o$ | 98.8 % | $S_f$ | 100 % |
| Compression Ratio, $C_{ec}$ | 0.27 | | | Dry Density | $\gamma_d$ | 80 pcf | $\gamma_d$ | 88 pcf |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| LL | -- | PL | -- | PI | -- | $G_s$ | 2.7 (assumed) |

Classification CLAY (CL), dark gray

Source B-7 at 110 feet

**301 MISSION STREET**
San Francisco, California

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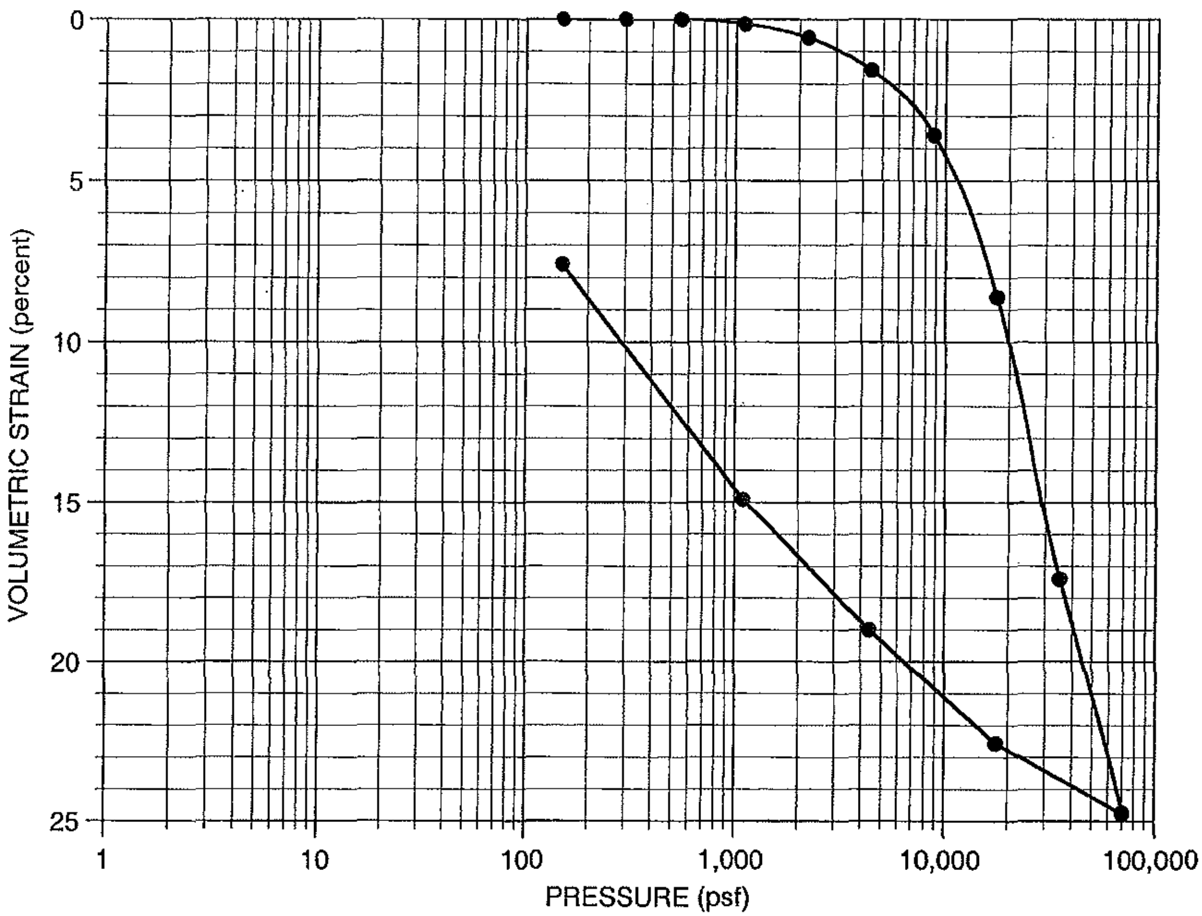
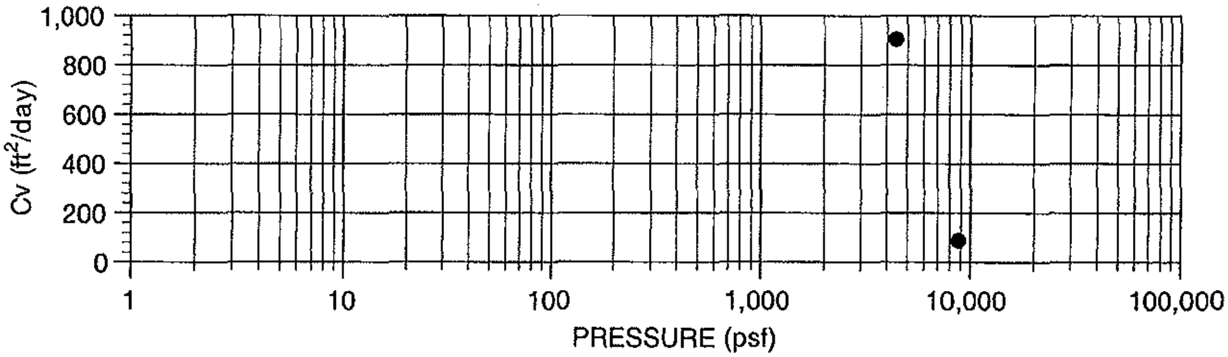
## CONSOLIDATION TEST REPORT

Date 07/01/04 | Project No. 3157.02 | Figure C-13





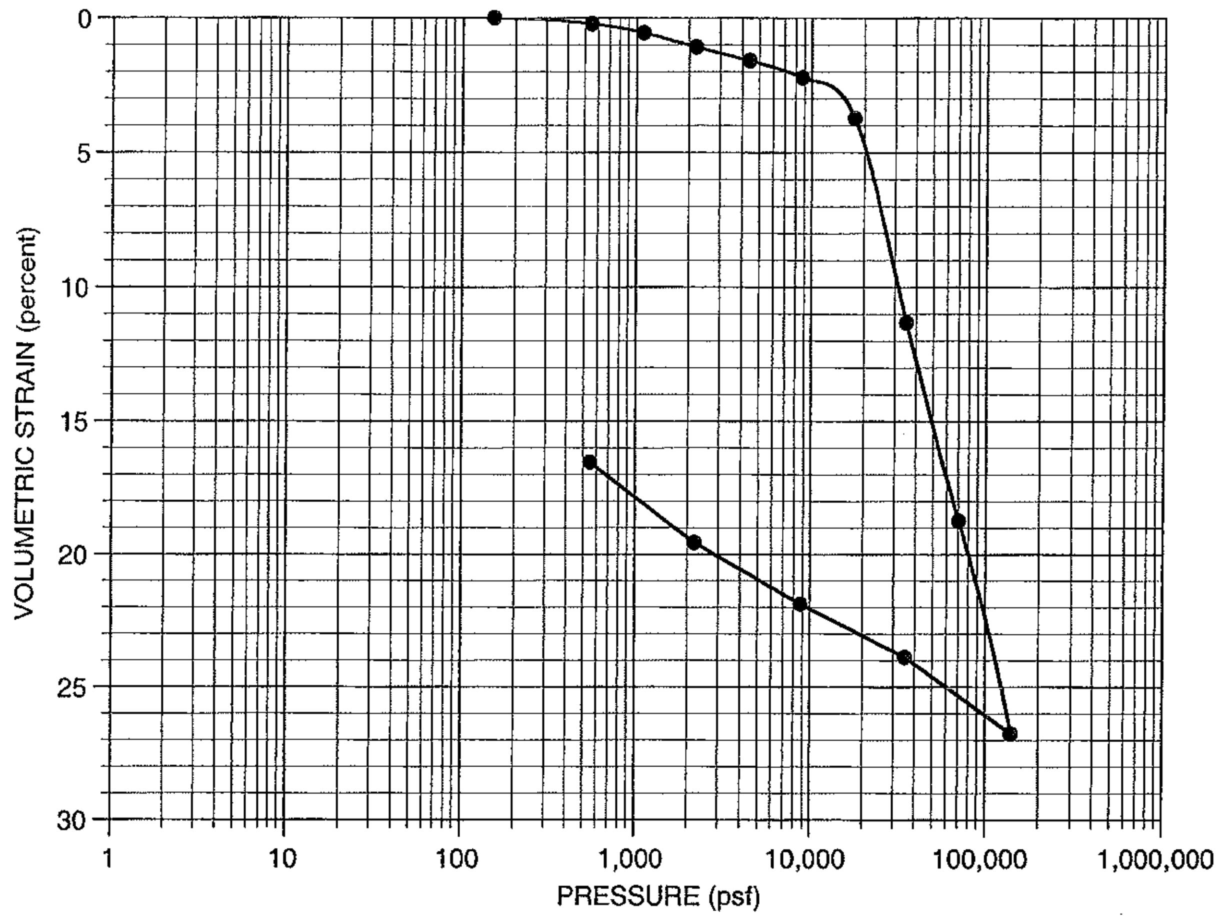
| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Type Shelby Tube | | Condition | | Before test | | After test |
| Diameter (in) -- | Height (in) -- | Water Content | $w_o$ | 42.4 % | $w_f$ | 38.6 % |
| Overburden Pressure, $P_o$ | 6,800 psf | Void Ratio | $e_o$ | 1.155 | $e_f$ | 1.041 |
| Preconsol. Pressure, $P_c$ | 11,250 psf | Saturation | $S_o$ | 99.2 % | $S_f$ | 100 % |
| Compression Ratio, $C_{ec}$ | 0.26 | Dry Density | $\gamma_d$ | 78 pcf | $\gamma_d$ | 83 pcf |
| LL -- | PL -- | PI -- | | $G_s$ 2.7 (assumed) | | |
| Classification CLAY (CL), dark gray | | | | Source B-7 at 150 feet | | |

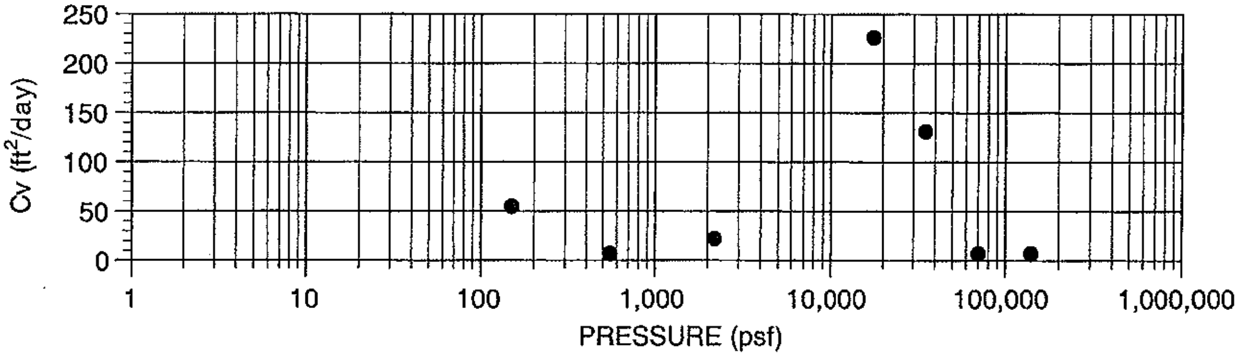
**301 MISSION STREET**
San Francisco, California

## CONSOLIDATION TEST REPORT

**Treadwell&Rollo**

Date 01/11/05 | Project No. 3157.02 | Figure C-14





| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Type | Shelby Tube | Condition | | Before test | | After test |
| Diameter (in) 2.7 | Height (in) 1.0 | Water Content | $w_o$ | 36.7 % | $w_f$ | 27.5 % |
| Overburden Pressure, $P_o$ | 7,900 psf | Void Ratio | $e_o$ | 1.016 | $e_f$ | 0.756 |
| Preconsol. Pressure, $P_c$ | 18,100 psf | Saturation | $S_o$ | 99.2 % | $S_f$ | 100 % |
| Compression Ratio, $C_{ec}$ | 0.25 | Dry Density | $\gamma_d$ | 85 pcf | $\gamma_d$ | 98 pcf |

| | | | |
|---|---|---|---|
| LL -- | PL -- | PI -- | $G_s$ 2.7 (assumed) |
| Classification CLAY (CL), green-gray | | Source B-7 at 190 feet | |

**301 MISSION STREET**
San Francisco, California

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## CONSOLIDATION TEST REPORT

Date 01/11/05 | Project No. 3157.02 | Figure C-15

# APPENDIX D

# APPENDIX D

## Probabilistic Seismic Hazard Analysis

## APPENDIX D

## 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 a site-specific design response spectrum for the project, we performed the following:

- a PSHA to develop a uniform hazard response spectrum for 10 percent probability of exceedance in 50 years (475-year return period). This is consistent with the definition of the Design Basis Earthquake (DBE) in the 2001 San Francisco Building Code (SFBC).
- development of horizontal recommended spectrum.

The rock spectrum for the hazard level was developed using the computer code EZFRISK 6.22 (Risk Engineering 2004). 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.

## D1.0 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. Therefore, we modeled the fault rupture lengths 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.

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

where:

$\nu_i$ = 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.

## A2.0 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 D-1.

**TABLE D-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 |

The segmentation of faults, maximum 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 by Cao et al. (2003) and WGCEP (2003) in our seismic hazard model. Table D-2 presents the distance and direction from the site to the fault, maximum magnitude, slip rate, and fault length for individual fault segments and combination segments used in our model.

## TABLE D-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) | 13.4 | West | 7.90 | 19 | 473 |
| San Andreas – Peninsula (SAP) | 13.4 | West | 7.15 | 17 | 85 |
| San Andreas – SAP+SAN+SAO | 13.4 | West | 7.83 |  | 411 |
| San Andreas – SAS+SAP | 13.4 | West | 7.42 | 17 | 147 |
| San Andreas – SAS+SAP+SAN | 13.4 | West | 7.76 |  | 338 |
| Hayward-Rodgers Creek – NH | 15.6 | East | 6.49 | 9 | 35 |
| Hayward-Rodgers Creek – NH+RC | 15.6 | East | 7.11 | 9 | 98 |
| Hayward-Rodgers Creek – SH+NH | 15.6 | East | 6.91 | 9 | 88 |
| Hayward-Rodgers Creek – SH+NH+RC | 15.6 | East | 7.26 | 9 | 151 |
| San Andreas – SAN | 15.7 | West | 7.45 | 24 | 191 |
| San Andreas – SAN+SAO | 15.7 | West | 7.70 | 24 | 330 |
| Hayward-Rodgers Creek – SH | 16.6 | East | 6.67 | 9 | 53 |
| San Gregorio – SGN | 19.1 | West | 7.23 | 7 | 110 |
| San Gregorio – SGS+SGN | 19.1 | West | 7.44 | 5 | 176 |
| Mt Diablo – MTD | 32.8 | East | 6.65 | 2 | 25 |
| Hayward-Rodgers Creek – RC | 33.2 | North | 6.98 | 9 | 63 |
| Calaveras – CC+CN | 34.2 | East | 6.90 |  | 104 |
| Calaveras – CN | 34.2 | East | 6.78 | 6 | 45 |
| Calaveras – CS+CC+CN | 34.2 | East | 6.93 |  | 123 |
| Concord/GV – CON | 37.4 | East | 6.25 | 4 | 20 |
| Concord/GV – CON+GVS | 37.4 | East | 6.58 |  | 42 |
| Concord/GV – CON+GVS+GVN | 37.4 | East | 6.71 |  | 56 |
| Concord/GV – GVS | 39.4 | Northeast | 6.24 | 5 | 22 |
| Concord/GV – GVS+GVN | 39.4 | Northeast | 6.24 | 5 | 36 |
| Monte Vista-Shannon | 41.4 | Southeast | 6.80 | 0.4 | 41 |
| Point Reyes | 42.1 | West | 6.80 | 0.3 | 47 |
| West Napa | 43.7 | Northeast | 6.50 | 1 | 30 |
| Greenville – GN | 50.6 | East | 6.66 | 2 | 27 |
| Greenville – GS+GN | 50.6 | East | 6.94 | 2 | 51 |
| Concord/GV – GVN | 56.5 | Northeast | 6.02 | 5 | 14 |
| Hayward – South East Extension | 57.0 | Southeast | 6.40 | 3 | 26 |
| Great Valley 6 | 60.5 | East | 6.70 | 1.5 | 45 |
| Calaveras – CC | 64.6 | Southeast | 6.23 | 15 | 59 |
| Calaveras – CS+CC | 64.6 | Southeast | 6.36 | 15 | 78 |
| Greenville – GS | 65.4 | East | 6.60 | 2 | 24 |
| Great Valley 5 | 65.4 | East | 6.50 | 1.5 | 28 |
| Great Valley 4 | 71.5 | Northeast | 6.60 | 1.5 | 42 |
| Hunting Creek-Berryessa | 75.8 | North | 6.90 | 6 | 60 |
| San Andreas – Santa Cruz Mnts. (SAS) | 76.7 | Southeast | 7.03 | 17 | 62 |
| Great Valley 7 | 77.0 | East | 6.70 | 1.5 | 45 |
| Sargent | 82.9 | Southeast | 6.80 | 3 | 53 |
| Zayante-Vergeles | 86.6 | Southeast | 6.80 | 0.1 | 56 |
| Maacama-garberville | 91.2 | North | 6.90 | 9 |  |
| Monterey Bay-Tularcitos | 99.8 | Southeast | 7.10 | 0.5 | 84 |

## D3.0 ATTENUATION RELATIONSHIPS

Based on subsurface conditions, the site is categorized as stiff soil (SFBC designation $S_D$). In order to estimate site-specific spectra at the ground surface we averaged results obtained by using various attenuation relationships for stiff soil conditions. These relationships are primarily dependent on the magnitude of the earthquake and the distance from the site to the fault. Four stiff soil attenuation relationships were used in our analyses. These included: Abrahamson and Silva (1997), Boore et al. (1997), Sadigh et al. (1997), and Campbell (1997). The attenuation relationships used in the study were developed using different earthquake databases that treat the magnitude and distance effects differently. The average of the relationships was used to develop the recommended surface spectra.

## D4.0 PSHA RESULTS

The results of the PSHA for the DBE hazard level is shown on Figure D-1. The average of the attenuation relationships is also shown on the figure. Figure D-2 presents a comparison of the recommended surface spectra (DBE) with the corresponding 2001 SFBC soil profile type $S_D$ spectra.

The proposed 60-story tower and podium structure will be both have underground portions which at foundation level will either be about 25 feet or about 60 feet below the ground surface, respectively. It has long been recognized that spectral values show reductions with depth below the ground surface. Such effects have been supported analytically and have been shown by recordings from downhole arrays and in comparisons of recordings in the free field with those in adjacent structures at their basement levels. In general the data suggest that response spectra at depths of about 15 to 40 feet below the ground surface is lower than the surface spectra for periods less than about 1.0 second.

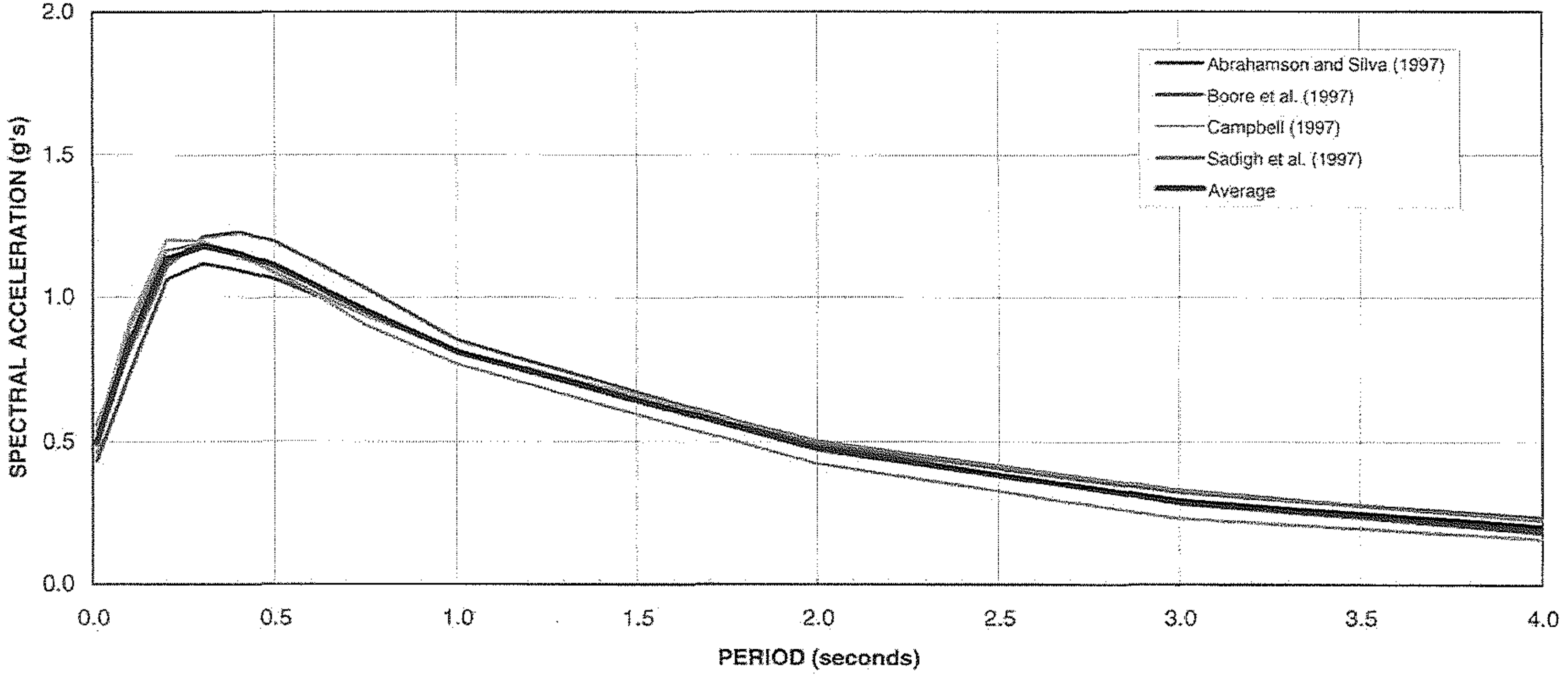
Golesorkhi and Gouchon (2000) developed recommended ratios between spectra at depth to surface spectra that can be used to modify surface spectra for basement/depth effects. Figure D-3 shows this ratio and also provides a comparison with recorded data. These ratios are based on analytical studies and data by Seed (1986), Tsai (1990), Ostadan (1992), Sykora and Bastani (1998), and most recently Stewart (1999) and were used to modify the surface spectra and develop the basement level spectra. Furthermore, FEMA 440 Appendix 8, discusses effects of reduction of surface (free field) spectrum as a function depth of embedment of the foundation. The reductions presented in the FEMA document are within the same range as recommended by Golesorkhi and Gouchon (2000). Therefore, it is our opinion that the basement reduction is justified and appropriate. The recommended horizontal surface and basement level spectra are presented on Figure D-4. We recommend the use of the basement level spectra at the foundation level for design.

Digitized values of the recommended surface and basement spectra for a damping ratio of 5 percent are presented in Table D-3.

### TABLE D-3

**Spectral Acceleration (g) for Damping Ratio of 5 percent 10 percent probability of Exceedance in 50 years (DBE)**

| Period (sec) | Ground Surface | Basement |
|---|---|---|
| 0.01 | 0.495 | 0.318 |
| 0.1 | 0.842 | 0.590 |
| 0.2 | 1.132 | 0.849 |
| 0.3 | 1.179 | 0.933 |
| 0.4 | 1.153 | 0.933 |
| 0.5 | 1.108 | 0.918 |
| 0.75 | 0.953 | 0.818 |
| 1.0 | 0.811 | 0.745 |
| 2.0 | 0.473 | 0.473 |
| 3.0 | 0.290 | 0.290 |
| 4.0 | 0.199 | 0.199 |
| 5.0 | 0.160 | 0.160 |
| 6.0 | 0.133 | 0.133 |



Damping Ratio = 5%

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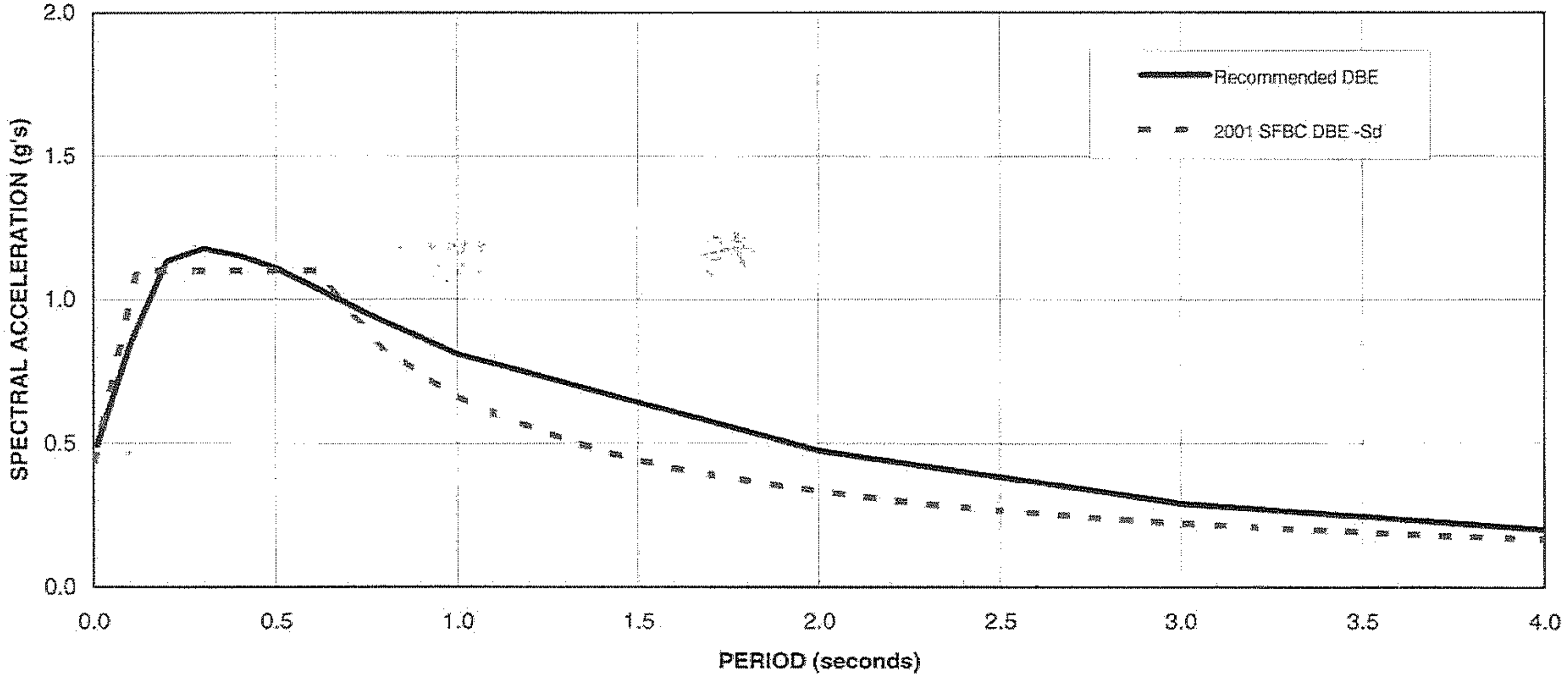
San Francisco, California

**RESULTS OF PSHA, 10 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS**

| | | |
|---|---|---|
| Date 11/19/04 | Project No. 3157.02 | Figure D-1 |

Treadwell&Rollo



Damping Ratio = 5%

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

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San Francisco, California

**COMPARISON OF RECOMMENDED DBE SURFACE AND 2001 SFBC SPECTRA**

| | | |
|---|---|---|
| Date 11/19/04 | Project No. 3157.02 | Figure D-2 |

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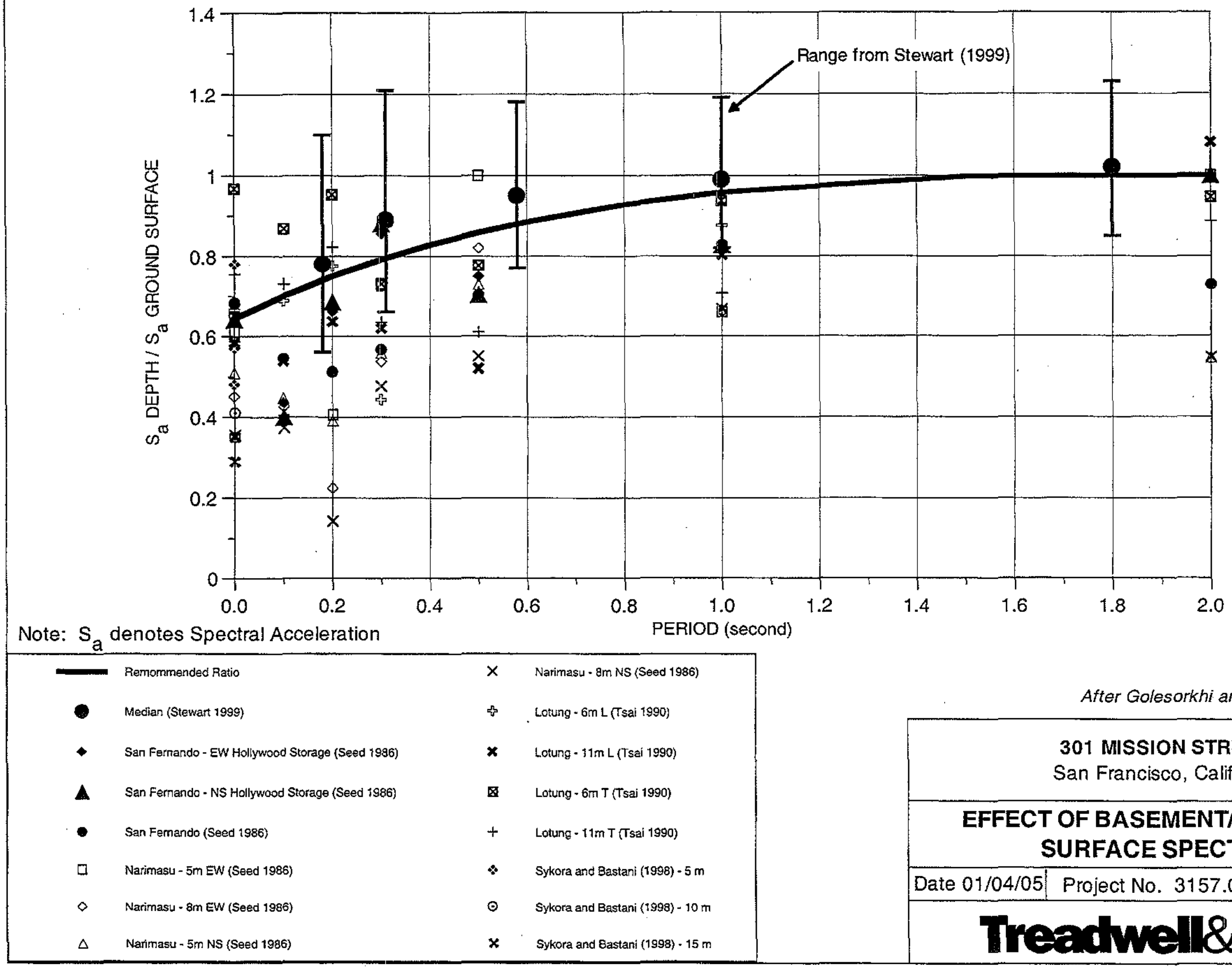

Note: $S_a$ denotes Spectral Acceleration

*After Golesorkhi and Gouchon (2000)*

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**EFFECT OF BASEMENT/DEPTH ON SURFACE SPECTRA**

| Date 01/04/05 | Project No. 3157.02 | Figure D-3 |
|---|---|---|

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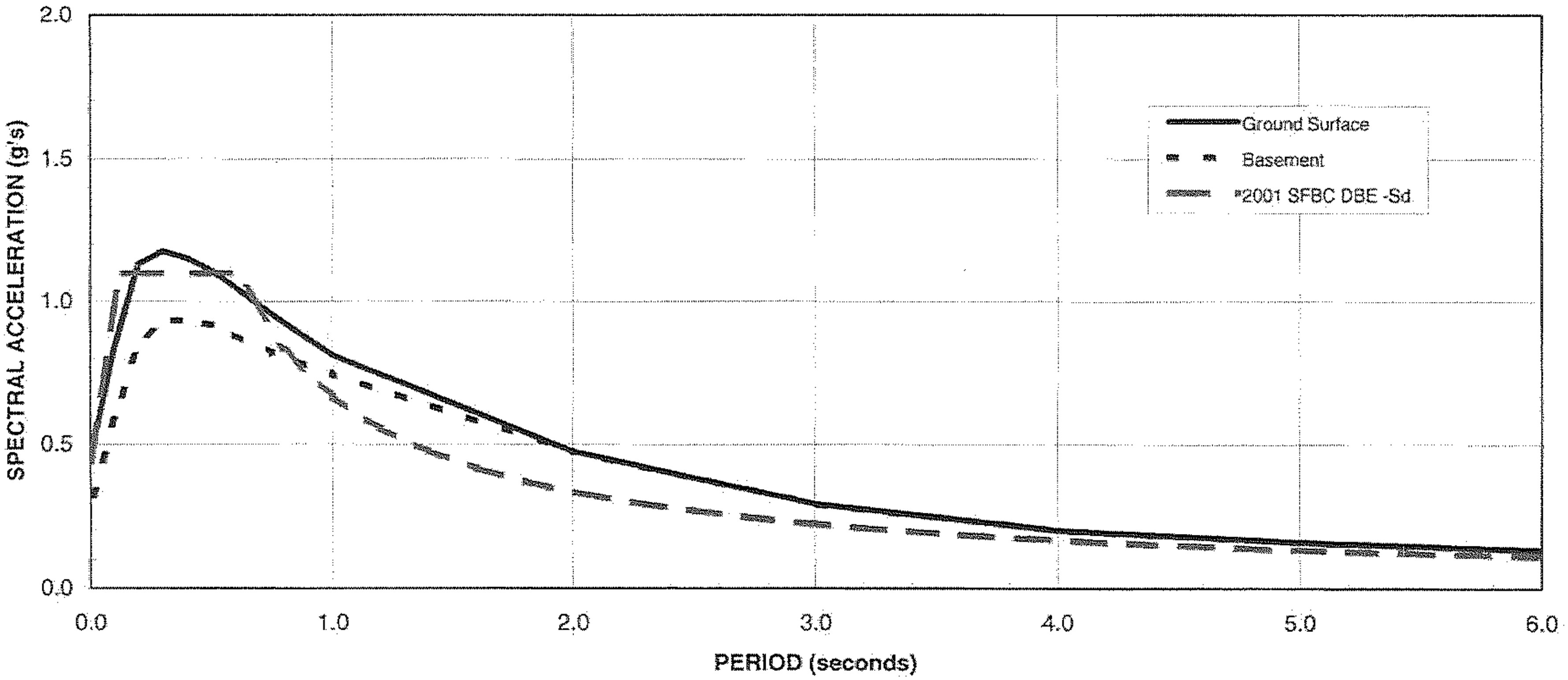

Damping Ratio = 5%

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

**301 MISSION STREET**
San Francisco, California

**RECOMMENDED SPECTRA**

| | | |
|---|---|---|
| Date 11/19/04 | Project No. 3157.02 | Figure D-4 |

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# APPENDIX E

# APPENDIX E

## Borings from Previous Investigations by Dames & Moore

## BORING 1

DATE DRILLED 11/12-13/80
SURFACE ELEVATION +2.7'

| DEPTH IN FEET | LABORATORY TEST DATA | | | | | | | | SAMPLING | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | ATTERBERG LIMITS | | STRENGTH TEST DATA | | | | | | |
| | TESTS REPORTED ELSEWHERE | LIQUID LIMIT | PLASTIC LIMIT | TYPE OF STRENGTH TEST | NORMAL OR CONFINING PRESSURE, PSF | SHEAR STRENGTH, PSF | MOISTURE CONTENT, % | DRY DENSITY, PCF | TYPE OF SAMPLER | SAMPLING RESISTANCE |
| 0 | | | | DS/CD<br>DS/CD | 600<br>1200 | 600<br>1060 | 9 | 116 | U | 15 |
| 10 | | | | DS/CD<br>DS/CD | 1500<br>3000 | 1340<br>2130 | 20 | 110 | U | 20 |
| 20 | | | | TX/UU | 1700 | 830 | 43<br>40 | 79<br>78 | P | 200 PSI |
| 30 | | | | | | | 22 | 104 | TW | 10 |
| 40 | | | | | | | | | | |
| 50 | -200 (79%) | | | | | | 19 | 110 | U | 100/6" |
| | -200 (15%) | | | | | | 20 | | SPT | 78 |
| 60 | | | | | | | 20 | 113 | U | 92/6" |
| | -200 (9%) | | | | | | 18 | | SPT | 97 |
| 70 | -200 (10%) | | | | | | 22 | 111 | U | 90/6" |
| | -200 (13%) | | | | | | 21 | | SPT | 119 |
| 80 | | | | | | | 18 | 116 | U | 68/6" |
| 90 | C | 83 | 23 | TX/UU | 3500 | 2740 | 31<br>33<br>32 | 90<br>89<br>91 | P | 600 PSI |
| 100 | | | | TX/UU | 4000 | 2160 | 42<br>44 | 81<br>77 | TW | 32 |
| 110 | | | | | | | | | | |
| 120 | C | 64 | 28 | TX/UU | 4600 | 2800 | 44<br>43<br>44 | 78<br>74<br>77 | P | 400 PSI |
| 130 | | | | | | | | | | |
| 140 | | | | TX/UU | 8300 | 4330 | 42 | 78 | P | 400 PSI |
| 150 | | | | | | | | | | |

| SYMBOLS | DESCRIPTION |
|---|---|
| GC | 3" ASPHALTIC CONCRETE & 4" CONCRETE |
| | BROWN GRAVELLY SAND WITH SOME CLAY (MEDIUM DENSE) |
| SP | BROWN FINE TO MEDIUM SAND WITH SCATTERED GRAVEL (MEDIUM DENSE)<br>(GRADING WITH SOME CRUSHED ROCK) |
| GW | DARK GRAY TO BLACK FINE TO COARSE GRAVEL & ROCK (MEDIUM DENSE) |
| SP | BLACK FINE SAND WITH TRACE OF SILT & SOME SHELLS (MEDIUM DENSE)<br>(GRADING CLAYEY) |
| CH | BLACK TO GRAY SILTY CLAY WITH TRACE OF ORGANICS (MEDIUM STIFF)<br>(GRADING WITH SOME SAND SEAMS) |
| SP/SC | GRAY FINE TO MEDIUM SAND WITH POCKETS OF GRAY SILTY CLAY & TRACE OF GRAVEL (DENSE) |
| CH | GRAY SILTY CLAY WITH TRACE OF SAND<br>(GRADING SANDIER)<br>(GRADING MORE CLAYEY) |
| SP | GRAY FINE SAND (VERY DENSE) |
| SP/SC | MOTTLED LIGHT GRAY & BROWN FINE SAND TO CLAYEY FINE SAND WITH SOME CEMENTATION (VERY DENSE) |
| SP | BROWN TO REDDISH-BROWN FINE SAND (VERY DENSE)<br>(GRADING BROWN & FINE TO MEDIUM) |
| SP/SM | BROWN & REDDISH-BROWN FINE SAND TO SILTY FINE SAND WITH TRACE OF CLAY (CEMENTED) (VERY DENSE) |
| SP/SC | GRAYISH-BROWN FINE SAND TO CLAYEY FINE SAND (PARTIALLY CEMENTED) (VERY DENSE)<br>(GRADING MORE CLAYEY) |
| SP/SM | DARK GRAY VERY FINE TO FINE SAND TO SILTY SAND (VERY DENSE) |
| CH | GREENISH-GRAY SILTY CLAY WITH SCATTERED LENSES OF SAND (VERY STIFF)<br>(GRADING WITH LAYER OF FINE SAND)<br>(GRADING WITH ORGANIC MATERIAL, WOOD & SHELL FRAGMENTS)<br>(GRADING SANDY)<br>(GRADING GRAY)<br>(GRADING SANDY)<br>(GRADING HARD)<br>(GRADING DARK GRAY) |

GENERAL NOTES:

1. ELEVATIONS REFER TO CITY OF SAN FRANCISCO DATUM
2. BLOW COUNTS SHOWN ARE FOR THE LAST 12 INCHES (OR PORTION THEREOF) OF A TOTAL 18 INCHES PENETRATION OF THE SAMPLER. THE U-SAMPLER WAS DRIVEN WITH SLIP-JARS WEIGHTING 340-POUNDS AND FALLING 18 INCHES. THE SPT-SAMPLER WAS DRIVEN WITH A 140 POUND HAMMER FALLING 30-INCHES.

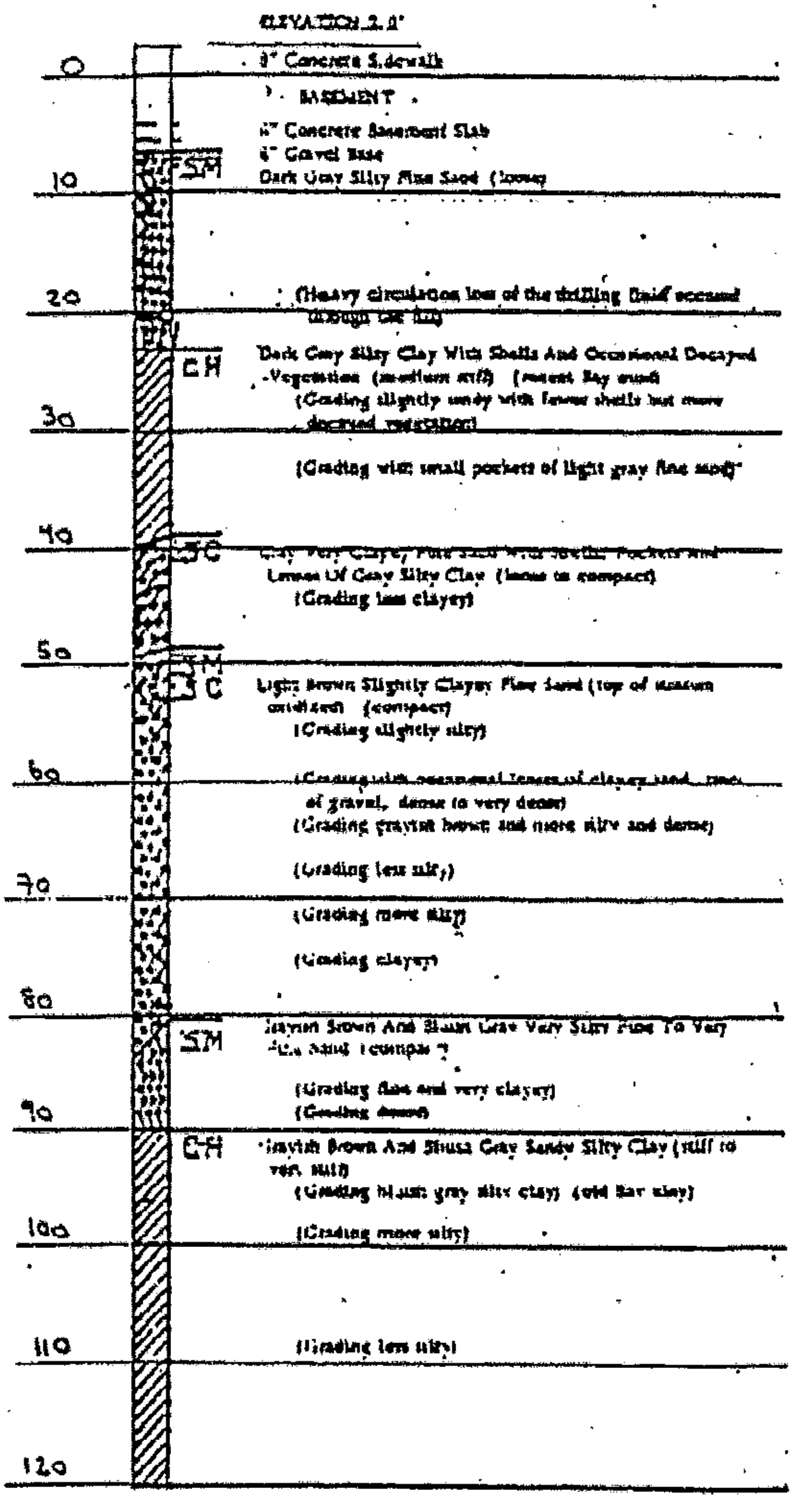
**301 MISSION STREET**
San Francisco, California

**Treadwell&Rollo**

## LOG OF BORING 1
## (BY DAMES & MOORE)

Date 01/12/05 | Project No. 3157.02 | Figure E-1

# BORING. NO 3



ELEVATION 2.0'

8" Concrete Sidewalk

BASEMENT

6" Concrete Basement Slab
6" Gravel Base
Dark Gray Silty Fine Sand (loose)

(Heavy circulation loss of the drilling fluid occurred through the fill)

Dark Gray Silty Clay With Shells And Occasional Decayed Vegetation (medium stiff) (recent Bay mud)
(Grading slightly sandy with fewer shells but more decayed vegetation)

(Grading with small pockets of light gray fine sand)

Gray Very Clayey Fine Sand With Shells, Pockets And Lenses Of Gray Silty Clay (loose to compact)
(Grading less clayey)

Light Brown Slightly Clayey Fine Sand (top of stratum oxidized) (compact)
(Grading slightly silty)

(Grading with occasional lenses of clayey sand, trace of gravel, dense to very dense)
(Grading grayish brown and more silty and dense)

(Grading less silty)

(Grading more silty)

(Grading clayey)

Grayish Brown And Bluish Gray Very Silty Fine To Very Fine Sand (compact)

(Grading fine and very clayey)
(Grading dense)

Grayish Brown And Bluish Gray Sandy Silty Clay (stiff to very stiff)
(Grading bluish gray silty clay) (old Bay clay)

(Grading more silty)

(Grading less silty)

Reference: Logs of Soil Borings, Sheet S7, by Dames & Moore, dated 21 February 1966.

**301 MISSION STREET**
San Francisco, California

Treadwell&Rollo

## LOG OF BORING 3
## (BY DAMES & MOORE)

Date 01/12/05 | Project No. 3157.02 | Figure E-2

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