GEOTECHNICAL INVESTIGATION
301 MISSION STREET
San Francisco, California

Millennium Partners
San Francisco, California

14 August 2001
Project No. 3157.01

Treadwell&Rollo
Environmental and Geotechnical Consultants

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Mr. Mark Farrar
Millennium Partners
720 Market Street, 9th Floor
San Francisco, California 94102

Subject: Geotechnical Investigation
301 Mission Street
San Francisco, California

Dear Mr. Farrar:

Treadwell & Rollo, Inc. is pleased to present this geotechnical investigation report for the proposed 301 Mission Street project in San Francisco. Copies have been distributed as indicated at the end of this report.

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 155 feet below the existing ground surface. We recommend the structure be supported on a pile foundation gaining support in the sand and clay below the Marine Deposits, 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
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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation 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; they are 129 Fremont Street, 124 Beale Street, 301 and 345 Mission Street, as shown on Figure 2.

Concurrently with our geotechnical investigation, we performed environmental services which consisted of a Phase I Environmental Site Assessment, an Asbestos and Lead Containing Building Material Survey and an Environmental Site Characterization for the project site. The results of our environmental services are presented in a separate report.

2.0 PROJECT DESCRIPTION

Plans by Gary Edward Handel + Associates, the project architect, show the proposed development consists of constructing a 52-story tower comprised of retail and living space, a 12-story structure for office and living space, a 5-story structure for retail and office space, and a 5-story-high atrium and lobby. Three levels of underground parking will occupy the entire project site. The underground parking will extend about 35 feet below the ground surface. On the basis of the available topographic information, we estimate the finished floor of the lowest

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

level of the parking garage will be at about Elevation -32 feet[2]. The footprints of the proposed buildings and the parking garage area are shown on Figure 3.

3.0 SCOPE OF SERVICES

The scope of our geotechnical services was outlined under Task 3 of our proposal dated 16 May 2001. We performed all the services described under Task 3 with the exception of site-specific response spectra, approval of which is still pending.

We evaluated subsurface conditions at the site by drilling five borings, performing laboratory tests and performing engineering analyses to develop 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
- 1998 San Francisco Building Code near-source and site factors
- construction considerations

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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.

4.0 FIELD INVESTIGATION

Prior to performing the field investigation, we reviewed available subsurface information from previous geotechnical investigations performed in the site vicinity, including:

- *Geotechnical Investigation, Proposed Five Fremont Center Project, San Francisco, California, for Metropolitan Bechtel Shorenstein*, prepared by Dames & Moore, dated 13 March 1981
- *Geotechnical Investigation, 350 Mission Street Building Seismic Strengthening, San Francisco, California,* prepared by Treadwell & Rollo, Inc., dated 3 July 1997
- *Geotechnical Investigation, First and Howard Project, City Block No. 3720, San Francisco, California,* prepared by Treadwell & Rollo, Inc., dated 6 July 1999
- *Geotechnical Investigation, 199 Fremont Street, San Francisco, California,* prepared by Treadwell & Rollo, Inc., dated 11 March 1998

We also 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).

4.1 Borings Performed for This Investigation

To evaluate subsurface conditions beneath the site, we drilled five exploratory borings (designated as B-1 through B-5) at the approximate locations shown on Figure 2. Because of the presence of existing buildings at the site, and underground utility and overhead obstructions on the adjacent streets, borings were drilled within the vacant lot only (see Section 6.1).

The borings were drilled to depths ranging from 60.5 to 155.5 feet below the existing ground surface between 26 June and 3 July 2001. 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. Due to complications during drilling, boring B-2 could not be drilled to its planned depth of 80 feet. A sampler was lost in the hole, and therefore the hole was terminated at a depth of 32.5 feet and grouted. Boring B-2b was drilled adjacent to boring B-2 to the planned depth of 80.5 feet below ground surface.

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-6. The material encountered was classified according to the soil classification system described on Figure A-7.

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 D for copies of their 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, fines content (percent passing the No. 200 sieve), 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-10.

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 a 6-story concrete/brick building with one basement at 301 Mission Street, a 6-story concrete building with one basement at 124 Beale Street, and 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 a vacant lot. The structure was demolished and the vacant lot was created by filling the basement with rubble and building 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.

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 located 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 Mayer 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 consisting of about five to eleven 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 17 and 15 feet below ground surface, respectively. 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 43 to 44 feet below the site grade, corresponding to Elevations ranging from -40 to -40.5 feet. Based on Dames & Moore data, 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].

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 and B-5.

The sandy soil is underlain by stiff to hard clay and sandy clay, locally known as Old Bay Clay, to the maximum explored depth of 155.5 feet (Elevation -152 feet). Consolidation tests performed on a representative sample of the clay indicate the soil is overconsolidated.

6.3 Groundwater

The groundwater level in our geotechnical soil borings was generally obscured by the drilling fluid, and because of requirements to backfill the borings immediately after drilling, groundwater

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

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 Elevations ranging from -9 to -11.5 feet. The approximate elevations where groundwater was encountered is noted next to the 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 at the project site 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. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

7.1 Regional Seismicity

The major active faults in the area are the San Andreas, Hayward, San Gregorio, Rodgers Creek 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 Moment magnitude[4]

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

(Working Group on California Earthquake Probabilities (WGCEP) 1999 and California Division of Mines and Geology (CDMG) 1996) event are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
San Andreas (1906 Event)	14	Southwest	7.9
San Andreas (Peninsula)	14	Southwest	7.2
Hayward (Total)	15.5	Northeast	7.1
Hayward (North)	15.5	Northeast	6.6
San Andreas (North Coast South)	18	West	7.5
San Gregorio (North)	19	West	7.3
Hayward (South)	19	East	6.9
Mount Diablo Thrust	32.5	East	6.7
Rodgers Creek	33.5	North	7.1
Calaveras (North of Calaveras Reservoir)	34	East	7.0
Concord	37.5	Northeast	6.5
Green Valley (South)	40	Northeast	6.5
Monte Vista	41	South	6.8
Point Reyes	42	Northwest	6.8
West Napa	43.5	North	6.5
Greenville (North)	44.5	Northeast	6.6

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 1999, the Working Group on California Earthquake Probabilities (WGCEP 1999) at the U.S. Geologic Survey (USGS) predicted a 70 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2030. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2

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

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

7.2 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[5] and differential compaction[6]. 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

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[5] 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.

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

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. Because the site will be excavated to accommodate the basement levels, the loose to medium dense sand encountered in the upper 35 feet will be removed within the building footprint. Therefore, settlement from differential compaction will not occur below the foundation level. However, layers of saturated, loose to medium dense sand exist below the proposed basement excavation, within the Marine Deposits and the dense sand layer below.

The results of our analyses indicate the saturated, loose to medium dense clayey and silty sand encountered below the proposed excavation is 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 occur beneath the building footprint.

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 could disrupt utilities and damage sidewalks and streets.

7.3 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 Deposits below the entire site
- the depth of excavation for the three basement levels
- the presence of Marine Deposits at the proposed base of excavation
- the presence of groundwater at a level higher than the proposed excavation depth

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

We considered deep (driven piles) and shallow (mat) foundations for the support of the proposed 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 basements. In addition, a large amount of the Marine Deposits below the fill will also be removed. However, Marine Deposits will be exposed at the base of the planned excavation and are unsuitable for support of a mat foundation. In addition, the medium dense sandy layers encountered within the dense sand underlying the Marine Deposits 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 structure.

On the basis of the results of our analyses and evaluation, we conclude the proposed structure should be supported on driven 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 are the most appropriate pile types for the project. Although driven piles will transfer building loads to less compressible strata, some minor settlement (less than one inch) of the pile foundations will still occur.

8.2 Construction Considerations

The main construction considerations are dewatering for the basement excavation (which will extend about 33 feet below the groundwater level) and shoring requirements. Additional concerns are the presence of concrete rubble and debris in the fill, and the presence of Marine Deposits exposed at the proposed base of the basement excavation. These issues are discussed in the following sections.

8.2.1 Shoring

We understand the finished floor for the lowest basement level will be about 35 feet below existing ground surface, corresponding to Elevation -32 feet. Assuming excavations for proposed pile caps will extend another four feet, we estimate construction of the below-grade garage slab and pile caps will require excavations extending about 39 feet below the existing ground surface (maximum estimated excavation depth). Because there is insufficient space to slope the sides of the deep excavations, 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) walls
- 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.

We conclude soldier pile and lagging or sheet piles with internal bracing may be appropriate shoring systems. However, it would likely be difficult to drive the sheet piles through the fill due to the presence of concrete and brick debris, and it may not be possible to drive the sheet piles through the dense to very dense sand layer encountered below a depth of 44 feet, corresponding to Elevation -41 feet.

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. Tiebacks will have low capacities in the fill and Marine Deposits that extend to approximately Elevation -41 feet, and therefore will be impractical. Internal bracing can be either cross-lot or inclined rakers.

We conclude that soldier pile and lagging, SPTC and soil/cement walls are viable options. 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

Available project information indicates the finished floor for the lowest basement level will be at about Elevation -32 feet. Excavations for proposed pile caps will extend on the order of another four feet for an estimated maximum excavation depth at about Elevation -36 feet. Consequently, excavations for the below-grade garage slab and pile caps will extend about 33 feet below the anticipated high groundwater level, estimated at Elevation -3 feet. The groundwater level at the site should be lowered to a depth of at least three feet below the bottom of the planned maximum excavation 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 system can be stopped.

The efficiency of the dewatering system will depend to some extent on the type of shoring system used. For example, an SPTC wall would likely be the most water-tight and thus require the least 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 likely 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 for a properly designed and constructed shoring system should be within about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of adjacent structures during excavation. This monitoring system should provide timely data which can be used to modify the shoring system if needed.

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 drilled and cased through the fill prior to driving the piles. Predilling will help maintain pile alignment, and reduce pile damage and heave of adjacent improvements. A follower capable of driving the piles to cutoff elevations would also be required.

8.2.5 Unstable Subgrade

Saturated, soft to medium stiff clay and loose to medium dense sand may be encountered at the subgrade level. 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 about 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 foundation design, garage slabs, and lateral earth pressures for retaining walls 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, and other obstructions will be encountered during shoring installation and excavation within the sandy fill.

Onsite 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[7]. 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 subgrade 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 driven pile foundations consisting of 14-inch-square prestressed precast concrete piles be used to support the proposed structure. 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. Axial, uplift, and lateral pile capacities for the recommended piles are presented in the following subsections.

9.2.1 Axial Pile Capacity

We recommend the 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 conditions. The recommended pile capacity relates only to pile support and the structural capacity of the pile should be checked.

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

Because the density of the sand layer varies across the site, precise 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. We also recommend additional field investigation in the eastern half of the site, as discussed in Section 9.10, to establish the elevation of the top of the bearing layer in the areas of the site currently occupied by buildings (i.e. 301 Mission Street, 124 Beale Street and 129 Fremont Street).

For the proposed finished garage slab elevation and assuming a four-foot-thick pile cap (pile cutoff at Elevation -36 feet), we estimate lengths for end bearing piles will range from approximately 30 to 50 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 capacities due to group effects.

Based on the available subsurface information and our experience, we expect some piles may not meet refusal. 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.

9.2.2 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 -66 feet. Figure 10 was developed for 30-foot long piles, with a cutoff Elevation at -36 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 diameters in the direction of loading, the single pile lateral capacities should be reduced. Reduction factors, corresponding to the number of piles in a group, for three pile diameter center to center spacing, are given in Table 3. We can provide lateral load analyses for pile groups when the arrangement, number, and spacing of piles have been established.

TABLE 3
Pile Group Reduction Factors for Three Pile Diameter Center to Center Spacing

Number of Piles in Pile Group	Reduction Factor
2	0.84
3 and 4	0.83
5	0.82
6	0.73

Because the potential for liquefaction exists, passive pressure resistance adjacent to below ground elements, including pile caps, should not be accounted for in the design.

9.2.3 Indicator Pile Program

Before production piles are cast, we recommend at least 25 indicator piles be driven to observe the driving characteristics of the piles and the performance of the driving equipment. 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 to correlate with information obtained from the test borings, to aid in evaluating predrilling requirements 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 production piles. Pile reinforcement 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 driven from current grade, the pile location should be drilled or excavated with a diameter larger than the diameter of the follower for a depth extending from the indicator pile-driving grade to the pile cutoff elevation. Indicator piles should be driven with the same equipment that will be used to drive production piles so that appropriate practical refusal blow count relationships can be established.

We recommend performing a Wave Equation Analysis of Pile (WEAP) for the proposed 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 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.

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 50,000 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.

9.2.5 Vibration Monitoring

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 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[8]	60 pcf	40 pcf + 15H psf
Below the water table	90 pcf	85 pcf + 15H psf

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 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.

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

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.

9.4 Basement Floor Slab

Because of the potential for liquefaction-induced settlement of the sandy strata beneath the site, and the need to resist hydrostatic uplift, we recommend a structurally supported slab be used for the basement floor. A structural slab can reduce the detrimental effects of liquefaction on both architectural and structural elements; it may also be used to tie the lateral support system together. The slab should be designed for the anticipated traffic loads and be designed to span between pile caps and grade beams.

The basement slab will extend below groundwater level and should therefore be appropriately waterproofed. Design for hydrostatic uplift pressures should assume a design groundwater at Elevation -3 feet. We recommend the waterproofing be 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 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 slab-on-grade 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 Seismic Design

For seismic design in accordance with the 1998 San Francisco Building Code, we recommend using soil profile type S_D. The site is about 14 kilometers from the San Andreas Fault, a type A fault; hence near-source factors N_a=1.0 and N_v=1.04 should be used.

9.6 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.

9.7 Shoring

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

A soldier pile and lagging shoring system with internal bracing can be designed using the lateral earth pressures presented on Figure 11. In calculating these pressures, we assumed the excavation will be dewatered so that the groundwater level is at least three feet below the base of the excavation. SPTC and mixed-in-place soil/cement walls with internal bracing can be designed using the lateral earth pressures presented on Figure 12.

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 pressures presented on Figures 11 and 12.

Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. Soldier-pile-and-lagging shoring is a flexible system. Therefore, the deflection of the shoring and, consequently, the settlement of the adjacent ground surface is a concern. Additional settlement can occur due to: 1) sloughing of sand prior to placement of timber lagging, and 2) erosion of sand through gaps between lagging. To reduce the potential for additional settlement to occur, we recommend the shoring contractor consider: 1) placing timber lagging as quickly as possible during excavation (exposed excavated surface should not be more than 2 feet high before placing lagging), 2) placing geotextile behind lagging, and 3) injecting grout behind the lagging if sloughing occurs prior to placing lagging.

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 structural 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 our office and the project structural engineer's office review temporary shoring plans. In addition, we recommend a representative from our office observe the installation of the temporary shoring system.

9.8 Dewatering

The groundwater should be drawn down so that the piezometric level in the sand layers below the base of the excavation is at least three feet below the bottom of the maximum proposed excavation. This level 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 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.

9.9 Construction Monitoring

The contractor should establish survey points on the shoring and on adjacent streets and buildings within 50 feet of the excavation perimeter prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding facilities during construction. In addition, a thorough crack survey of the adjacent structures should be performed by a surveyor prior to the start of construction and immediately after its completion. The need for a ground movement monitoring program using inclinometers will be evaluated once the shoring system has been selected.

9.10 Additional Subsurface Investigation

Because of the presence of existing structures onsite and obstructions on adjacent streets, the subsurface conditions beneath the eastern half of the project site could not be investigated. We therefore recommend the subsurface conditions within the eastern half of the site be investigated by drilling additional borings once the existing structures have been demolished. We anticipate four to five borings will be adequate to provide the necessary subsurface information. The final planning for the additional investigation can be arranged once the project enters its construction and demolition phases.

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 level observation wells, shoring system, indicator and production piles, and placement and compaction of any backfill. These observations will allow us to compare actual with anticipated soil conditions and verify that the contractors work conforms with 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.

REFERENCES

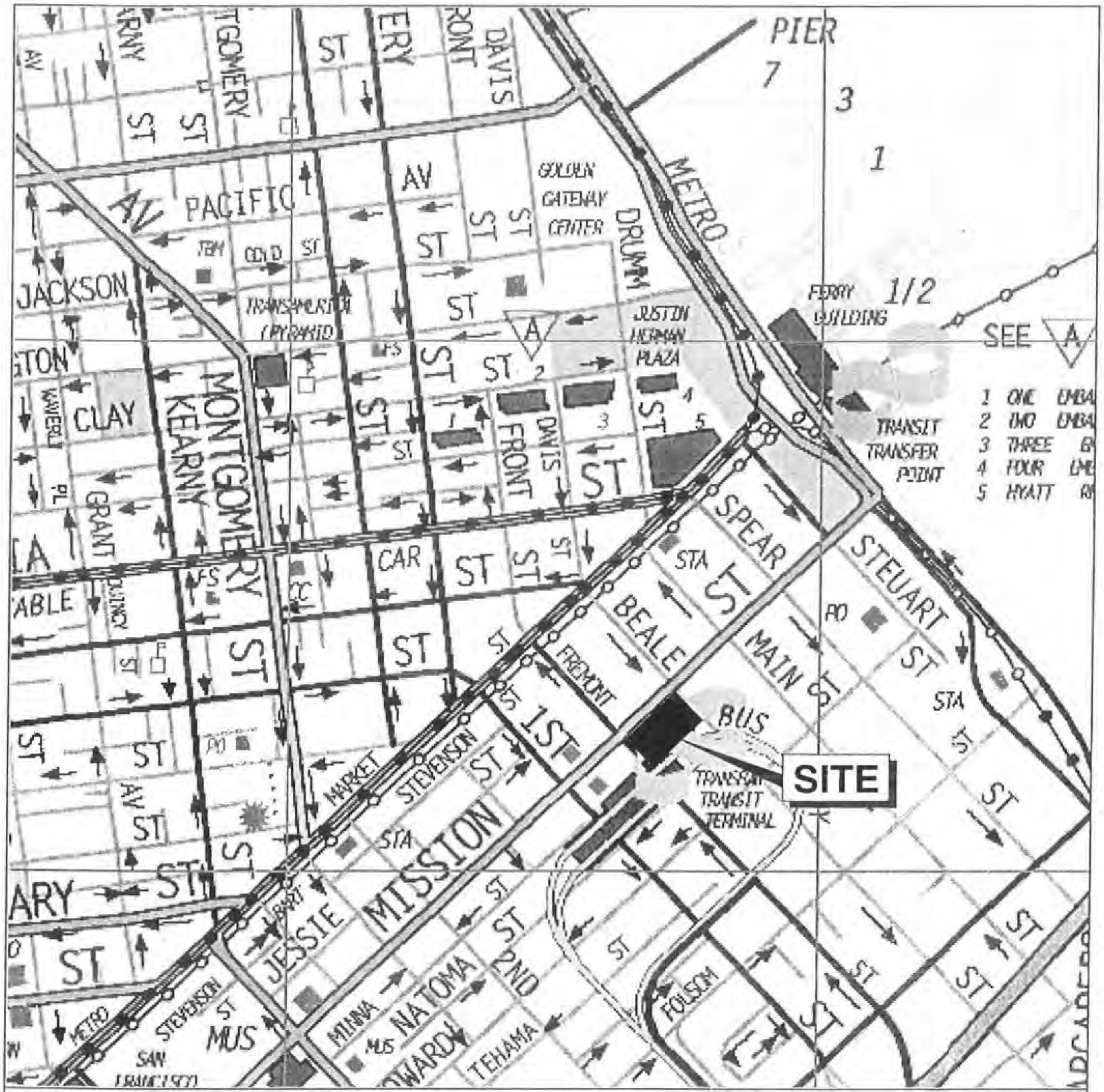
California Division of Mines and Geology (1996). "Probabilistic seismic hazard assessment for the State of California." DMG Open-File Report 96-08.

California Division of Mines and Geology (2000). "State of California Seismic Hazard Zones, Zones of Potential for Liquefaction", City and County of San Francisco."

Toppozada, T. R. and Borchardt G. (1998). "Re-Evaluation of the 1836 "Hayward Fault" and the 1838 San Andreas Fault earthquakes." *Bulletin of Seismological Society of America*, 88(1), 140-159.

Working Group on California Earthquake Probabilities (WGCEP) (1999). "Earthquake probabilities in the San Francisco Bay region: 2000 to 2030 – A summary of findings." Open File Report 99-517.

FIGURES



Base map: The Thomas Guide
San Francisco County
1999

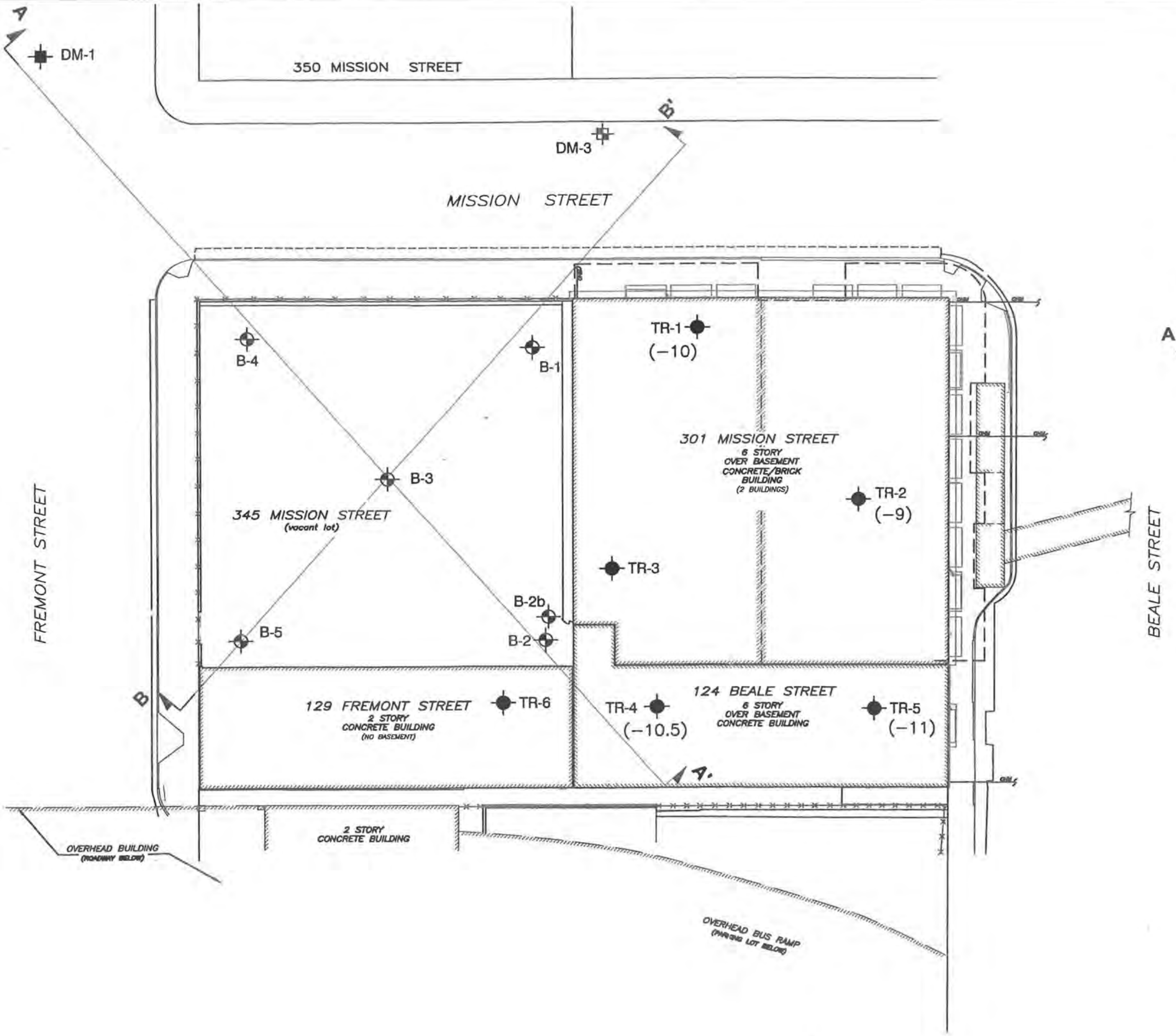
No scale

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

SITE LOCATION MAP

Date 08/08/01	Project No. 3157.01	Figure 1



EXPLANATION

B-1 ⊕	Approximate location of soil boring by Treadwell & Rollo, Inc., June to July 2001
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
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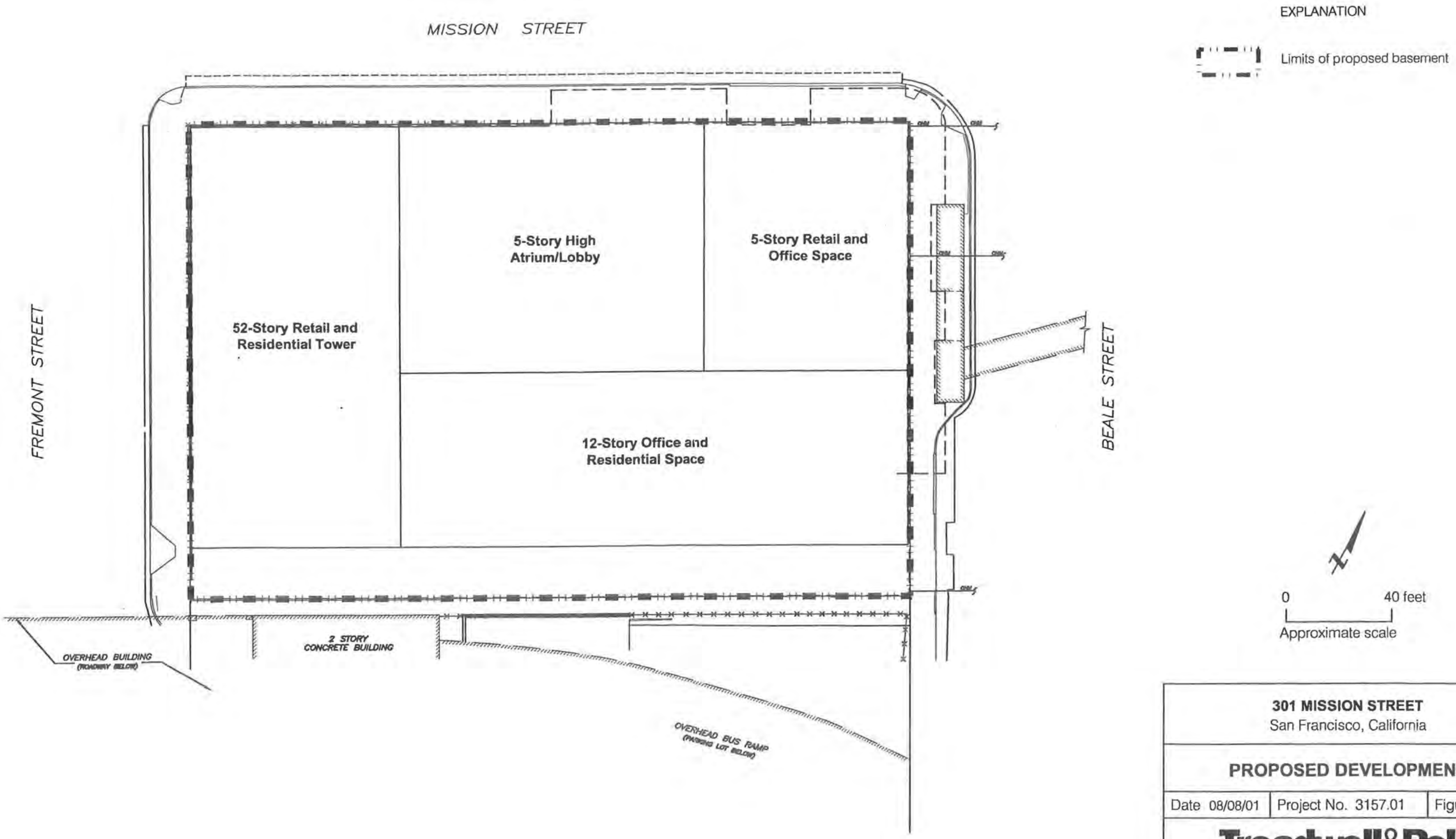
EXISTING CONDITIONS WITH BORING LOCATIONS

Date 08/08/01	Project No. 3157.01	Figure 2

Treadwell&Rollo

Existing Conditions with Boring Locations.dwg



301 MISSION STREET
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PROPOSED DEVELOPMENT

Date 08/08/01	Project No. 3157.01	Figure 3

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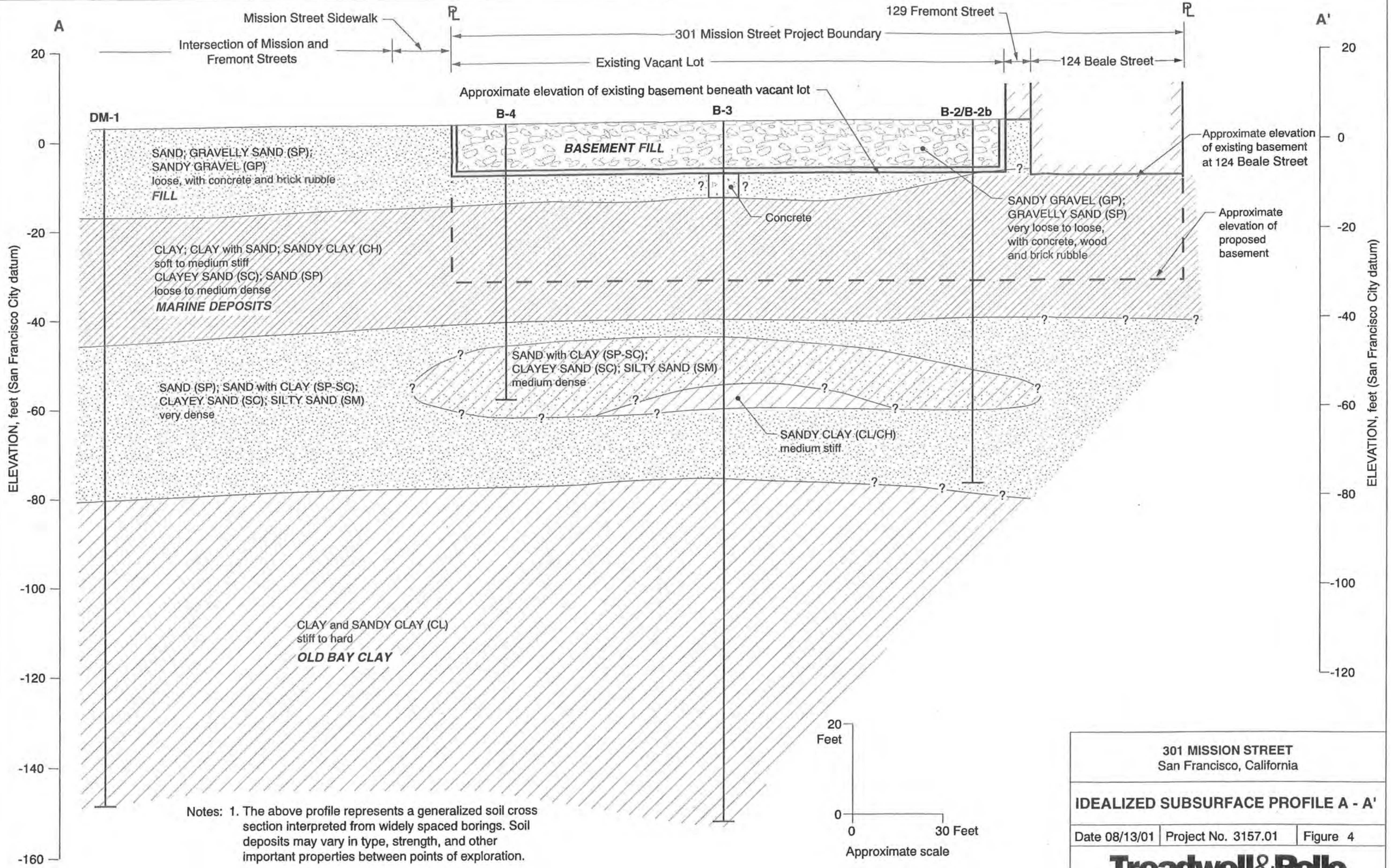
PROPOSED DEVELOPMENT.DWG



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
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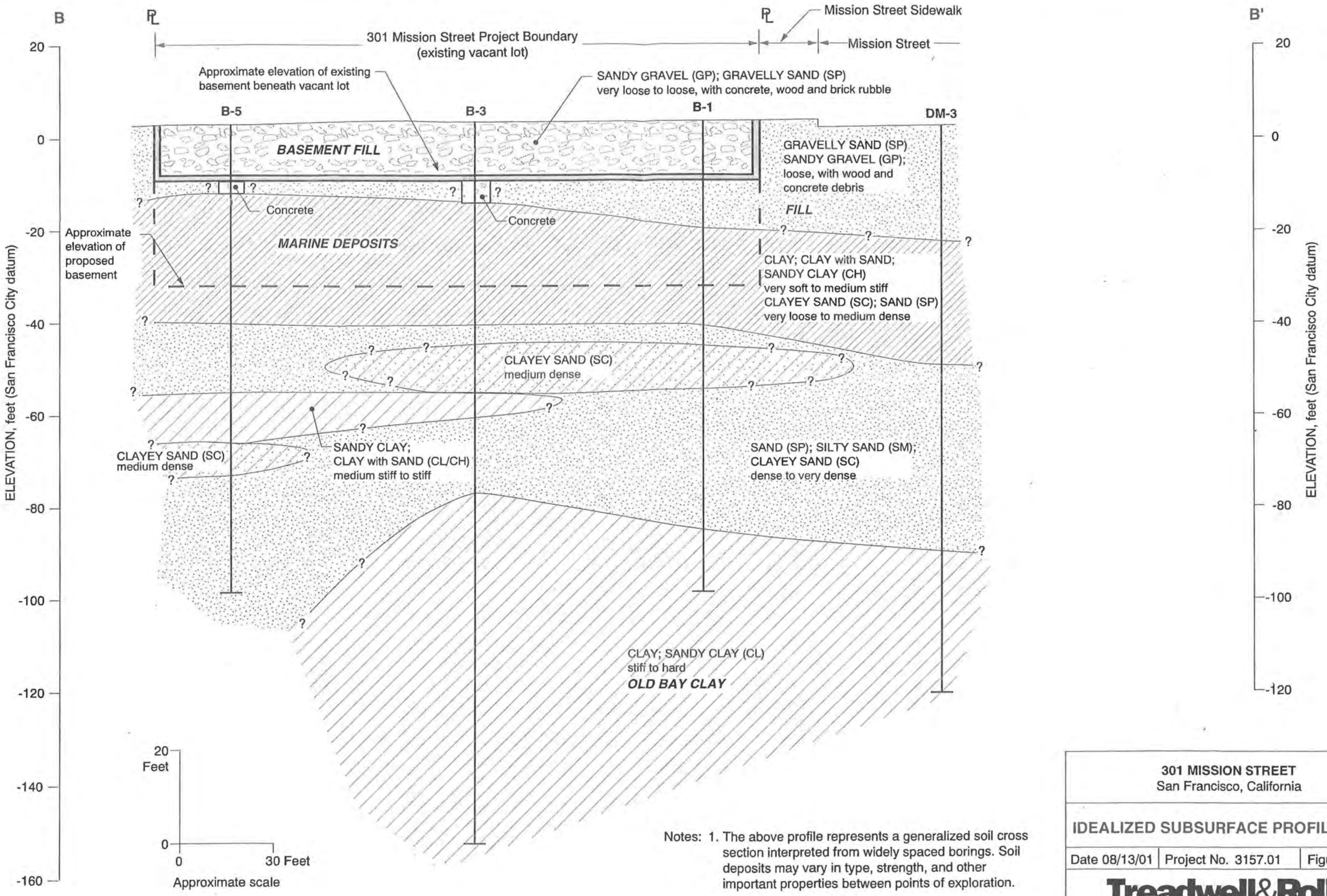
IDEALIZED SUBSURFACE PROFILE A - A'

Date 08/13/01 | Project No. 3157.01 | 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.

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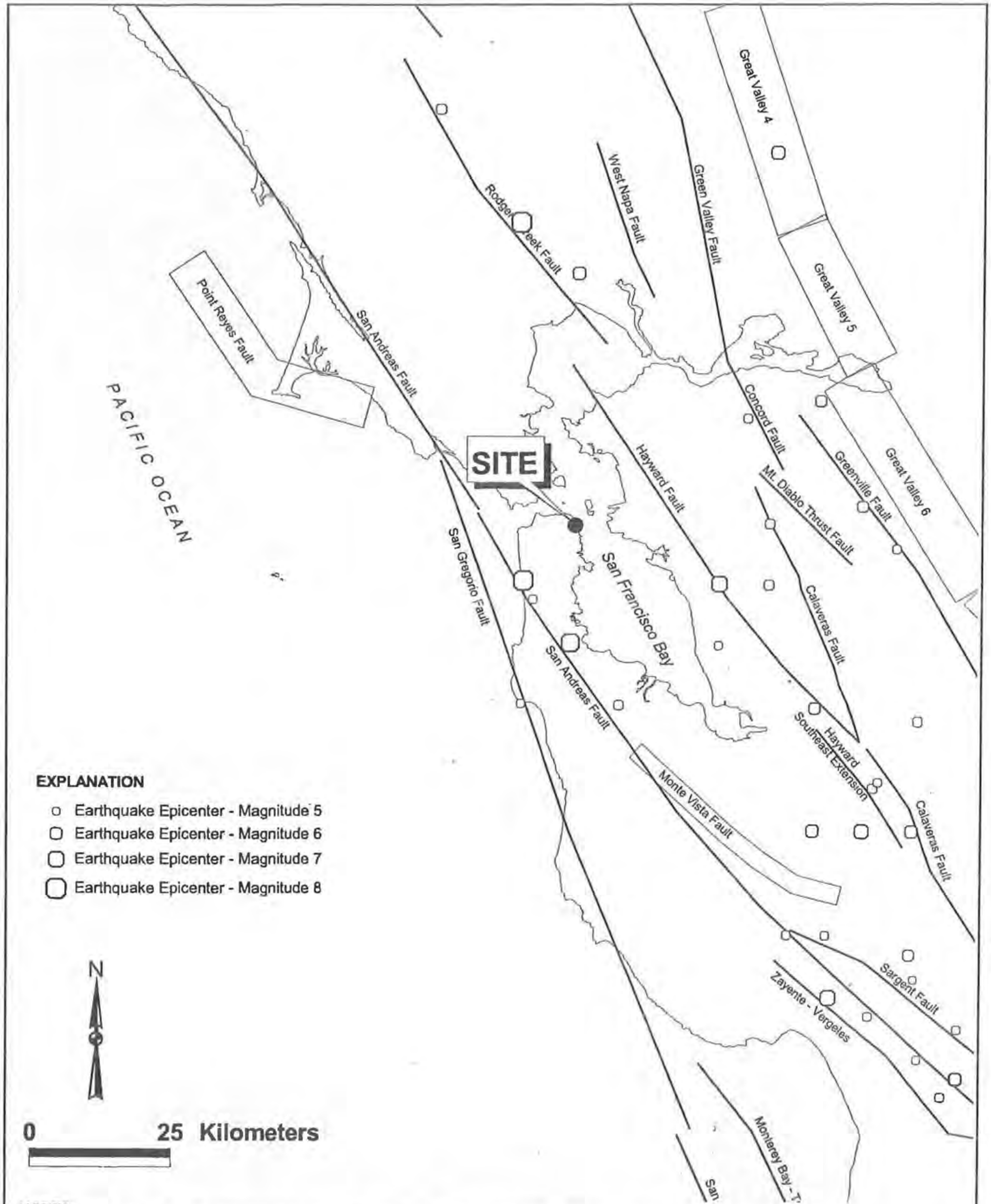
IDEALIZED SUBSURFACE PROFILE B - B'

Date 08/13/01	Project No. 3157.01	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 January 1996.

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MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Treadwell&Rollo

Date: 08/08/01	Project No. 3157.01	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.

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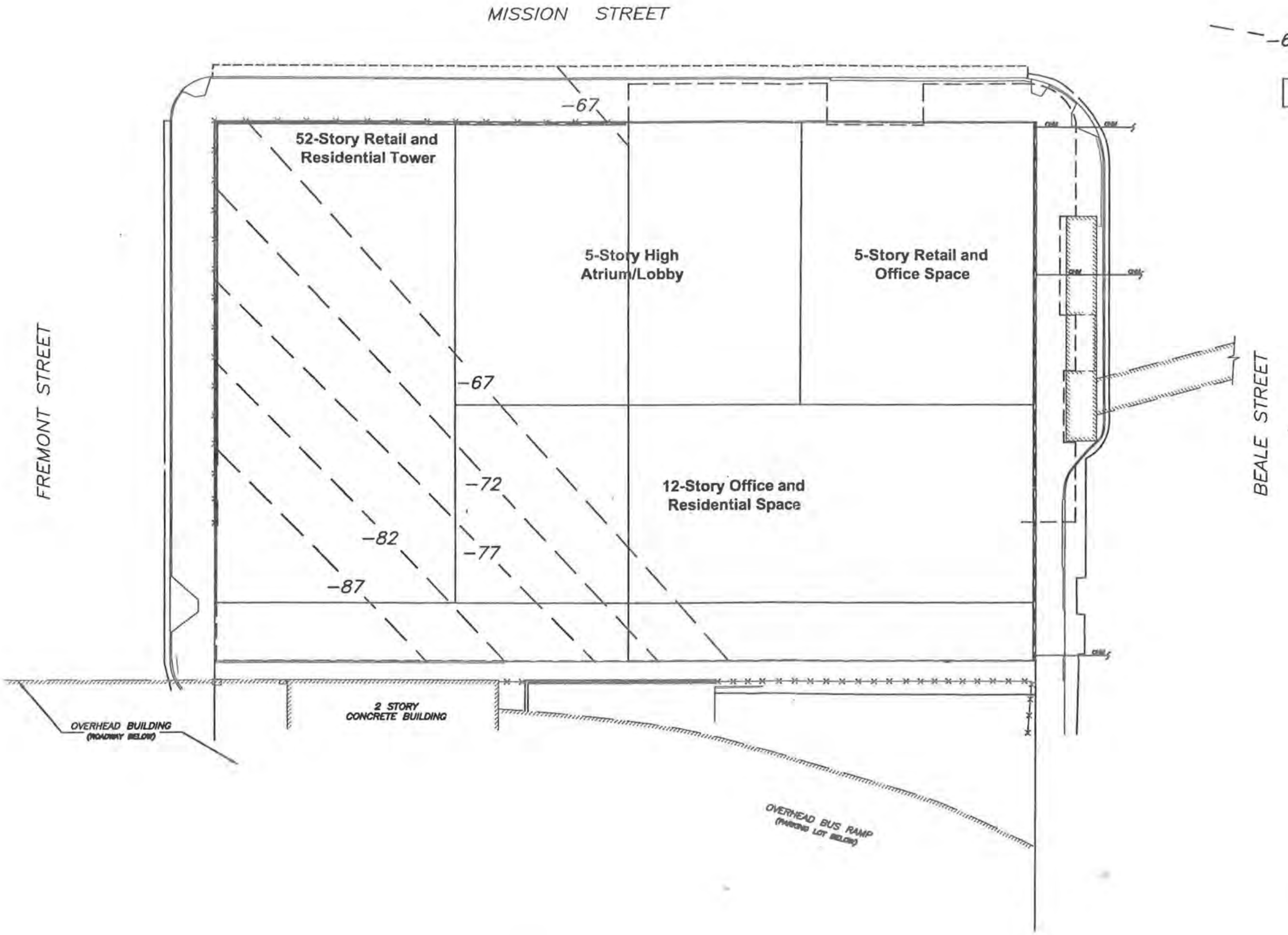
San Francisco, California

MODIFIED MERCALLI INTENSITY SCALE

Treadwell&Rollo

Date 08/08/01 | Project No. 3157.01 | Figure 7



EXPLANATION

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

▭ A more accurate estimate of the top of bearing layer in this zone should be made by an additional subsurface investigation after the existing structures have been demolished.

NOTE:

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

0 40 feet

Approximate scale

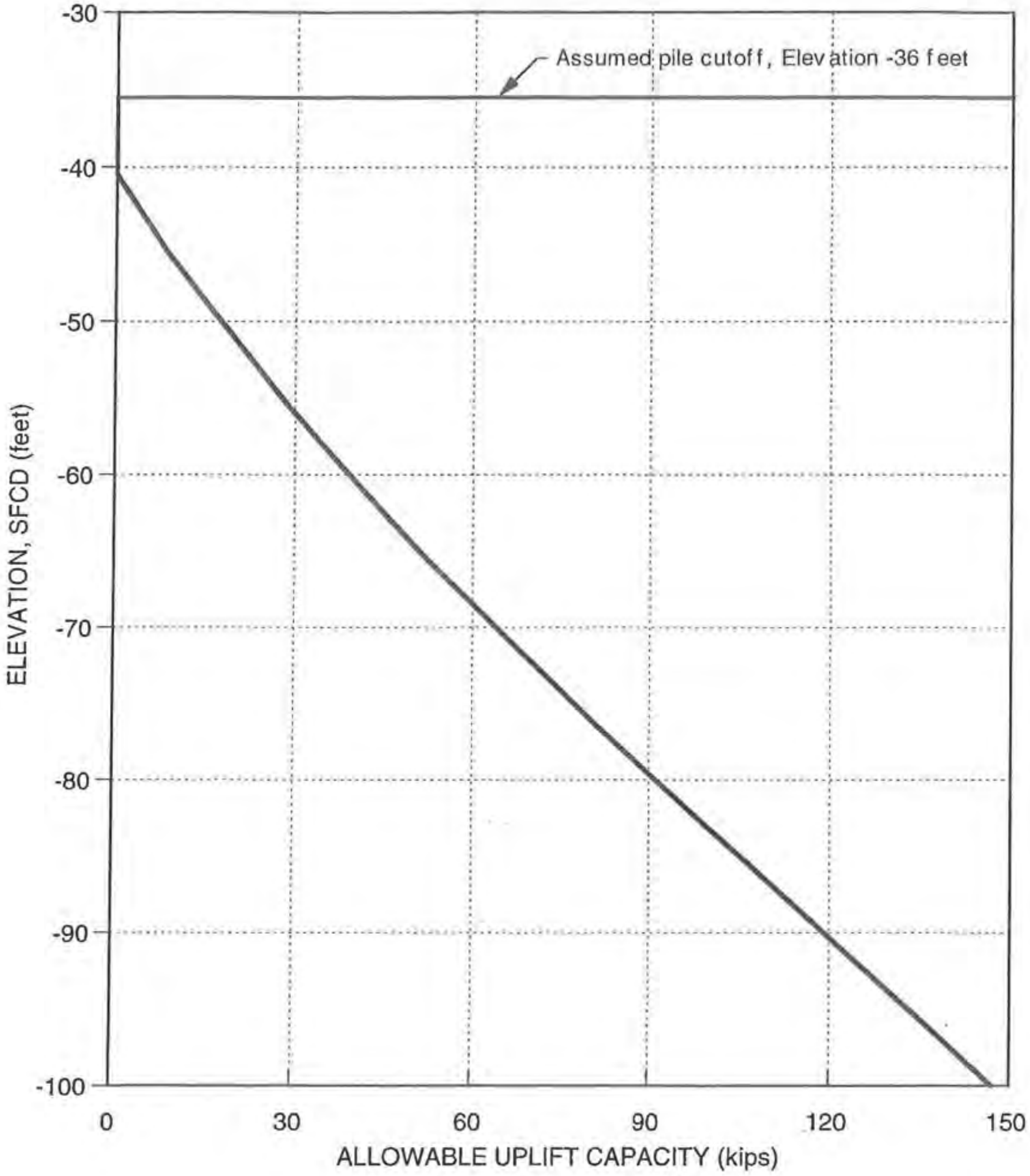
301 MISSION STREET
San Francisco, California

TOP OF BEARING LAYER CONTOURS

Date 08/13/01	Project No. 3157.01	Figure 8

Treadwell&Rollo

TOP OF CONTOURS.DWG



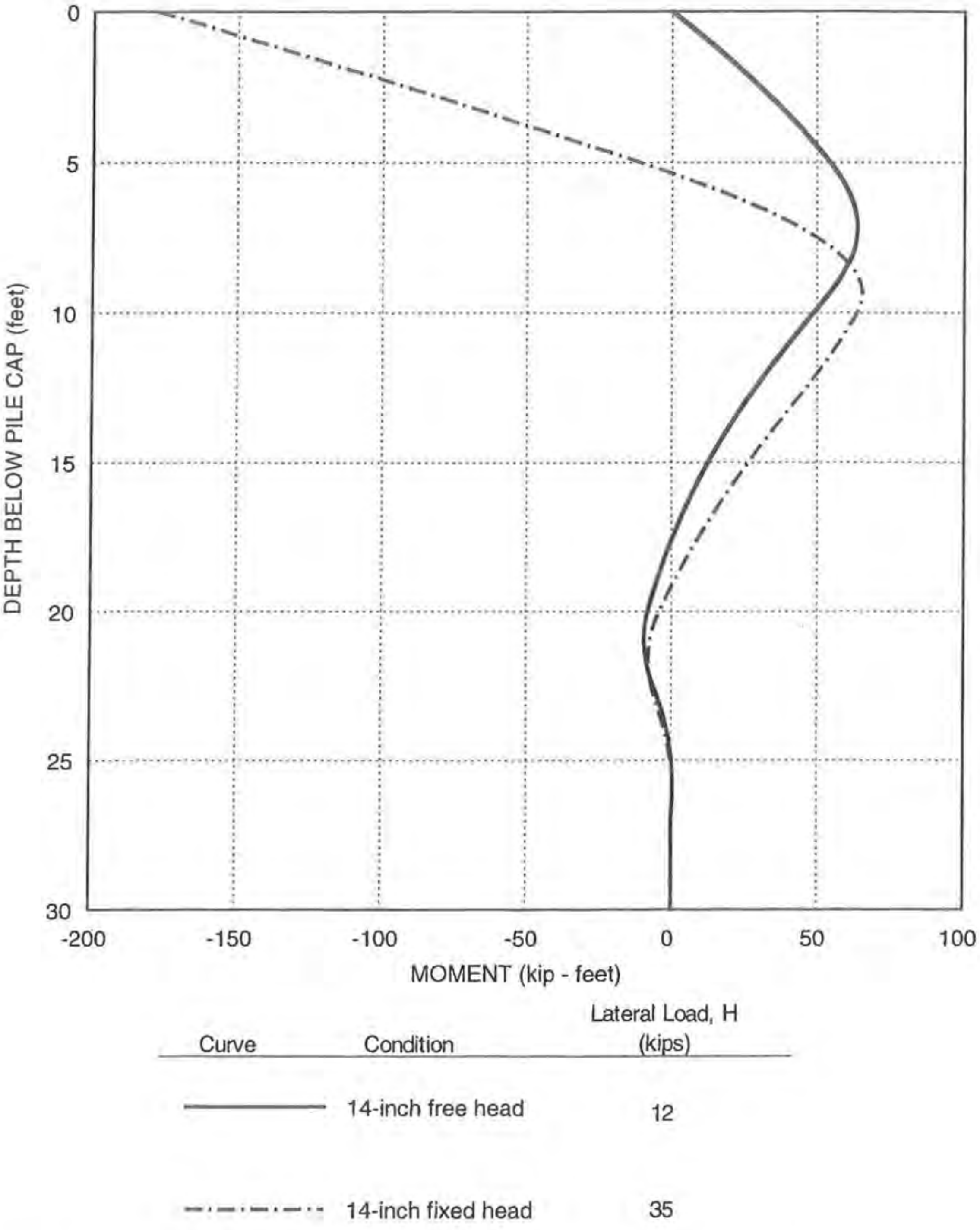
Notes:
1. The indicated capacities are based on the shear strength of the supporting soil; the structural capacity of the pile may govern.
2. Piles should be spaced no closer than three diameters center to center.
3. City and County of San Francisco datum.

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

PILE UPLIFT CAPACITIES
FOR SUSTAINED LOADS

Date 08/08/01 | Project No. 3157.01 | Figure 9



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 diameters; for spacing less than 8 diameters, see Section 9.2.2 of report.
4. Assumes there is no applied moment at the pile head.

301 MISSION STREET
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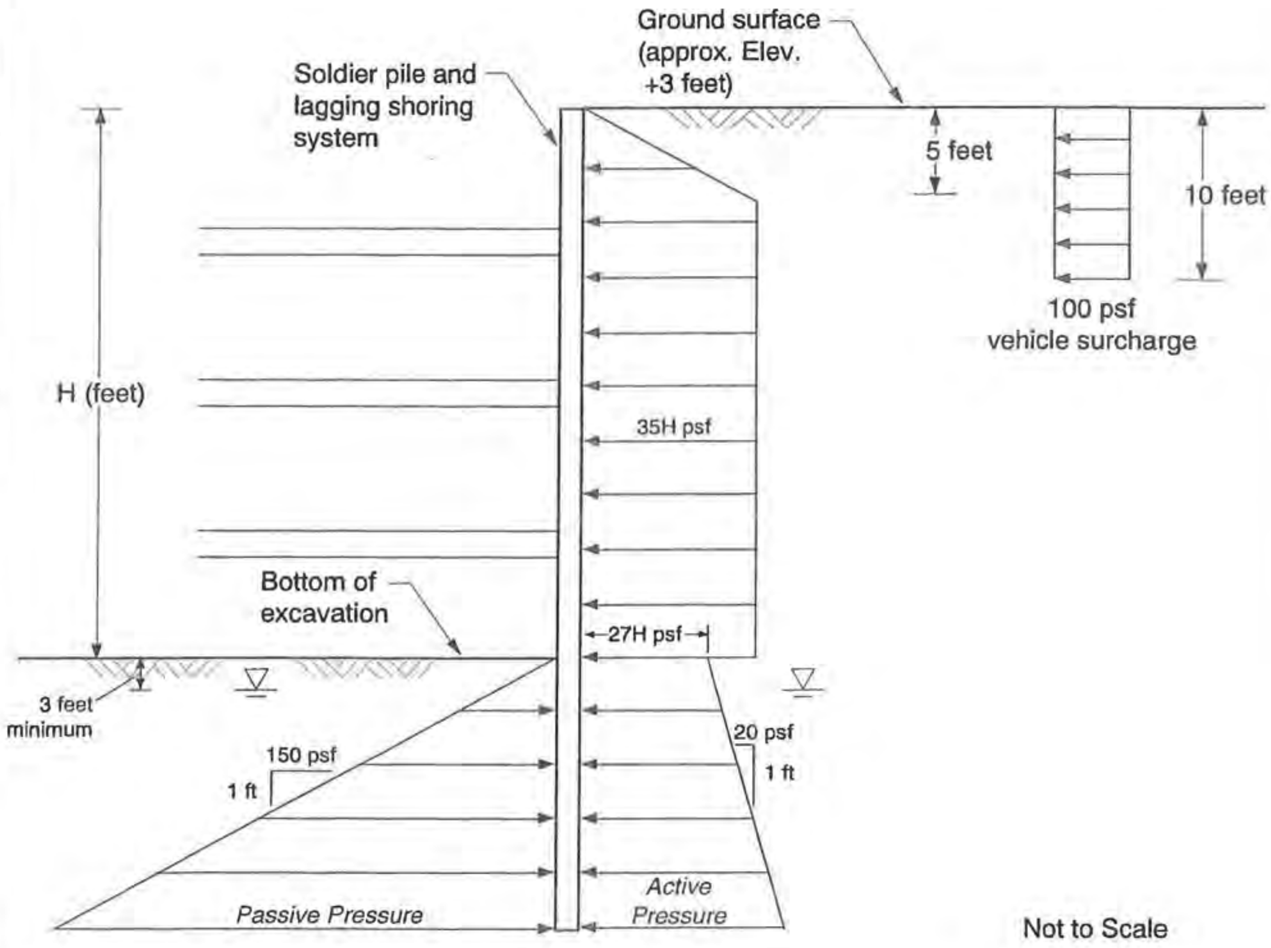
BENDING MOMENT PROFILE FOR
14-INCH SQUARE
PRECAST-PRESTRESSED CONCRETE PILES

Date 08/13/01 | Project No. 3157.01 | Figure 10



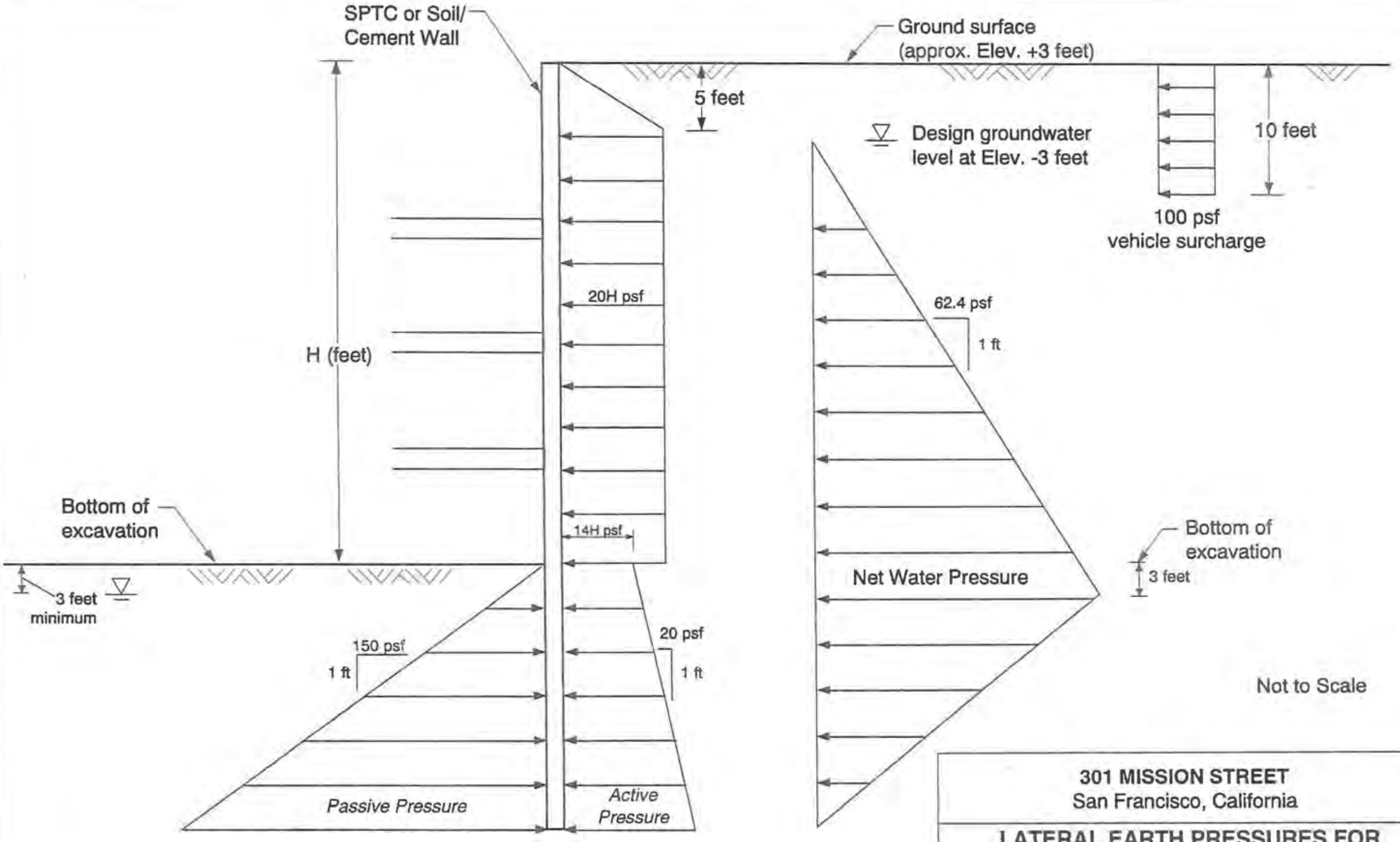
Notes:
1. Groundwater level lowered to at least 3 feet below base of excavation.
2. Passive pressure values include a factor of safety of 1.5.
3. Water pressures of the active and passive pressures cancel each other out and are not shown on the diagram.
4. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
5. Active pressures below the base of the excavation should be assumed to act over one pile diameter.

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

LATERAL EARTH PRESSURES FOR SOLDIER PILE AND LAGGING SHORING SYSTEM WITH INTERNAL BRACING

Date 08/14/01	Project No. 3157.01	Figure 11



301 MISSION STREET
San Francisco, California

LATERAL EARTH PRESSURES FOR
SPTC OR SOIL/CEMENT WALL
SHORING SYSTEM WITH INTERNAL BRACING

Date 08/13/01 | Project No. 3157.01 | Figure 12

Treadwell&Rollo

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 include a factor of safety of 1.5.

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)

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Surface Elevation: 3.5 feet[2]		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
1					SANDY GRAVEL (GP) light brown, loose, dry, with concrete and brick debris	↑						
2												
3												
4												
5				GP								
6												
7												
8												
9												
10												
11					CONCRETE SLAB 6-inches thick	FILL						
12					SANDY GRAVEL (GP) light brown, loose, moist, with wood and concrete debris unstabilized groundwater level at 13 feet noted during drilling							
13	S&H	▨	30/3"	∇								
14												
15												
16												
17				GP								
18												
19												
20												
21	S&H	●	4									
22												
23					CLAY with SAND (CH) gray, very soft to soft, wet, with shells	↓						
24												
25												
26	O	▨	50 psi	CH							52.9	69
27												
28												
29												
30												

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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	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 with SAND (CH) (continued)						
31	S&H		0								
32											
33											
34											
35											
36	O		50 psi			TxUU	1,400	685		37.1 35.0	85 87
37				CH	Consolidation Test, See Figure C-8						
38											
39											
40										48.4	72
41	S&H		2								
42											
43											
44					SAND (SP) gray, very dense, wet						
45											
46	SPT		51	SP							
47											
48					CLAYEY SAND (SC) 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) olive-gray, dense, wet						
59				SC							
60											

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

Log of Boring B-1

PAGE 3 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
					CLAYEY SAND (SC) (continued)						
61	S&H	▨	43							16.2	119
62											
63				SC							
64											
65											
66											
67					CLAYEY SAND (SC) yellow-brown, very dense, wet						
68											
69											
70	S&H	▨	30/5"								
71											
72				SC							
73											
74											
75											
76											
77											
78					SILTY SAND (SM) 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]** gray, very stiff, wet						
89				CL							
90											

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/10/01

Treadwell&Rollo

Project No.: 3157.01

Figure: A-1c

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

Log of Boring B-1

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 (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.

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Project No.: 3157.01

Figure: A-1d

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)

Surface Elevation: 3.5 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
0–11				GP	SANDY GRAVEL with RUBBLE (GP) light brown, loose, dry, with concrete and brick debris	FILL						
11					CONCRETE SLAB, 5 to 6-inches thick							
11.5–20					CLAYEY SAND (SC) dark gray, very loose, wet, with shells							
15–16.5	S&H	▨	2	SC								
20–30					CLAY with SAND (CH) gray, very soft to soft, wet, with shells							
20–22.5	O	▨	50 psi								39.0	85
25–26.5	S&H	▨	0	CH								

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Project No.: 3157.01

Figure: A-2a

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

Log of Boring B-2

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value¹			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

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/8/01

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

Log of Boring B-2b

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

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)

Logged by: R. Nelson

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
					Surface Elevation: 3.5 feet[2]							
					SANDY GRAVEL with RUBBLE (GP) light brown, loose, dry, with concrete, brick and metal debris	↑						
1												
2												
3												
4												
5				GP		FILL						
6												
7												
8												
9												
10						↓						
11					CONCRETE SLAB 5- to 6-inches thick							
12					SANDY CLAY (CH) 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												

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Project No.: 3157.01

Figure: A-3a

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

Log of Boring B-2b

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
31					CLAYEY SAND (SC) 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) gray, soft to medium stiff, wet, with shells						
39											
40											
41	S&H	▨	4	CH							
42											
43											
44					SAND with CLAY (SP-SC) 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) gray, medium dense to dense, wet						
57											
58				SP-SC							
59											
60											

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

Log of Boring B-2b

PAGE 3 OF 3

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	SPT	◢	30		SAND with CLAY (SP-SC) (continued)						
62				SP-SC							
63											
64											
65					SAND with CLAY (SP-SC) 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) 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.

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Project No.: 3157.01

Figure: A-3c

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/13/01

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

Log of Boring B-3

PAGE 1 OF 6

Boring location: See Site Plan, Figure 2
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)
Logged by: R. Nelson

Surface Elevation: 3.5 feet^2

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
0					GRAVELLY SAND (SP) gray-brown, dense, dry, with concrete and brick debris FILL						
6	SPT		46	SP							
10					∇ 6-27-01						
11					∇ 6-26-01 CONCRETE SLAB 7-inches thick WOOD						
13	S&H		24/5"								
14					CONCRETE						
17					SAND (SP) dark gray, loose, wet						
19	S&H		5	SP							
22					CLAY (CH) gray, soft, wet, with shells and some fine sand						
26	O		50 psi	CH						28.9	95
30											

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: 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
30				CLAY (CH) (continued)						
31	O	50 psi							51.2	72
32			CH							
35				SANDY CLAY (CH) 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
38.5				CLAYEY SAND (SC) gray, medium dense, wet						
41	O	50 psi	SC		TxUU	1,500	595	39	32.0	91
44				SILTY SAND (SM) green-gray, very dense, wet						
45	S&H	30/4"	SM							
47.5				CLAYEY SAND (SC) green-gray, medium dense, wet						
51	SPT	23								
53			SC							
59.5			CH							
60										

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Project No.: 3157.01

Figure: A-4b

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	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	ST		25 psi		SANDY CLAY (CH) dark gray, medium stiff, wet, with shells					47.1	75
62				CH							
63											
64					SILTY SAND (SM) 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) orange-brown and olive, hard, wet						
81											
82											
83											
84											
85	S&H		38	CL						20.1	112
86											
87											
88											
89											
90											

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Project No.: 3157.01
Figure: A-4c

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

Log of Boring B-3

PAGE 4 OF 6

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
					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]** 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					stiff						
115	S&H	▨	11								
116											
117											
118											
119											
120											

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/10/01

Treadwell&Rollo

Project No.: 3157.01

Figure: A-4d

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

Log of Boring B-3

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES SPT N-Value'	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 (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											

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Figure: A-4e

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 (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.

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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)

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

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/8/01

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) 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) green-gray, very dense, wet						
45	S&H	▨	30/5"								
46											
47				SP							
48											
49					SILTY SAND (SM) gray, medium dense, wet LL=17, PI=4, See Figure C-1						
50		▨									
51	SPT	▨	12						21	27.7	
52											
53											
54				SM							
55											
56											
57											
58					CLAYEY SAND (SC) green-gray, medium dense, wet						
59				SC							
60											

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/8/01

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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			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
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

PROJECT:	**301 MISSION STREET** San Francisco, California	# Log of Boring B-5	PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

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)

DEPTH (feet)	SAMPLES Sampler Type	 Sample	 SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Surface Elevation: 3 feet[2]		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
0–11				GP	SANDY GRAVEL with RUBBLE (GP) brown, loose, dry, with concrete and brick debris	FILL						
11					CONCRETE SLAB ~11-inches thick							
12					CONCRETE							
15				SC-CH	CLAYEY SAND/SANDY CLAY (SC/CH) dark-gray, very loose/very soft to soft, wet, with shells							
20–21.5	S&H	▨	2									
30												

Treadwell&Rollo

Project No.: 3157.01

Figure: A-6a

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

Log of Boring B-5

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
31	S&H	▨	5		CLAYEY SAND/SANDY CLAY (SC/CH) (continued) loose/medium stiff					23.6	101
32											
33											
34				SC-CH							
35											
36											
37											
38					CLAYEY SAND (SC) dark gray, medium dense, wet, with some fine gravel						
39											
40											
41	S&H	▨	25	SC						22.0	101
42											
43					SAND (SP) 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) gray, medium stiff to stiff, wet						
59				CL							
60											

TEST GEOTECH LOG 315701_G.GPJ T&R.GDT 8/10/01

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	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 (CL) (continued)						
61	SPT		8								
62											
63											
64				CL							
65											
66											
67											
68											
69					CLAYEY SAND (SC) green-gray, medium dense, wet						
70											
71	S&H		19							32.2	87
72				SC							
73											
74											
75											
76					SILTY SAND (SM) yellow-brown, dense, wet						
77											
78											
79											
80											
81	S&H		37	SP							
82											
83											
84											
85											
86					SAND (SP) gray, very dense, wet						
87											
88				SP							
89											
90											

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Project No.: 3157.01

Figure: A-6c

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

Log of Boring B-5

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	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT	▨	50/6"		SAND (SP) (continued)						
91											
92				SP							
93											
94											
95					CLAYEY SAND (SC) 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.

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Project No.: 3157.01

Figure: A-6d

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

SAMPLE DESIGNATIONS/SYMBOLS

⊟	Sample taken with split-barrel sampler other than Standard Penetration Test sampler. Darkened area indicates soil recovered
⧄	Classification sample taken with Standard Penetration Test sampler
▯	Undisturbed sample taken with thin-walled tube
⊠	Disturbed sample
⧇	Sampling attempted with no recovery
▯	Core sample
▽	Unstabilized groundwater level
▼	Stabilized groundwater level

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)

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

CLASSIFICATION CHART

Date 07/23/01 | Project No. 3157.01 | Figure A-7

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
							Surface Conditions: 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 brown, moist, with brick fragments FILL
2						SP ▽	SAND grey, wet Groundwater encountered at 3 feet
3	TR-1-3.5	▨					
4	TR-1-4.0	▨					
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

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

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Project No.: 3157.01

Figure: B-1

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 Sample Number	Sample	Blow Count	Recovery (inches)	OVM (ppm)	LITHOLOGY	MATERIAL DESCRIPTION Surface Conditions: CONCRETE SLAB
							16-inch concrete slab
1							
2						SP	▽ SILTY SAND brown, loose, with brick fragments — FILL
3	TR-2-3.5 TR-2-4.0	▨ ▨				SP	SAND grey, loose, wet, fine-grained groundwater encountered at 2 feet.
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
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

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)			Surface Conditions: CONCRETE SLAB
							10-inch layer of concrete
1						SP	SILTY SAND brown, moist, with brick fragments FILL
2						SP	SAND grey, dense, dry, trace of clayey sand
3	TR-3-3.5	▨					
	TR-3-4.0	▨					
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

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

TEST ENVIRONMENTAL 315701_E.GPJ T&R.GDT 8/13/01

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 Sample Number	Sample	Blow Count	Recovery (inches)	OVM (ppm)	LITHOLOGY	MATERIAL DESCRIPTION
							Surface Conditions: CONCRETE SLAB
							8-inch concrete slab
1							SAND brown, then grey after 1-foot, loose, moist
2						SP	
3	TR-4-3.0 TR-4-3.5	▨ ▨					∇
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

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

Treadwell&Rollo

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 Sample Number	Sample	Blow Count	Recovery (inches)	OVM (ppm)	LITHOLOGY	MATERIAL DESCRIPTION Surface Conditions: CONCRETE SLAB
							6-inch concrete slab
1						SP	SILTY SAND light-brown, moist, with brick fragments **FILL**
2							
3	TR-5-3.0 TR-5-3.5	▨ ▨				SP	SAND ∇ grey, dense, wet, fine-grained, poorly-graded
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
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

TEST ENVIRONMENTAL 315701_E.GPJ T&R.GDT 8/13/01

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 Surface Conditions: CONCRETE SLAB	
							6-inch concrete slab	
1							SAND dark brown, loose, dry, fine-grained, poorly-graded with red brick	FILL
2								
3								
4						SP		
5							black coal waste	
6							porcelain	
7							wood pieces	
8	TR-6-8.0	▨						
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
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

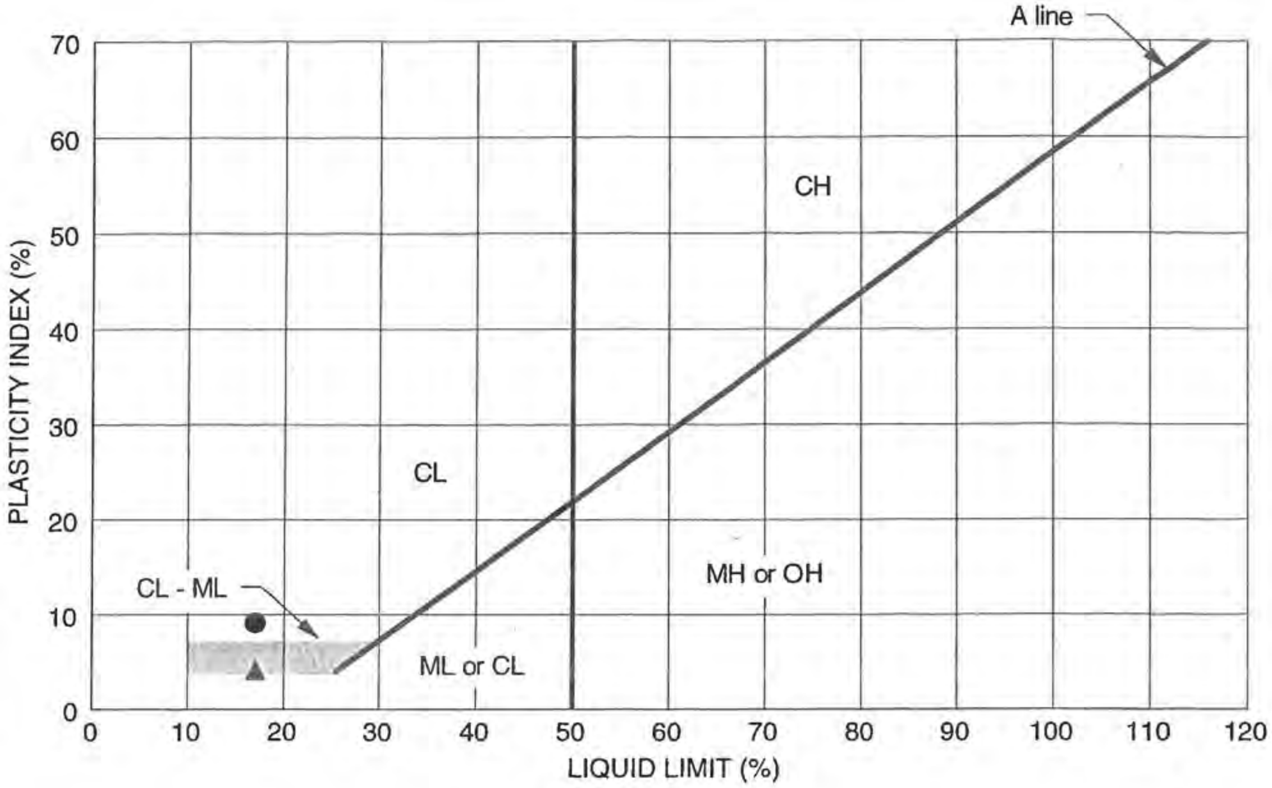
Project No.: 3157.01

Figure: B-6

TEST ENVIRONMENTAL 315701_E.GPJ T&R.GDT 8/13/01

APPENDIX C

Laboratory Test Data



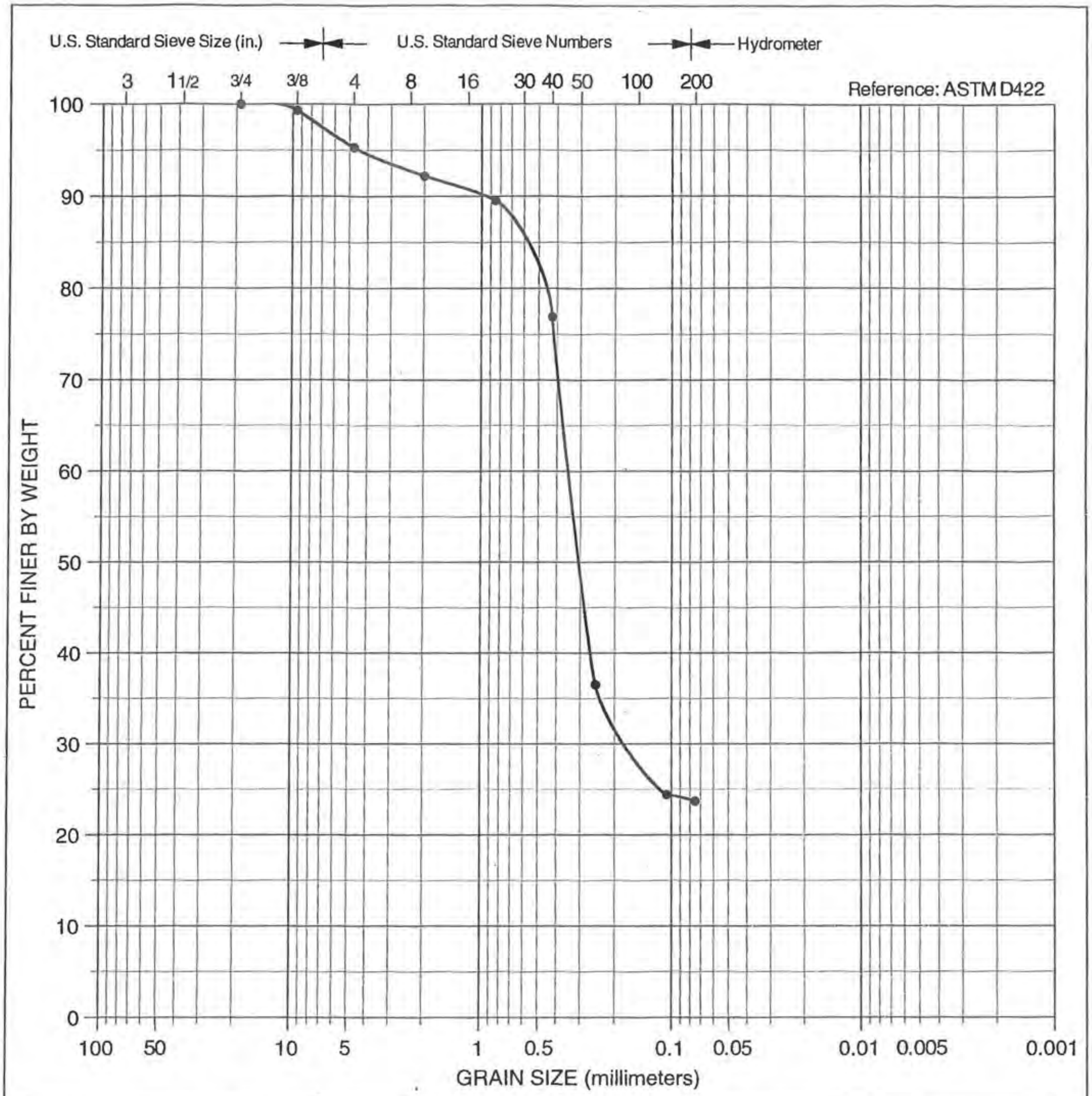
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 50 feet	CLAYEY SAND (SC), gray	24.1	17	9	19
▲	B-4 at 50 feet	SILTY SAND (SM), gray	27.7	17	4	21

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

PLASTICITY CHART

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



Cobbles	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay
	Gravel		Sand			

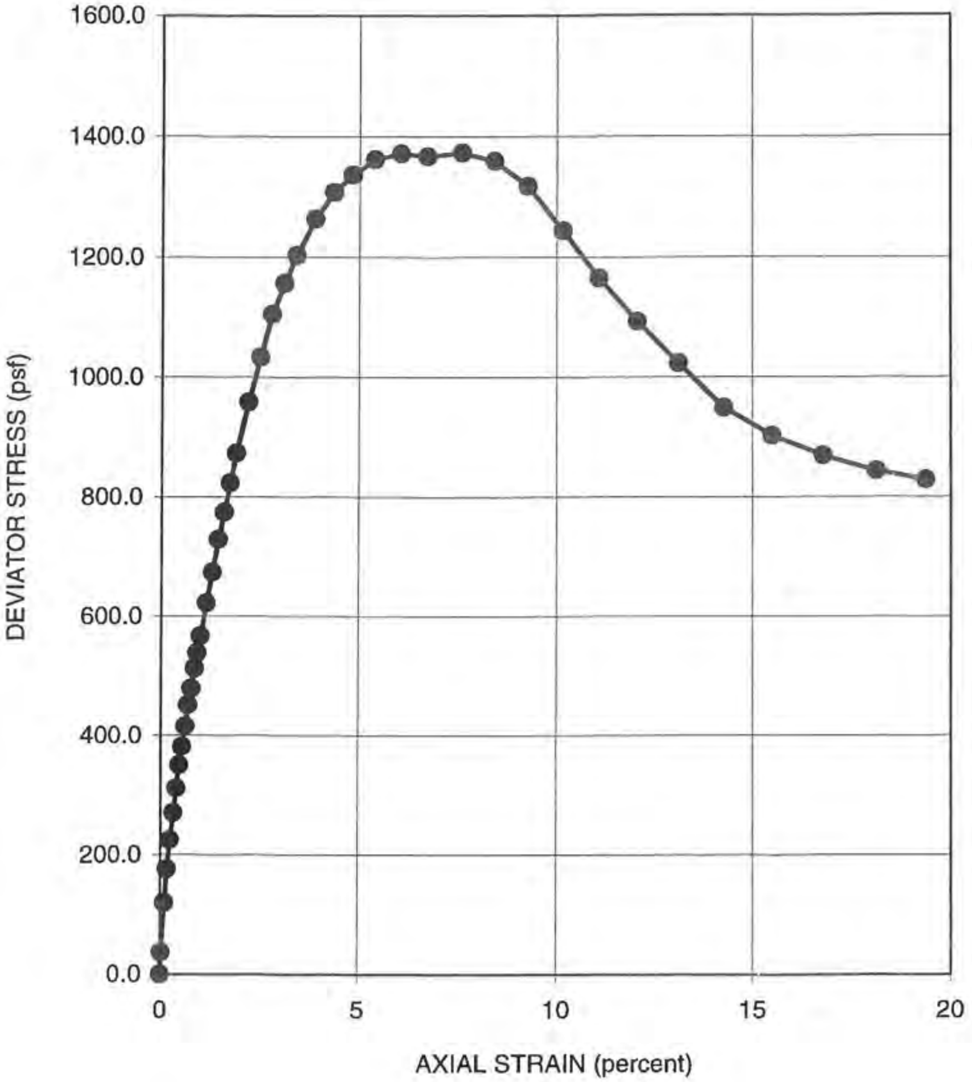
Symbol	Sample Source	Classification
●	B-26 at 36 feet	CLAYEY SAND (SC), gray

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

PARTICLE SIZE ANALYSIS

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



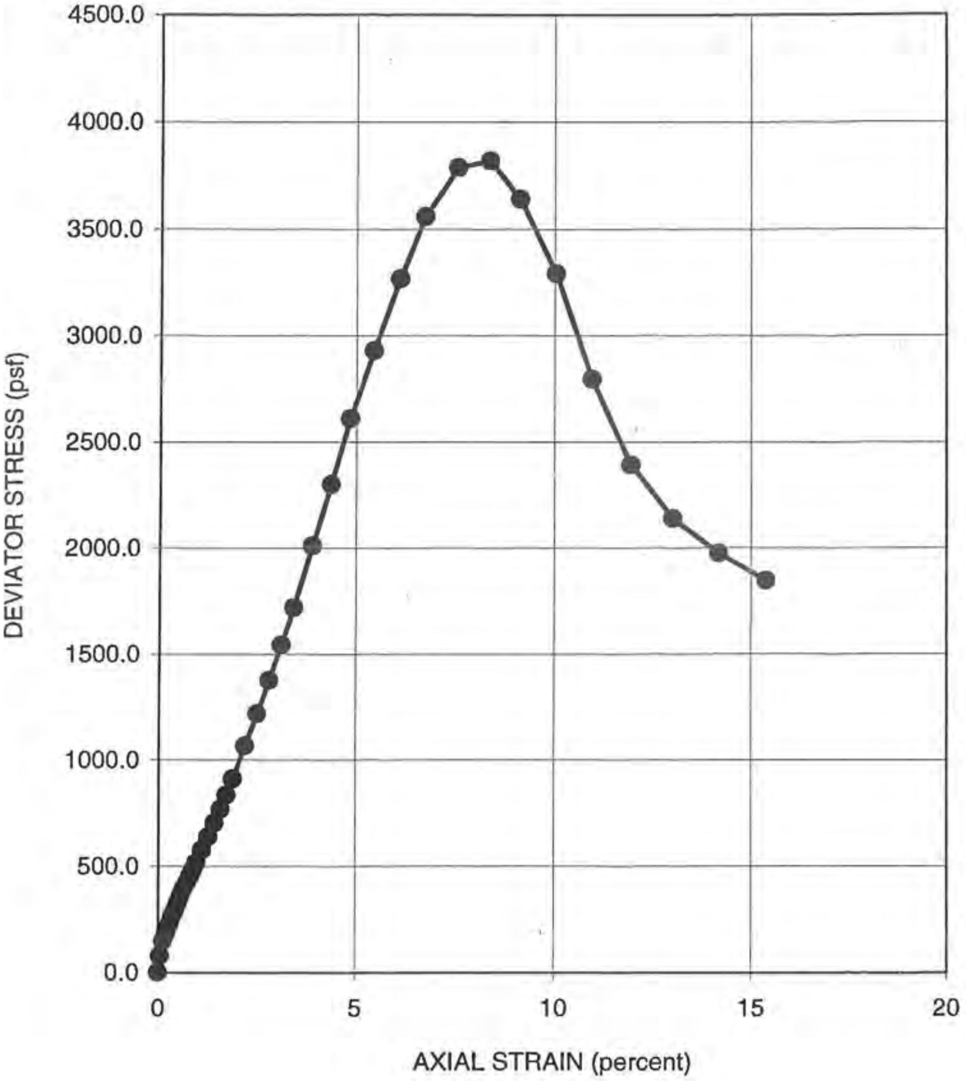
SAMPLER TYPE	Osterberg			SHEAR STRENGTH	685	psf
DIAMETER (in.) 2.9	HEIGHT (in.) 6.0			STRAIN AT FAILURE	7.6	%
MOISTURE CONTENT		37.1	%	CONFINING PRESSURE	1400	psf
DRY DENSITY		85	pcf	STRAIN RATE	0.67	% / min
DESCRIPTION	CLAY with SAND (CH), gray			SOURCE B-1 at 35 Feet		

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date	08/01/01	Project No.	3157.01	Figure	C-3



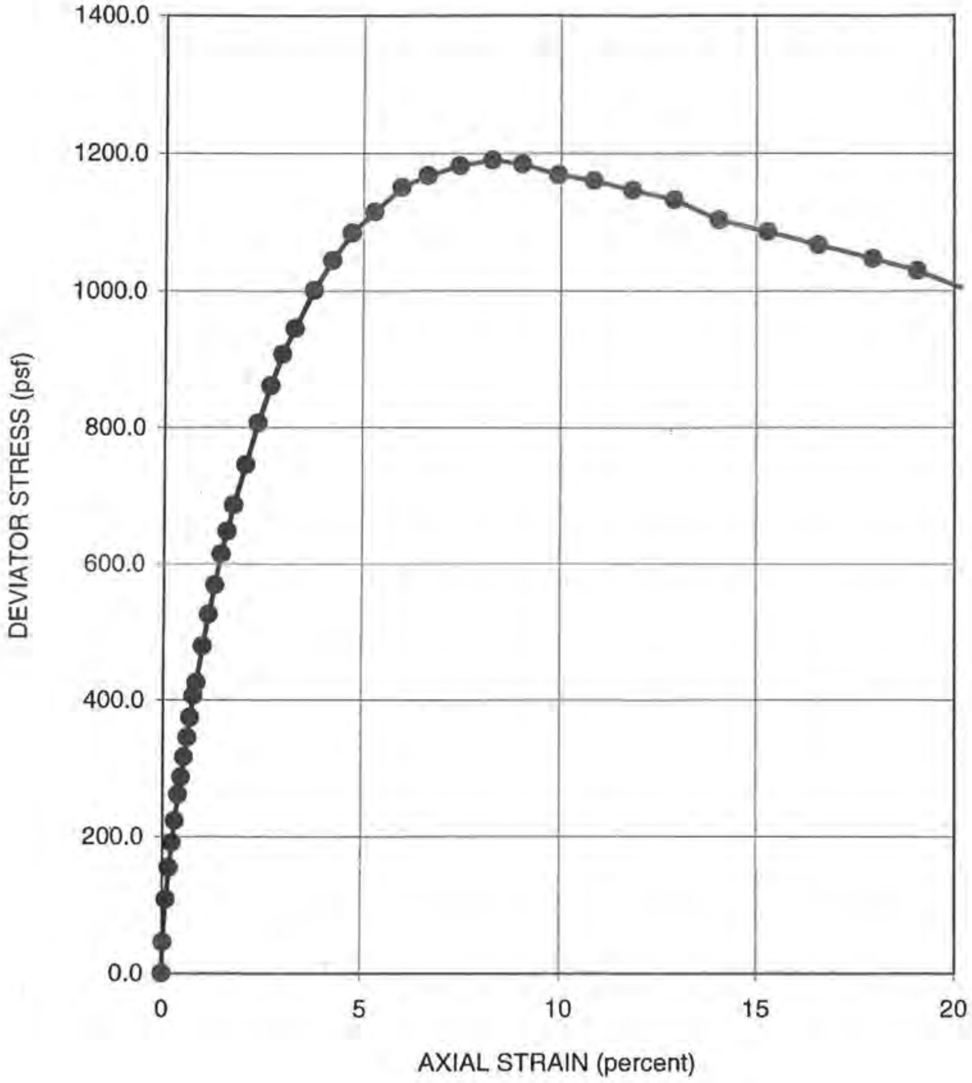
SAMPLER TYPE	Osterberg			SHEAR STRENGTH	1910	psf
DIAMETER (in.)	2.9	HEIGHT (in.) 6.0		STRAIN AT FAILURE	8.4	%
MOISTURE CONTENT		20.5	%	CONFINING PRESSURE	3000	psf
DRY DENSITY		110	pcf	STRAIN RATE	0.67	% / min
DESCRIPTION	CLAY (CL), gray			SOURCE B-1 at 90 Feet		

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 08/01/01 | Project No. 3157.01 | Figure C-4



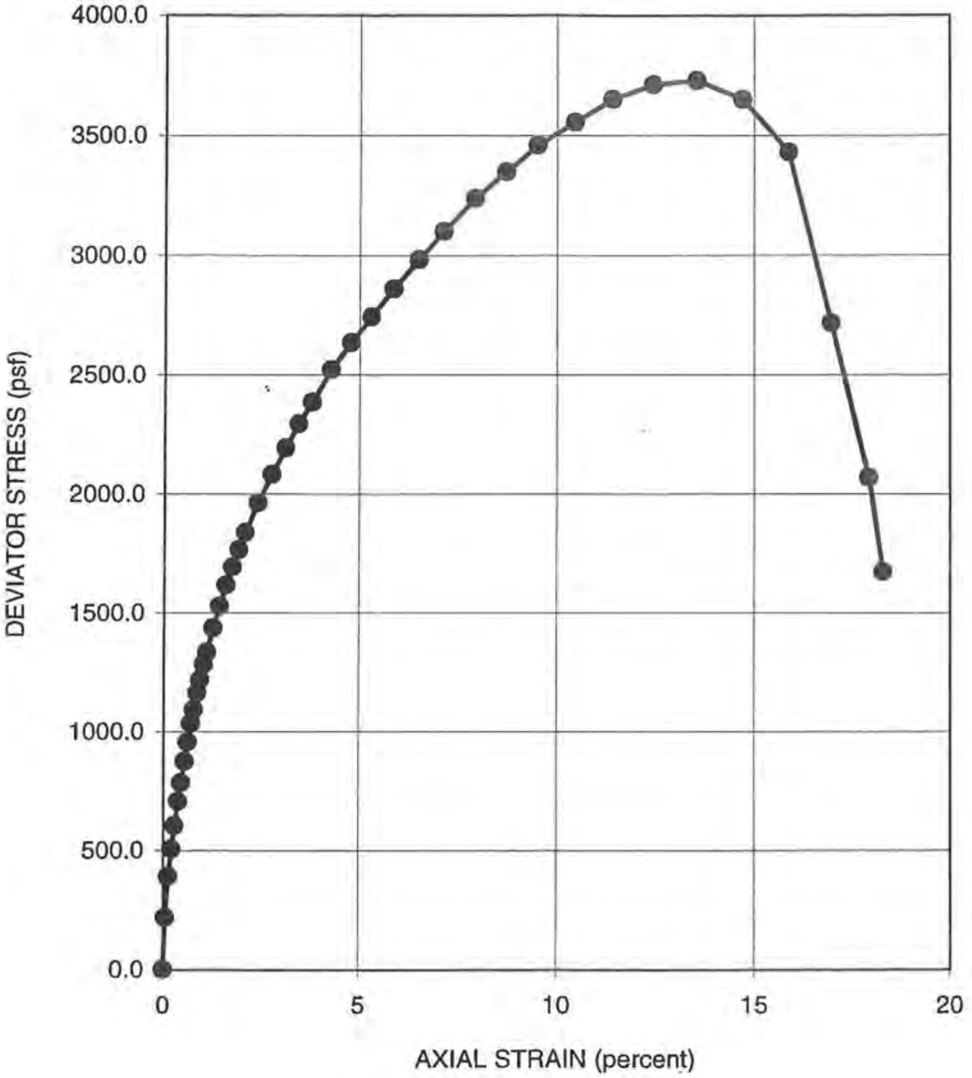
SAMPLER TYPE	Osterberg			SHEAR STRENGTH	595	psf
DIAMETER (in.)	2.9	HEIGHT (in.) 6.0		STRAIN AT FAILURE	8.2	%
MOISTURE CONTENT		32.0	%	CONFINING PRESSURE	1500	psf
DRY DENSITY		91	pcf	STRAIN RATE	0.67	% / min
DESCRIPTION	SANDY CLAY (CH), gray			SOURCE B-3 at 40 Feet		

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 08/01/01 | Project No. 3157.01 | Figure C-5



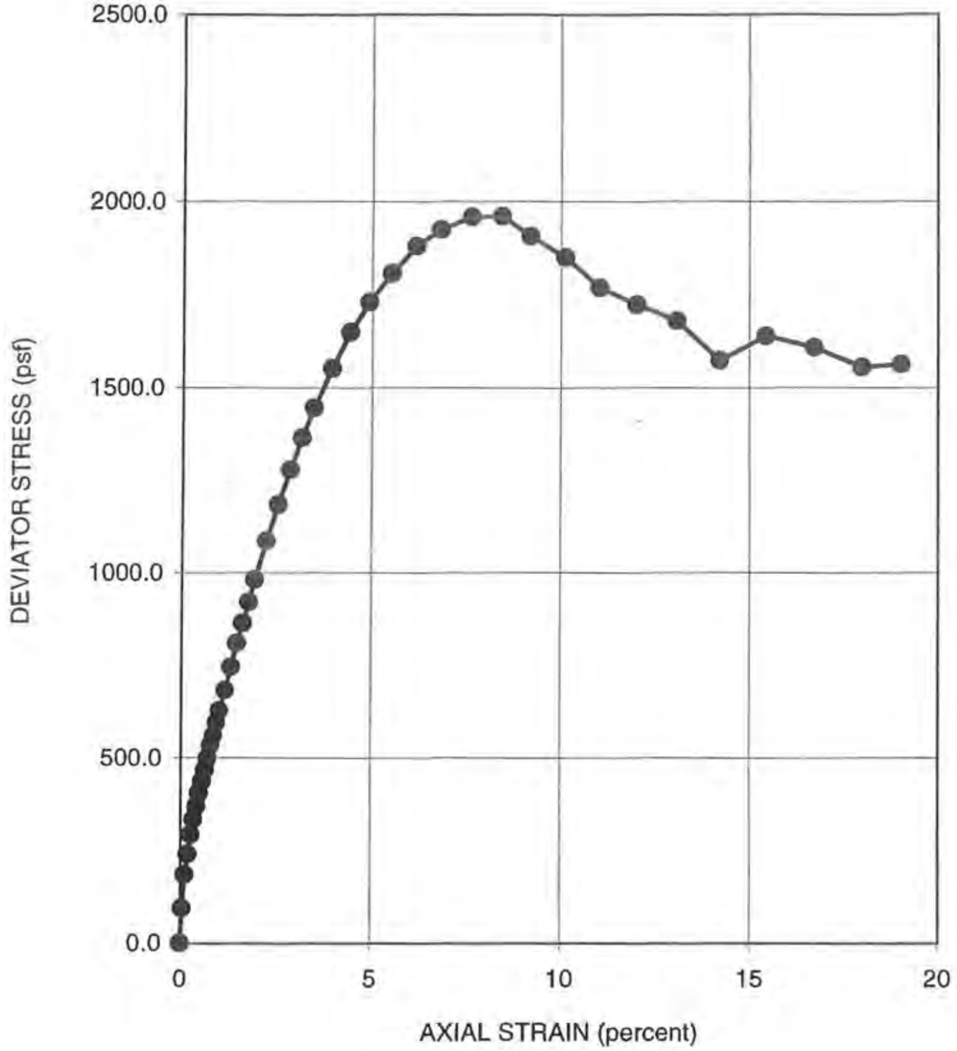
SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	1865	psf
DIAMETER (in.) 2.4	HEIGHT (in.) 5.8		STRAIN AT FAILURE	13.5	%
MOISTURE CONTENT	25.2	%	CONFINING PRESSURE	3500	psf
DRY DENSITY	100	pcf	STRAIN RATE	0.69	% / min
DESCRIPTION	CLAY (CL), gray		SOURCE B-3 at 105 Feet		

301 MISSION STREET
San Francisco, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 08/01/01 | 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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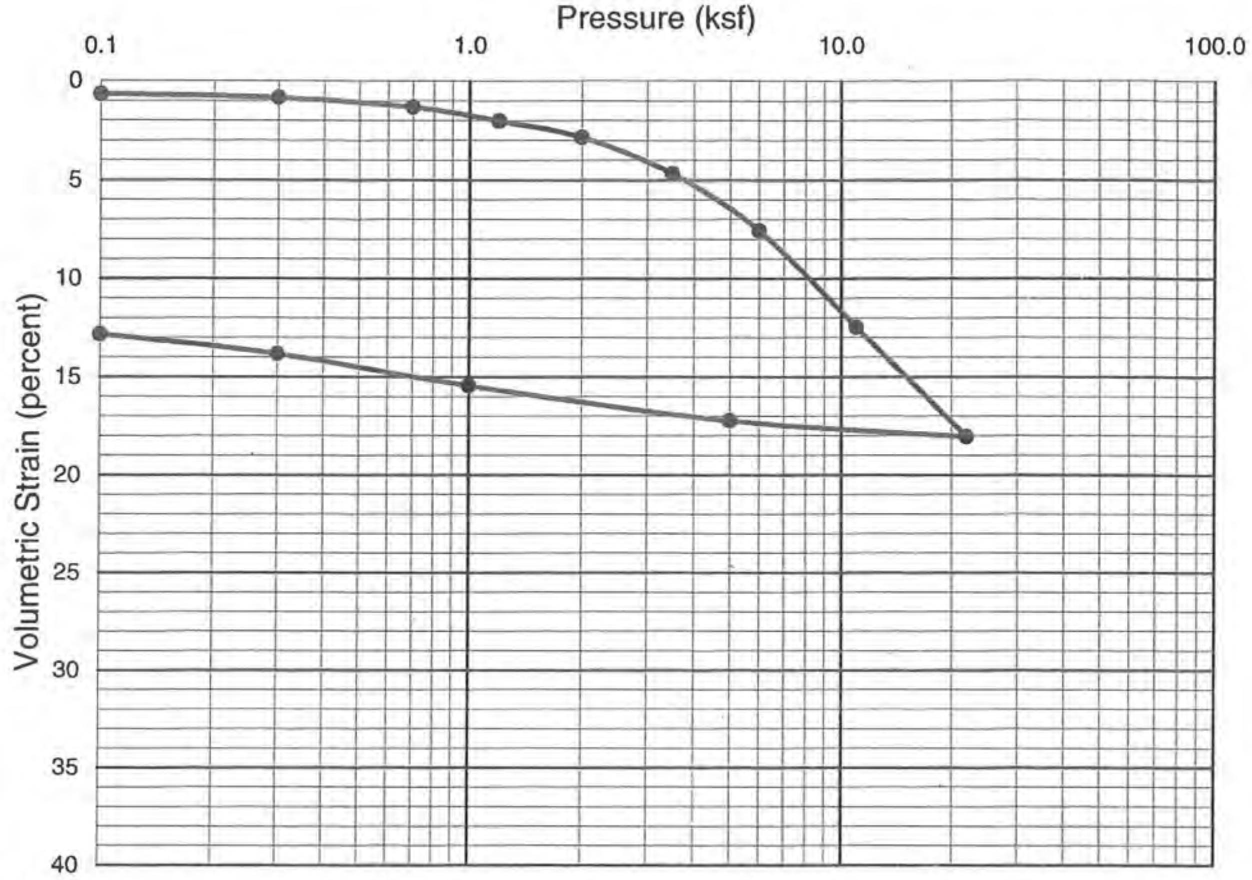
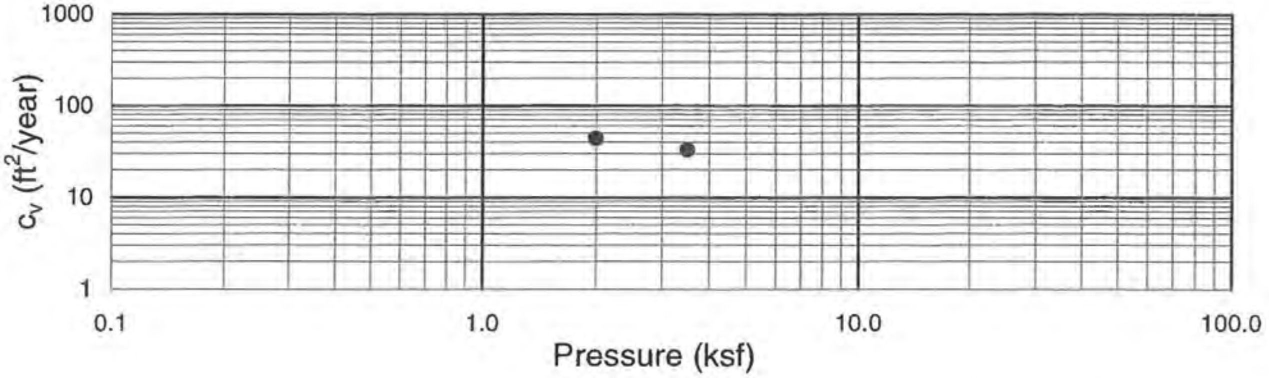
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 08/08/01 | 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	γ_d	87	pcf	γ_d	100	pcf
LL ---	PL ---	PI ---				G_s	2.70	(assumed)
Classification CLAY with SAND (CH), gray		Source	B-1 at 35 Feet					

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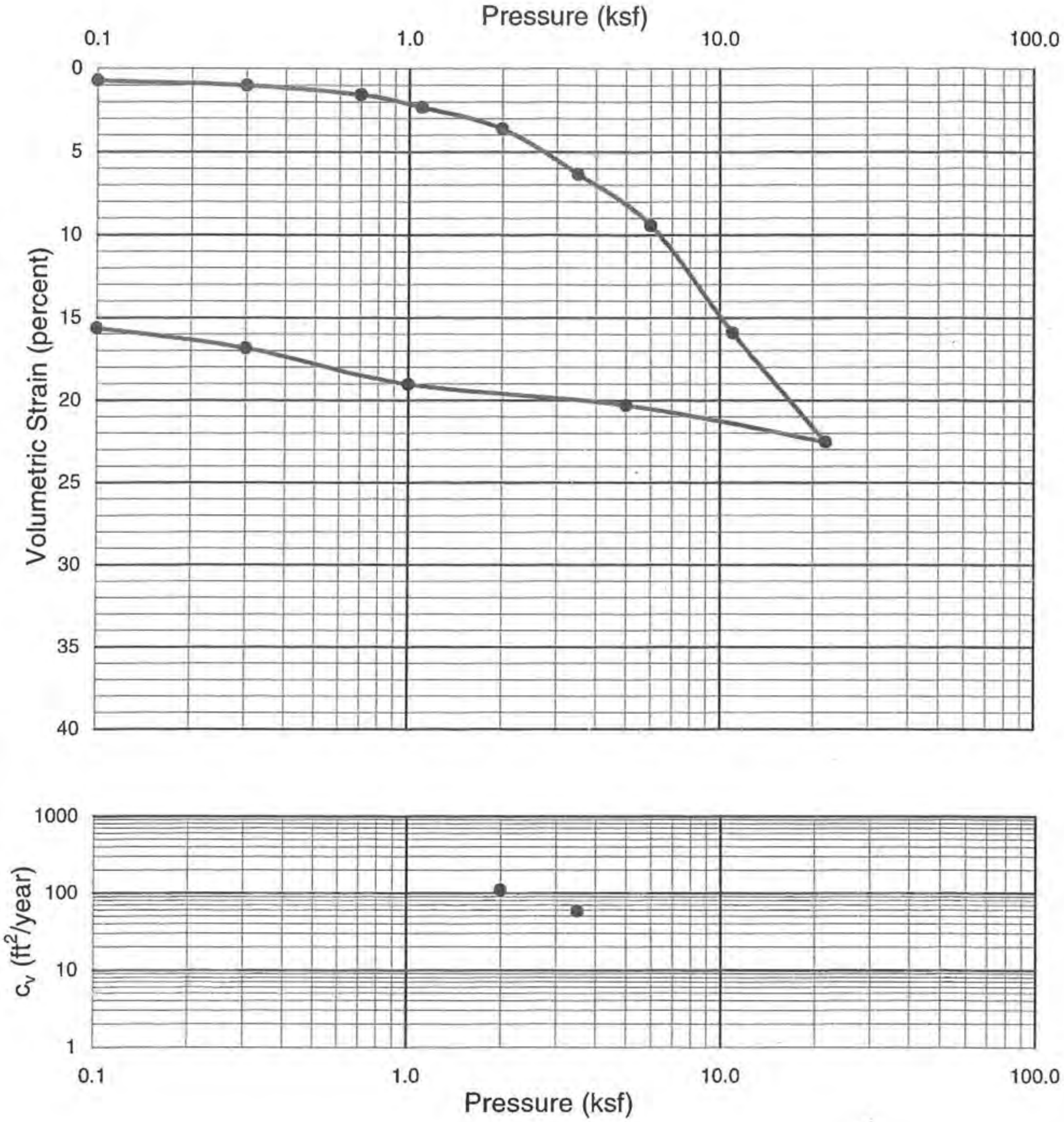
CONSOLIDATION TEST REPORT

Date 08/08/01 | 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, $C_{\varepsilon c}$	0.27	Dry Density	γ_d	75 pcf	γ_d	89 pcf
LL ---	PL ---	PI ---			G_s 2.70	(assumed)
Classification SANDY CLAY (CH), gray			Source	B-3 at 35 Feet		

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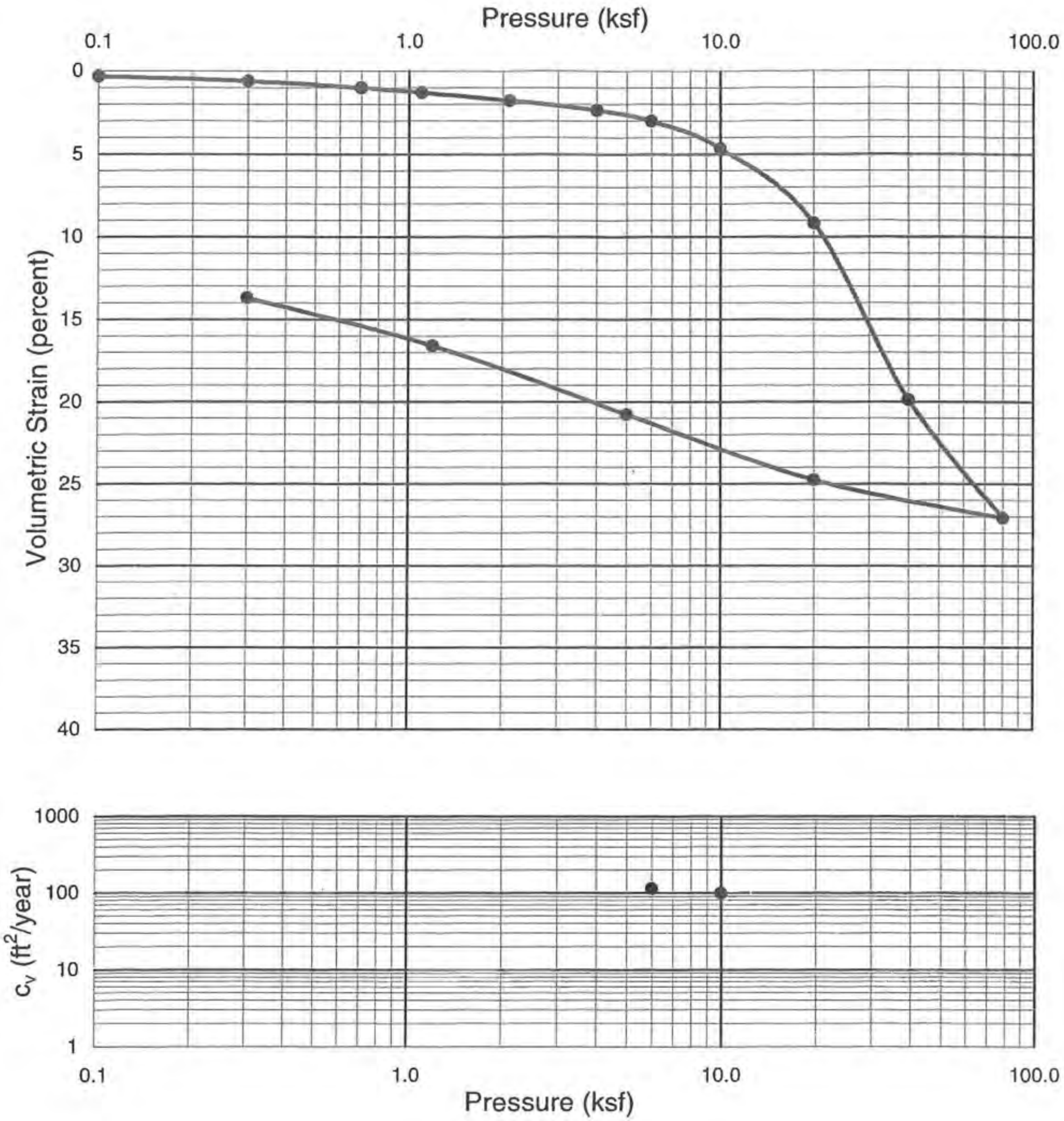
CONSOLIDATION TEST REPORT

Date 08/08/01 | 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	γ_d	76	pcf	γ_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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Date	08/08/01	Project No.	3157.01	Figure	C-10

APPENDIX D

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 DS/CD	600 1200	600 1080	9	116	U	13
10				DS/CD DS/CD	1500 3000	1340 2130	20	110	U	20
20				TX/UU	1700	830	43 40	79 78	P	200 PSI
30										
-37							22	104	TW	39
40										
-51										
50	-200 (7%)						19	110	U	100/6"
	-200 (15%)						20		SPT	78
60							20	113	U	80/6"
	-200 (9%)						18		SPT	97
70	-200 (16%)						22	111	U	90/6"
	-200 (13%)						21		SPT	119
80							18	116	U	68/6"
-86										
90	C	53	23	TX/UU	3500	2740	31 33 32	90 89 91	P	600 PSI
100				TX/UU	4000	2160	42 44	88 77	TW	32
110										
120	C	64	28	TX/UU	4600	2800	44 45 44	76 76 77	P	400 PSI
130										
140				TX/UU	5300	4330	42	78	P	400 PSI
150										

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.

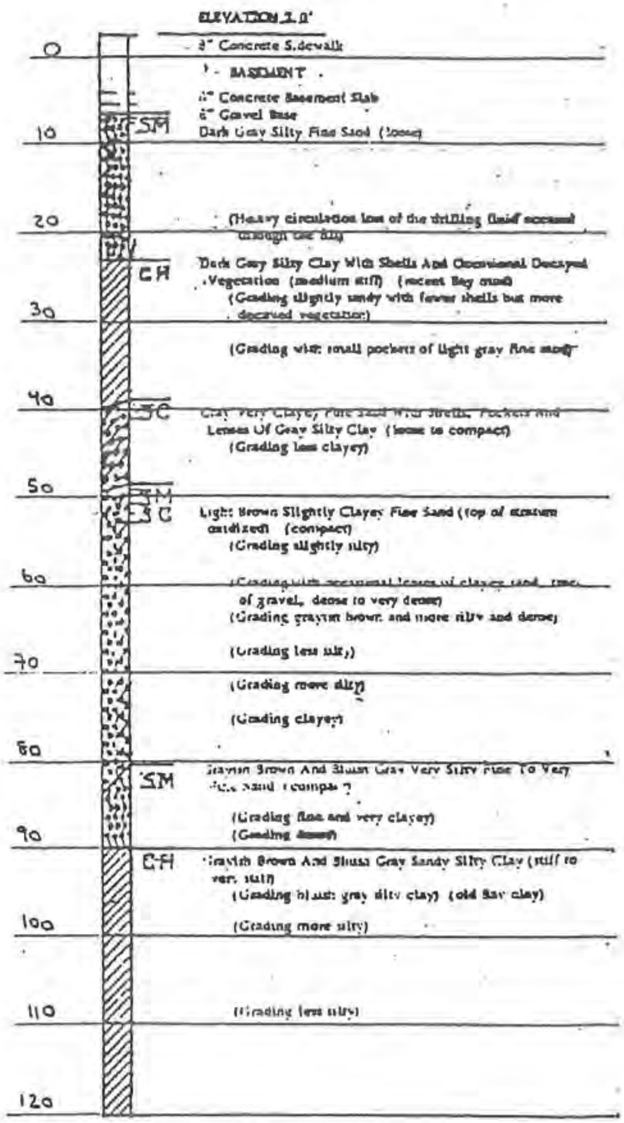
SAMPLES

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) (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) (GRADING CLAYEY)
CH	BLACK TO GRAY SILTY CLAY WITH TRACE OF ORGANICS (MEDIUM STIFF) (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 (GRADING SANDIER) (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) (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) (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) (GRADING WITH LAYER OF FINE SAND) (GRADING WITH ORGANIC MATERIAL, WOOD & SHELL FRAGMENTS) (GRADING SANDY) (GRADING GRAY) (GRADING SANDY) (GRADING HARD) (GRADING DARK GRAY)

LOG OF BORING

Dames & Moore

BORING. NO 3



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

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LOG OF BORING 3

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Project No. 2152.01

Figure 3

DISTRIBUTION

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QUALITY CONTROL REVIEWER:

Richard D. Rodgers

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