GEOTECHNICAL INVESTIGATION
1979 MISSION STREET
San Francisco, California

Maximus Real Estate Partners
San Francisco, California

30 January 2013
Project 731607001

Treadwell&Rollo
A LANGAN COMPANY

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Project 731607001

Mr. Seth Mallen
Principal
Maximus Real Estate Partners
345 Vidal Drive
San Francisco, California 94132

Subject: Geotechnical Investigation
1979 Mission Street
San Francisco, California

Dear Mr. Mallen:

Treadwell & Rollo, a Langan Company is pleased to present our geotechnical investigation report for the proposed commercial/residential development project at 1979 Mission Street in San Francisco. We are providing our services in accordance with our proposal dated 3 December 2012. A draft letter summarizing our preliminary conclusions and recommendations regarding building foundation and excavation support was published on 20 December 2012.

We understand you plan to construct a ten-story, commercial/residential structure above a basement on the site. The site has overall plan dimensions of 245 feet by 260 feet; it is bordered by Capp Street to the east, 16th Street to the south, Mission Street to the west, and several commercial buildings and Marshall Elementary School to the north. The Bay Area Rapid Transit (BART) 16th Street station is beneath Mission Street adjacent to the site; the BART entrance structure (plan dimensions 75 by 85 feet) occupies the southwest corner of the site. Currently, the site is occupied by two-story commercial buildings, Walgreens Store, and a parking lot; the commercial buildings and Walgreens will be demolished and removed.

BART has certain requirements regarding design and construction over or adjacent to its structures. BART's zone of influence (ZOI) is defined as the zone above an imaginary line drawn from the bottom of its substructure at a slope of 1.5 horizontal to 1 vertical towards the ground surface. BART requires that building loads within the BART ZOI do not impose surcharge pressure on the BART tunnel or the station walls.

To explore the subsurface conditions, we drilled test borings in the parking lot and reviewed the results of previous geotechnical investigations from sites in the vicinity. The borings indicate that the pavement is underlain by 2 to 4 feet of sandy fill. Below the fill to a depth of about 33 feet below the ground surface are inter-bedded layers of various alluvial deposits, consisting of loose to dense sand, stiff silt, and medium stiff to stiff clay. Below a depth of about 33 feet, very dense sand is present to the end of our borings, about 90 feet below the ground surface. Groundwater levels were encountered in our borings at a depth of about 8 feet below the ground surface.

The primary geotechnical issues associated with the proposed development are:

- selecting an appropriate foundation system that provides satisfactory building performance and meets BART ZOI requirements;
- selecting an appropriate shoring system(s) to retain the proposed excavation for the basement;
- mitigating shallow groundwater issues during construction and permanent condition.

On the basis of our investigation, we conclude a mat foundation should be used to support the structure. Within the BART ZOI, the mat should be supported on drilled piers that transfer the building load to the soil below the ZOI. To eliminate load transfer to the soil from frictional resistance within the BART ZOI, the drilled piers should be provided with double casings within the ZOI.

The excavation for the basement should be shored. Adjacent to BART substation, soldier-pile-with-lagging with internal bracing should be used. Elsewhere, a tied-back soldier-pile-and-lagging system can be used. The foundations of the two-story buildings north of the site should be underpinned using hand-excavated piers.

The report contains information regarding subsurface conditions, geologic hazards, and presents our recommendations regarding foundation design, shoring, permanent basement walls, and seismic design. Our conclusions and recommendations are based on limited subsurface exploration. Consequently, variations between expected and actual soil and groundwater conditions may be found at localized areas during construction. We should be retained to observe the installation of foundations and shoring, and excavation, grading and backfill operations, during which time we may make changes in our recommendations, if deemed necessary.

We appreciate the opportunity to work with you on this project. If you have any questions, please call.

Sincerely yours,
TREADWELL & ROLLO, A LANGAN COMPANY

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GEOTECHNICAL INVESTIGATION
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San Francisco, California

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed commercial/residential development at 1979 Mission Street in San Francisco, California. The site location is shown on Figure 1. We understand plans are to construct a 10-story, commercial/residential structure above a basement.

The site has overall plan dimensions of 245 feet by 260 feet. As shown on Figure 2, the site is bordered by Capp Street to the east, 16th Street to the south, Mission Street to the west, and several commercial buildings and Marshall Elementary School to the north. Currently, the site is occupied by two-story commercial buildings, Walgreens Store, and a parking lot; the commercial buildings and Walgreens will be demolished and removed. The parking lot grades vary from Elevation[1] 17 to 20 feet (City and County of San Francisco datum).

The Bay Area Rapid Transit (BART) 16th Street substation is beneath Mission Street west of the site; its entrance structure (plan dimensions 75 by 85 feet) occupies the southwest corner of the site. BART guidelines must be complied with where new construction is planned over or adjacent to its structure. BART requires that building that are within its zone of influence (ZOI), as defined by the ground within the zone above an imaginary line drawn from the bottom of its structure at a slope of 1.5 horizontal to 1 vertical to the surface, not impose a surcharge on the structure.

2.0 SCOPE OF SERVICES

A draft of our letter summarizing our preliminary conclusions and recommendations on the building foundation and excavation support was published on 20 December 2012. The objectives of our geotechnical investigation were to explore and evaluate subsurface soil and groundwater conditions beneath the site and to develop recommendations regarding the geotechnical aspects of the project. We evaluated subsurface conditions by 1) reviewing the results of geotechnical investigations in the site vicinity and 2) drilling test borings within the Walgreens' parking lot and performing laboratory tests on samples recovered from the boring.

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1 Elevations are obtained from preliminary site survey plan titled "ALTA/ACSM Land Title Survey of the Lands of the Jang Family, Limited Partnership", prepared by BKF Engineers, Surveyors, Planners, dated 21 December 2012.

On the basis of our field exploration and our engineering analyses, we developed our conclusions and recommendations for the geotechnical aspects of design and construction of the project, including:

- seismic hazards - ground rupture, liquefaction[2], lateral spreading[3], and differential compaction[4]
- appropriate foundation type(s) for the structure, taking into consideration the presence of BART substation west of the site
- design parameters for recommended foundation type(s), including bearing capacity and lateral resistance
- estimated settlement of foundation
- design earth and seismic pressures for basement walls
- preparation for foundation subgrade
- hydrostatic pressure on basement floors and walls, as appropriate
- appropriate temporary shoring system(s) to retain basement excavation
- design recommendations for recommended temporary shoring system
- dewatering during construction
- site grading, including criteria for fill quality and compaction
- 2010 San Francisco Building Code (SFBC) site class and seismic design parameters (maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at one-second period, S_{M1}, adjusted for site class effects)
- corrosion potential
- waterproofing and drainage for below-grade structure
- construction considerations

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2 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

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

4 Differential compaction (seismic densification) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing differential settlement.

3.0 FIELD INVESTIGATION

Prior to the field investigation, we obtained a drilling permit from the City and County of San Francisco Department of Public Health. Because the borings are on private property, we retained a private utility clearance subcontractor to check that the proposed boring locations are clear of buried utilities. As required by law, we also notified Underground Service Alert (USA) a week prior to drilling.

We drilled three borings, designated B-1 through B-3, at the approximate locations shown on Figure 2. The borings were drilled by Pitcher Drilling Company to depths ranging from 50 to 90 feet below the ground surface using rotary wash drilling equipment. Our field engineers logged the borings and obtained representative samples of the soil encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-3. The soil encountered was classified in accordance with the classification system described on Figure A-4.

Drilling was performed under the direction of our field engineer, who logged the soil encountered and obtained samples for classification and laboratory testing.

The boreholes were backfilled with cement grout and topped with cold-mixed asphalt. Drill cuttings were stored in 9, 55-gallon drums and temporarily stored on-site. We performed analytical tests for CAM 17 metals and total petroleum hydrocarbons (diesel, gasoline, and motor oil); the test results indicate the soil cuttings were not hazardous. The drums were subsequently removed from the site.

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

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

The samplers were driven with a 140-pound, automatic safety hammer falling about 30 inches. To account for sampler size and hammer energy, the blow counts required to drive the SPT and S&H sampler the final 12 inches of an 18-inch drive were converted to approximate SPT blow counts (N-values) using a conversion factor of 1.2 and 0.7, respectively, and are presented on the boring logs.

Upon completion of drilling, the holes were backfilled using cement grout, as required by the San Francisco Department of Public Health.

Prior to drilling the test borings, we reviewed the results of several geotechnical investigations from sites within three blocks of the subject site. Specifically, we reviewed the reports listed below:

- "Geotechnical Consultation, Mission Gardens – 1880 Mission Street, San Francisco, California," prepared by Treadwell & Rollo, dated 30 June 2011, Project No. 731566501, revised 28 July 2011.
- "Foundation and Shoring Recommendations, Mission Garden Mixed Use – 1880 Mission Street, San Francisco, California", prepared by Treadwell & Rollo, Project No. 4710.01, dated 16 November 2007.
- "Geotechnical Investigation, 655 to 695 South Van Ness Avenue, San Francisco, California," prepared by Treadwell & Rollo, Project No. 3695.01, dated 10 July 2003.
- "Geotechnical Investigation, Seismic Strengthening and Remodel, 154-174 Capp Street, San Francisco, California," prepared by Treadwell & Rollo, Project No. 3406.01, dated 22 May 2002.
- "Geotechnical Investigation, Friendship House Healing Center, 50-68 Julian Avenue, San Francisco, California," prepared by Treadwell & Rollo, Project No. 2962.01, dated 29 November 2000.
- "Geotechnical Investigation, Commercial/Residential Development, 16th and Valencia Streets, San Francisco, California," prepared by Harding Lawson Associates, Project No. 18019,001.04, dated 9 December 1986.
- "Soil Investigation, Mission Street Line, San Francisco Bay Area Rapid Transit," prepared by Harding Associates, dated 23 April 1964.

4.0 LABORATORY TESTING

We re-examined the soil samples in our office to confirm field classifications. We performed laboratory testing on selected samples to determine the physical and engineering properties of the subsurface soils.

Representative soil samples were delivered to a laboratory and were tested to measure moisture content, dry density, fines content, Atterberg Limits, and consolidation. The test results are presented in Appendix B and summarized in the boring logs.

Two soil samples were submitted to CERCO Analytical, Inc. for corrosivity analysis. The test results are attached in Appendix C and summarized in Section 7.4.

5.0 SITE CONDITIONS

5.1 Surface Conditions

The south and west sides of the site are occupied by commercial buildings and Walgreens Store, respectively; the commercial buildings are two stories high and may include a basement. The Walgreens parking lot elevations range from Elevation +17 to +20 feet (City and County of San Francisco datum).

5.2 Subsurface Conditions

Our test borings indicate that the parking lot pavement is underlain by 2 to 4 feet of sandy fill that contains debris, including bricks and wood. Below the fill to a depth of about 33 feet below the ground surface (bgs) are inter-bedded layers of alluvial deposits, consisting of loose to dense sand, stiff silt, and medium stiff to stiff clay. Below a depth of about 33 feet, very dense sand is present to the end of our borings, about 90 feet below the ground surface.

Groundwater levels were encountered in our borings at a depth of about eight feet bgs.

5.3 BART Zone of Influence

The 16th Street BART substation is beneath Mission Street west of the site. The exterior dimensions of the substation are approximately 54 feet (width) by 39 feet (height). The substation is embedded about 11 feet beneath Mission Street; therefore, the bottom slab rests on the ground approximately 50 feet beneath the street.

A typical cross section for the substation beneath Mission Street is shown on Figure 3. BART has certain requirements regarding design and construction over or adjacent to its subway structures. It established a zone of influence (ZOI), defined as the zone above a line of influence (LOI); the LOI is an imaginary line drawn from the critical point of the substructure at a slope of 1.5 horizontal to 1.0 vertical towards the ground surface. BART ZOI is depicted on the plan shown on Figure 4.

The new building will impose lateral surcharge pressures on the BART substructure from the foundation loads (including dead and seismic loads). BART will require that the surcharge pressures within the ZOI be evaluated to confirm that they will not affect the performance of its substructure. If the surcharge pressures exceed BART's design pressures, a deep building foundation deriving the capacity from the material below the ZOI will be required.

During construction, the basement excavation should be shored. BART requires that the shoring adjacent to BART structure be designed for at-rest earth pressures to reduce temporary wall movements. Because of its proximity to the BART substructure, in our opinion, internal bracing will be required to retain the temporary shoring.

6.0 SEISMIC CONSIDERATIONS

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

6.1 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 5. For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[5] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2007) and Cao et al. (2003)] are summarized in Table 1.

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

TABLE 1

Regional Faults and Seismicity

Fault Name	Distance (km)	Direction from Site	Mean Characteristic Maximum Moment Magnitude
San Andreas – 1906 Rupture	10	West	7.9
San Andreas – Peninsula	10	West	7.2
San Andreas – North Coast South	14	West	7.5
Northern San Gregorio	16	West	7.2
Total San Gregorio	16	West	7.4
North Hayward	19	Northeast	6.5
Total Hayward	19	Northeast	6.9
Total Hayward-Rodgers Creek	19	Northeast	7.3
South Hayward	19	East	6.7
Rodgers Creek	36	North	7.0
Mt Diablo – MTD	36	East	6.7
Total Calaveras	37	East	6.9
Monte Vista-Shannon	39	Southeast	6.8
Concord/Green Valley	40	East	6.7

Figure 5 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 6) 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 94 km from the site.

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

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (from 2007). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2007) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

6.2 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 site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading, post-liquefaction settlement[6], and cyclic differential compaction. We used the results of the borings to evaluate the potential of these phenomena occurring at the project site.

6 Post-liquefaction settlement occurs after sand particles of a liquefied sand layer rearrange themselves into a denser soil arrangement after dissipation of pore water pressures.

6.2.1 Fault Rupture

Historically, ground surface displacements closely follow the traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Faulting Zone Act, and no known active or potentially active faults exist on the site. In a seismically active area, a 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 low.

6.2.2 Strong Ground Shaking

During a major earthquake on one of the active faults in the region, the site will experience strong ground shaking similar to other areas of the seismically active San Francisco Bay Region. The intensity of the earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, magnitude and duration of the earthquake, and specific site geologic conditions. During its history, the site has been subjected to strong ground shaking from moderate to large earthquakes on the San Andreas, Hayward, and Calaveras, and San Gregorio faults, and future strong ground shaking should be expected.

6.2.3 Seismically-Induced Ground Deformations

Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, cyclic densification, and lateral spreading.

The site is in an area "where historic occurrence of liquefaction, or local geological, geotechnical or groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693 (c) would be required" (see Figure 7, Regional Seismic Hazard Zones Map). The test borings drilled on the site indicate the sand below the groundwater level is generally dense, except for sand layers at various depths within the upper 33 feet. Our analyses indicate that these sand layers can liquefy during a strong earthquake; however, because they are relatively thin and discontinuous, in our opinion liquefaction, if it occurs, would occur locally. Furthermore, available geologic reports and maps pertaining to ground failures in San Francisco caused by previous earthquakes (Lawson 1908; Youd and Hoose, 1978) indicate the site did not experience ground failures during the 1906 earthquake.

Lateral spreading refers to the finite, lateral displacement of sloping ground as a result of pore pressure build-up or liquefaction during an earthquake. Because the risk of extensive liquefaction at the site is low, we conclude that the risk of lateral spreading at the site during a strong earthquake is also low.

Differential compaction (cyclic densification) refers to compaction of non-saturated granular materials (sand and gravel above the groundwater table) caused by earthquake vibrations. The sandy fill and the native sand above the groundwater level is susceptible to differential compaction during a major earthquake on a nearby fault; depending on the thickness of loose fill, we estimate up to ½ inch of earthquake-induced settlement could occur at the ground surface outside the building. Within the building footprints, the loose sand will be removed during basement excavation; therefore, we do not anticipate differential compaction would occur beneath the building.

7.0 DISCUSSION AND CONCLUSIONS

On the basis of the results of our field exploration, laboratory testing, and engineering studies we conclude the proposed development is feasible from a geotechnical engineering standpoint. The primary geotechnical issues associated with the proposed development are:

- selecting an appropriate foundation system that provides satisfactory building performance and meets BART ZOI requirements,
- Selecting an appropriate shoring system(s) to retain the proposed excavation for the basement, and
- mitigating shallow groundwater issues during construction and for the permanent development.

Our discussion and conclusions regarding these issues and their impact on the design and construction of the proposed structure are discussed in the following sections.

7.1 Foundations and Settlement

Because a shallow groundwater level and isolated weak and potentially liquefiable soil are present at the site, we conclude the structure should be supported on a mat foundation. Within the BART ZOI, the mat should be supported on drilled piers that transfer the building load to the dense sand below the ZOI.

We estimate total settlement of a mat foundation designed using the allowable soil bearing pressure presented in this report should be about 1 inch. Differential settlement between adjacent columns should be less than ½ inch. Most of the settlement should occur during construction.

We estimate total settlement of a properly installed deep foundation (drilled piers) should be less than ½ inch. Differential settlement between adjacent columns supported on drilled piers should be on the order of 1/4 inch. Differential settlement between columns supported on a mat and deep foundation could be on the order of ¾ inch.

7.2 Groundwater

We encountered groundwater in our borings at about Elevation of +10 feet. Allowing for possible rise of groundwater during periods of heavy rainfall, we recommend a design groundwater level of Elevation +11 feet.

7.3 Construction Considerations

Construction considerations during shoring are discussed in this section.

7.3.1 Shoring

We expect site grading for the basement including the thickness of the mat will require an excavation of about 13 feet deep. There is insufficient space to slope the sides of the excavation; therefore, shoring will be required. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concerns are:

- protection of surrounding improvements, including BART Substation, streets, utilities, and adjacent structures
- proper construction of the shoring system to reduce potential for ground movements
- constructability
- cost

On the basis of our findings and our experience with sites having similar soil conditions, in our opinion a soldier-pile-and-lagging shoring system is appropriate for the site. This shoring system consists of steel

H-beams installed in predrilled holes; the holes are backfilled with concrete. Wood lagging is placed between the H-beams. We anticipate one row of tiebacks is needed to provide lateral support for the shoring. Care should be taken to locate utilities and other possible underground obstructions prior to installation. Encroachment permits are required from the adjacent neighbors and the City.

The two-story building along the northern property line should be underpinned. The underpinning can be performed using hand-excavated piers. The facing between the piers or piles should be lagged.

Along Mission Street, the shoring system should meet BART requirements: 1) the shoring system should be designed for earth at-rest pressures, 2) the shoring system should not impose surcharge pressure on the BART substructure, and 3) groundwater level should not be lowered more than 2 feet. We conclude the shoring should consist of soldier-pile-and-lagging with internal bracings (rakers). Piezometers should be installed under the Mission Street sidewalk to monitor the groundwater levels during excavation. If the levels drop more than 2 feet, the groundwater should be recharged using recharge wells.

7.3.2 Dewatering

Depending on the time of year the excavation is performed, the groundwater may be encountered as shallow as about 8 feet bgs. The excavation for the mat will extend approximately 13 feet bgs. Therefore, dewatering will be required. The contractor will need to obtain a dewatering permit from the City and County of San Francisco for discharging water into the local municipal waste water collection system. The dewatering permit requires chemical testing for characterizing the water to be discharged. The test results will determine if pretreatment of the groundwater is required prior to discharge of pumped groundwater from the site to the sanitary sewer system. Currently there is a fee for disposing of construction generated water in the City's waste water collection system.

7.3.3 Excavation Monitoring

During excavation, the shoring system may yield and deform, which could cause surrounding improvements to settle and move laterally. The magnitude of shoring movements and resulting ground deformations are difficult to estimate because they depend on many factors, such as soil conditions, type of shoring system and the contractor's skill in installing the shoring. Considering the excavation will be about 13 feet deep, we anticipate horizontal and vertical deformation for a properly installed shoring system should be less than 1 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. The monitoring should provide timely data which can be used to modify the shoring system if needed. In addition, inclinometers and piezometers should be installed to monitor movement of the shoring system during excavation and construction.

7.4 Corrosivity Evaluation

CERCO Analytical of Pleasanton, California performed corrosivity test on representative samples retrieved from a depth of about 6 feet bgs in Boring TR-1 and TR-2. Corrosion potential was determined based on the nominal resistivity measurement (100 percent saturation), chloride ion concentration, sulfate ion concentration, pH, and redox potential.

The results of corrosivity testing as well as a summary describing the corrosion characteristics of the near surface soil and protection recommendations are included in Appendix C.

8.0 RECOMMENDATIONS

Recommendations regarding temporary shoring, foundation design, basement slabs and walls, and seismic design, and site grading are presented in the following sections.

8.1 Temporary Shoring

We recommend retaining the sides of the basement excavation using the systems described below:

- General area: soldier-pile-and-lagging with tiebacks
- adjacent to commercial building north of the site: hand-excavated underpinning piers
- Within the BART ZOI: soldier-pile-and-lagging with internal bracings (rakers).

Design parameters for temporary soldier pile and lagging shoring system are shown on Figure 8. The design pressures are based on the assumption the groundwater within the site is drawn down to at least 3 feet below the excavation level.

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

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

8.1.1 Tieback Design Criteria and Installation Procedure

Tiebacks should be designed to derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point 0.2H feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the excavation depth in feet.

Allowable capacities of the tiebacks will depend upon the drilling method, tieback-hole diameter, grout pressure, and workmanship. Because specialty contractors who install the tiebacks use different types of installation procedures, the skin friction of the tieback will vary. For estimating purposes, we recommend using the allowable skin friction values of 1,200 psf. This value is for pressure-grouted tiebacks and includes a factor of safety of 1.5. Higher allowable skin friction values may be used if confirmed with pre-production performance tests. All tiebacks should have a minimum bonded length of 15 feet and minimum un-bonded length of 10 and 15 feet for bars and strands, respectively.

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

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

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. Recommendations for tieback testing are presented in Section 8.1.2. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer at the expense of the contractor.

8.1.2 Tieback Anchor Testing

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

8.1.2.1 Performance Tests

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

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

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

8.1.2.2 Proof Tests

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

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

8.1.2.3 Acceptance Criteria

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between one and ten minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the un-bonded length.

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

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

8.1.3 Internal Bracing

Internal bracing such as horizontal pipe struts or inclined rakers will be required within the BART ZOI, as discussed in Section 7.3.1. Footings can be used to support the rakers. The footings should be designed for an allowable bearing pressure of 3,000 psf.

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

8.1.4 Penetration Depth of Soldier Piles

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient capacity to support the axial load, if any. The vertical load component of the tiebacks can be assumed to be resisted by the friction along the back of the soldier piles. To compute the axial capacity of the soldier piles below the bottom of the excavation, we recommend using an allowable skin friction of 800 psf; the end bearing resistance of the soldier piles should be neglected.

8.2 Construction Monitoring

During excavation, the shoring system may yield and deform laterally, which could cause surrounding improvements to settle. A monitoring program should be established to evaluate the effects of the construction on the adjacent buildings, street, and other improvements. To monitor movements, we recommend installing survey points on the adjacent buildings and streets that are within 30 feet of the site. To monitor the groundwater level within BART ZOI, we recommend installing piezometers on Mission Street sidewalk adjacent to the site.

The instrumentation should be read regularly and the results should be reviewed in a timely manner. Initially, the instrumentation should be read weekly. The frequency of readings may, in the later stage of construction, be modified as appropriate. In addition, the conditions of existing buildings within 50 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction.

8.3 Dewatering

Dewatering will be required; the collected water should be properly disposed of. If the pumped groundwater is disposed of in the City storm drain, it is likely the discharge will have to be tested for hazardous substances and metered; a fee would be required by the City. The volume of water discharged should be monitored and a record of the amount be submitted to the owner.

8.4 Site Excavation, Subgrade Preparation and Backfill

Remnants of existing building foundations, building debris, and other obstructions may be encountered during excavation. If tiebacks were used to retain the excavation for the BART Substructure, they may still be present beneath the ground surface.

The soil exposed at the subgrade should be graded to produce a level, non-yielding surface. To provide a smooth surface a layer of crushed rock or lean concrete may be used. We should check the mat subgrade prior to placing crushed rock/lean concrete for proper bearing. Loose or soft material encountered at the subgrade should be removed and replaced with lean concrete. Where temporary slopes are to be cut, we recommend that they be no steeper than 1.5:1 (horizontal to vertical).

Materials to be used as fill and backfill under any flatwork can consist of onsite soil that is free of organic matter, and contain no rocks or lumps larger than four inches in greatest dimension. Imported fill, if needed, should also meet these criteria, be similar in type to the existing sandy soil and have a low expansion potential as defined by a liquid limit (LL) of less than 25 and a plasticity index (PI) of 8 or less. Samples of imported material should be submitted to us for approval and testing at least 72 hours before delivery to the site. We judge that majority of the on-site material is suitable to be used as backfill material.

Fill and backfill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 95 percent relative compaction[7].

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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 laboratory compaction procedure.

8.5 Mat Foundation

The proposed structure should be supported on a mat foundation bearing on dense native sand. Within the BART's ZOI, drilled piers should be used to support the mat. Our design recommendations for drilled piers are presented in Section 8.6.

Mat foundation supported on native dense sand should be designed using an allowable bearing pressure of 4,000 psf for dead plus live loads. For total loads, including wind or seismic forces, the allowable bearing pressures can be increased by one third. For design using subgrade modulus method, we recommend using a subgrade modulus of 50 kips per cubic foot (kcf).

Lateral loads on the mat can be resisted by a combination of passive resistance acting against the vertical faces of the mat and friction along the bases of the footings. Passive resistance may be calculated using lateral pressures corresponding to an equivalent fluid unit weight of 300 and 150 pounds per cubic foot (pcf) for above and below design groundwater level, respectively. We recommend design groundwater level at Elevation +11 feet. Frictional resistance should be computed using a base friction coefficient of 0.35. If the mat is waterproofed, a base friction coefficient of 0.2 should be used. The passive resistance and base friction values include a factor of safety of about 1.5 and may be used in combination without reduction.

Weak soil or non-engineered fill encountered in the bottom of mat excavation should be excavated and replaced with crushed rock or lean concrete. We should check mat subgrade prior to placement of reinforcing steel. The subgrade should be free of standing water, debris, and disturbed materials prior to placing concrete.

8.6 Drilled Piers

Foundation within the BART ZOI should consist of drilled piers deriving the capacity from the soil below the ZOI. To avoid surcharging BART Structure, BART will require that the pier section above the influence line be installed with double casing.

Drilled piers should be designed to derive their axial capacity from the skin friction in the soil layers below the ZOI. In local practice, the contribution of end bearing in supporting the load is ignored for drilled piers installed below the groundwater level. Piers should have a diameter of at least 24 inches; where they are installed in a group, they should be spaced at least three diameters on center. To compute the axial capacity of drilled piers, we recommend allowable unit skin friction presented in Table 3 below:

Table 3
Allowable Unit Skin Friction for Drilled Piers

Depth Below Mat, feet	Allowable Unit Skin Friction, psf
0 - 10	800
10 – 20	1,000
20- 30	1,200
30 – 40	1,500
Below 40	1,800

The allowable unit skin friction values presented in Table 3 are for dead plus live load and include a factor of safety of about 2.0. For temporary, compressive, total loads, including wind and/or seismic load, the skin friction values can be increased by one third. For temporary uplift loads, we recommend using allowable skin friction values presented in Table 3.

For design using the subgrade modulus method, we recommend a preliminary spring constant value of 10,000 kips per foot. We should check this value after preliminary structural design is completed.

Potentially caving sand will be encountered during drilling. Therefore, casing and/or drilling fluid will be required. Concrete placement should start upon completion of the drilling and clean out. Concrete should be placed from the bottom up in a single operation using a tremie and/or a pumper pipe. The tremie pipe should be maintained at least 5 feet below the upper surface of the concrete during casting of the piers. The concrete should have a slump between 7 and 9 inches. As the concrete is placed, casing used to stabilize the hole can be withdrawn. The bottom of the casing should be maintained at least 3 feet below the surface of the concrete.

8.7 Seismic Design

For seismic design in accordance with the provisions of 2010 San Francisco Building Code we recommend design parameters listed below:

- Maximum Considered Earthquake (MCE) S_S and S_1 of 1.50g and 0.70g, respectively.
- Site Class D
- Site Coefficients F_a and F_v of 1.0 and 1.5, respectively
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.50g and 1.05g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.00g and 0.70g, respectively.

8.8 Basement Walls and Slabs

Basement walls should be designed to resist lateral at-rest pressures imposed by the adjacent soil and any surcharge loads. Because the site is in a seismically active area, basement walls should be checked for the seismic condition. The pressure increment due to seismic loading should be added to active earth pressures. We used the procedures outlined in Sitar et al. (2012) to compute the seismic pressure increment. Table 4 presents the at-rest, active, and seismic increment in equivalent fluid unit weights. Seismic pressure increments are presented for peak ground accelerations of 0.4g and 0.6g for the design earthquake (DE) and maximum credible earthquake (MCE) levels of shaking, respectively.

TABLE 4
Basement Wall Design Earth Pressures

Retained Soil	At-rest Pressure, pcf	Active Pressure, pcf	Seismic Pressure Increment (added to the active pressure), pcf	
			DE	MCE
Above Groundwater Level	55	35	24	42
Below Groundwater Level	90	80	24	42

If surcharge loads occur above an imaginary 45-degree line (from the horizontal) projected up from the bottom of a basement wall, a surcharge pressure should be included in the wall design. We should be consulted to estimate the added pressure. Where vehicular traffic will pass within 10 feet of a basement wall, traffic surcharge, modeled by a uniform pressure of 100 psf in the upper 10 feet, should be added to the design pressures.

To protect against moisture migration, basement walls and slabs should be waterproofed and water stops placed at all construction joints. A waterproofing consultant should be retained to design and observe the installation of the waterproofing system.

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

8.9 Utilities

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

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

8.10 Concrete Pavement, Exterior Slabs and Pavers

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

9.0 FUTURE GEOTECHNICAL SERVICES

Prior to construction, we should review the shoring and project plans and their specifications to check their conformance to the intent of our recommendations. During construction, we should observe shoring installation, excavation, foundation subgrade preparation, drilled pier installation, and compaction of backfill. These observations will allow us to compare the actual with the anticipated subsurface conditions and check that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

10.0 LIMITATIONS

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

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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.

Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J., 2003, "The Revised 2002 California Probabilistic Seismic Hazard Maps."

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Sitar, N., Mikola, R.G., and Candia, G., 2012, "Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls," Geotechnical Engineering State of the Art and Practice, Geotechnical Special Publication No. 226, ASCE.

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Working Group on California Earthquake Probabilities (WGCEP), 2003, "Earthquake probabilities in the San Francisco Bay region: 2002 to 2031." Open File Report 03-214.

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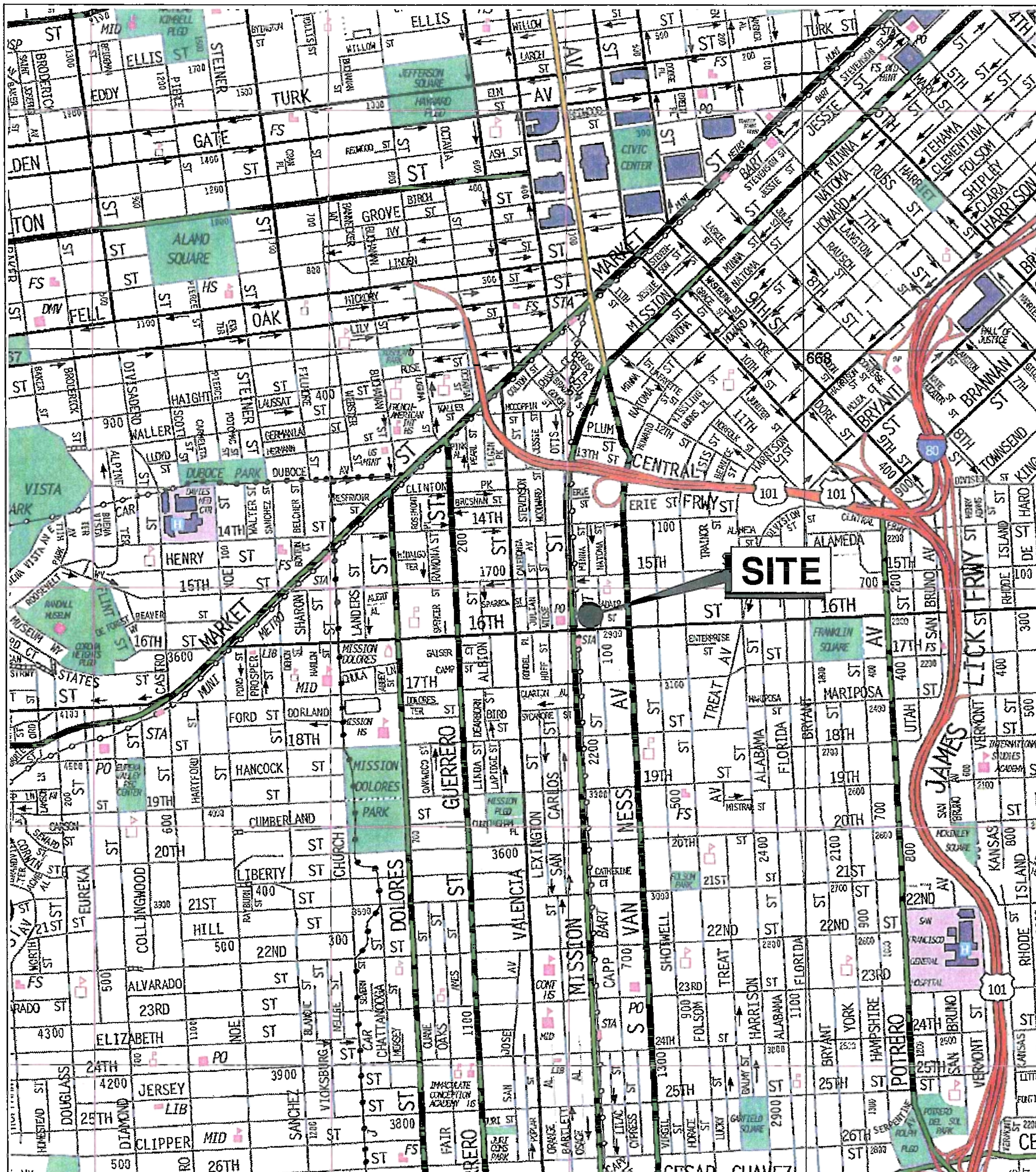
FIGURES



Base map: The Thomas Guide
San Francisco County
1999

0 1/4 1/2 Mile

Approximate scale

1979 MISSION STREET
San Francisco, California

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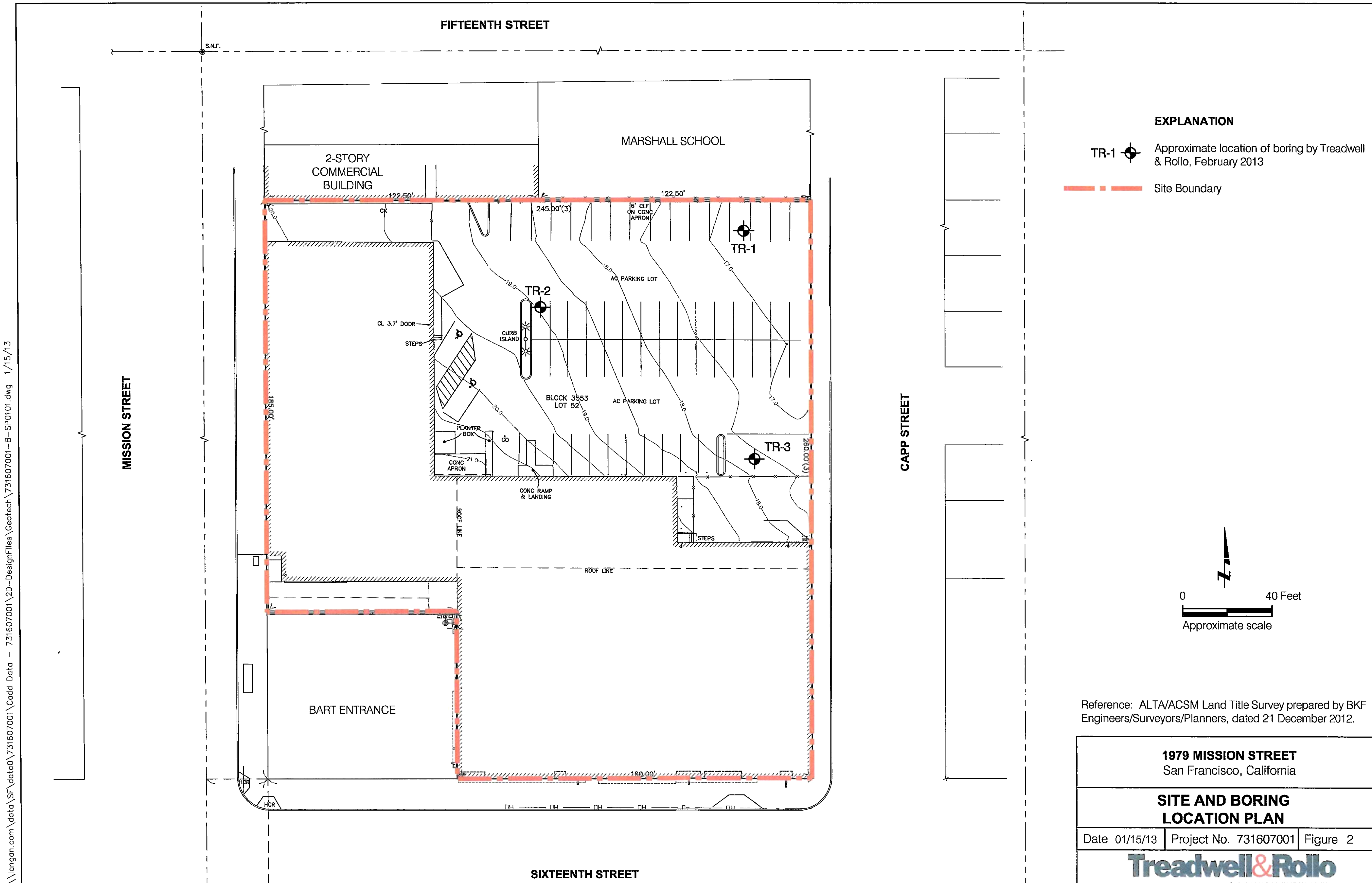
SITE LOCATION MAP

Date 12/20/12	Project No. 731607001	Figure 1



Reference: ALTA/ACSM Land Title Survey prepared by BKF Engineers/Surveyors/Planners, dated 21 December 2012.

1979 MISSION STREET San Francisco, California		
SITE AND BORING LOCATION PLAN		
Date 01/15/13	Project No. 731607001	Figure 2

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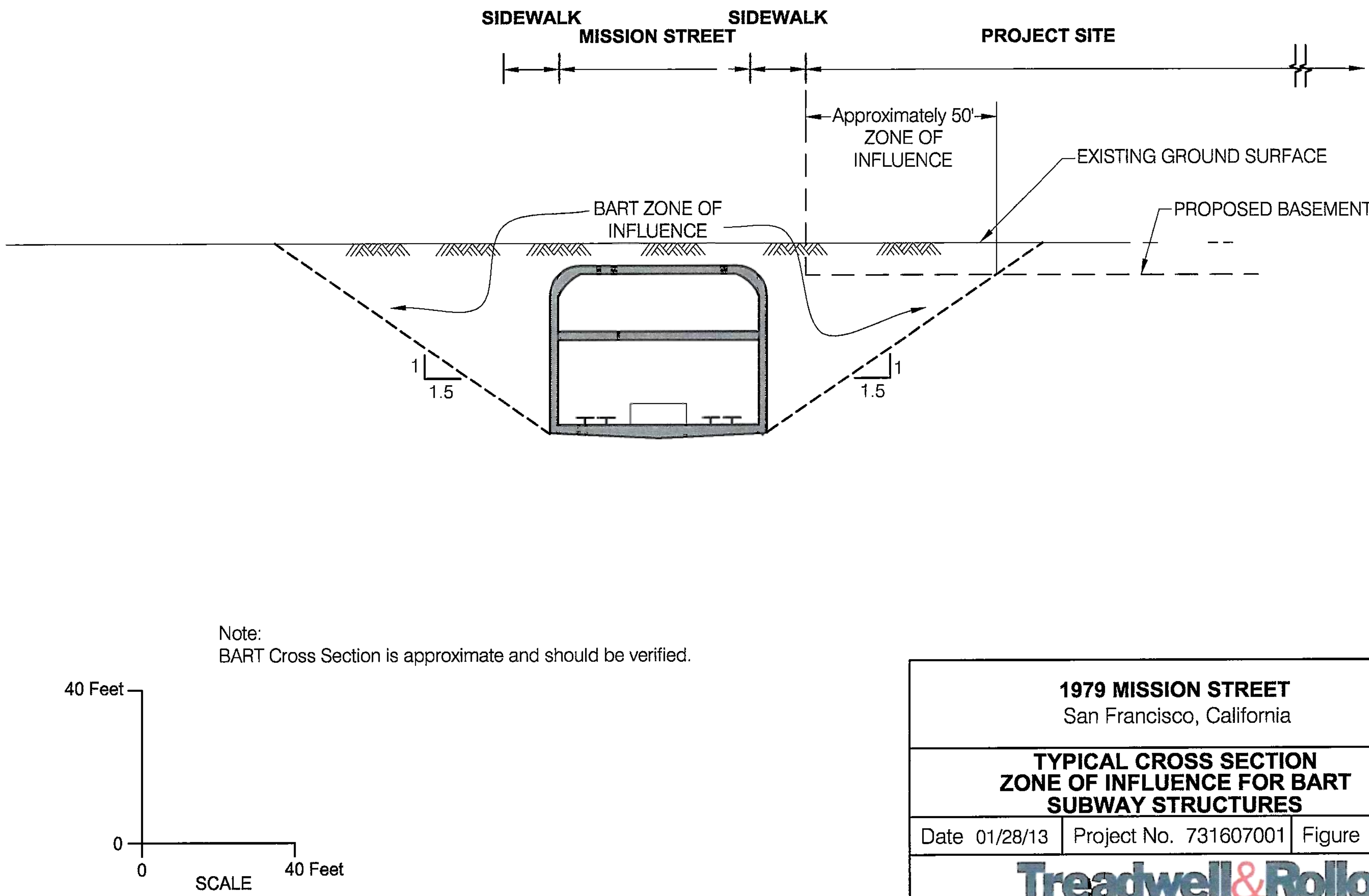

Note:
BART Cross Section is approximate and should be verified.

1979 MISSION STREET San Francisco, California		
TYPICAL CROSS SECTION ZONE OF INFLUENCE FOR BART SUBWAY STRUCTURES		
Date 01/28/13	Project No. 731607001	Figure 3

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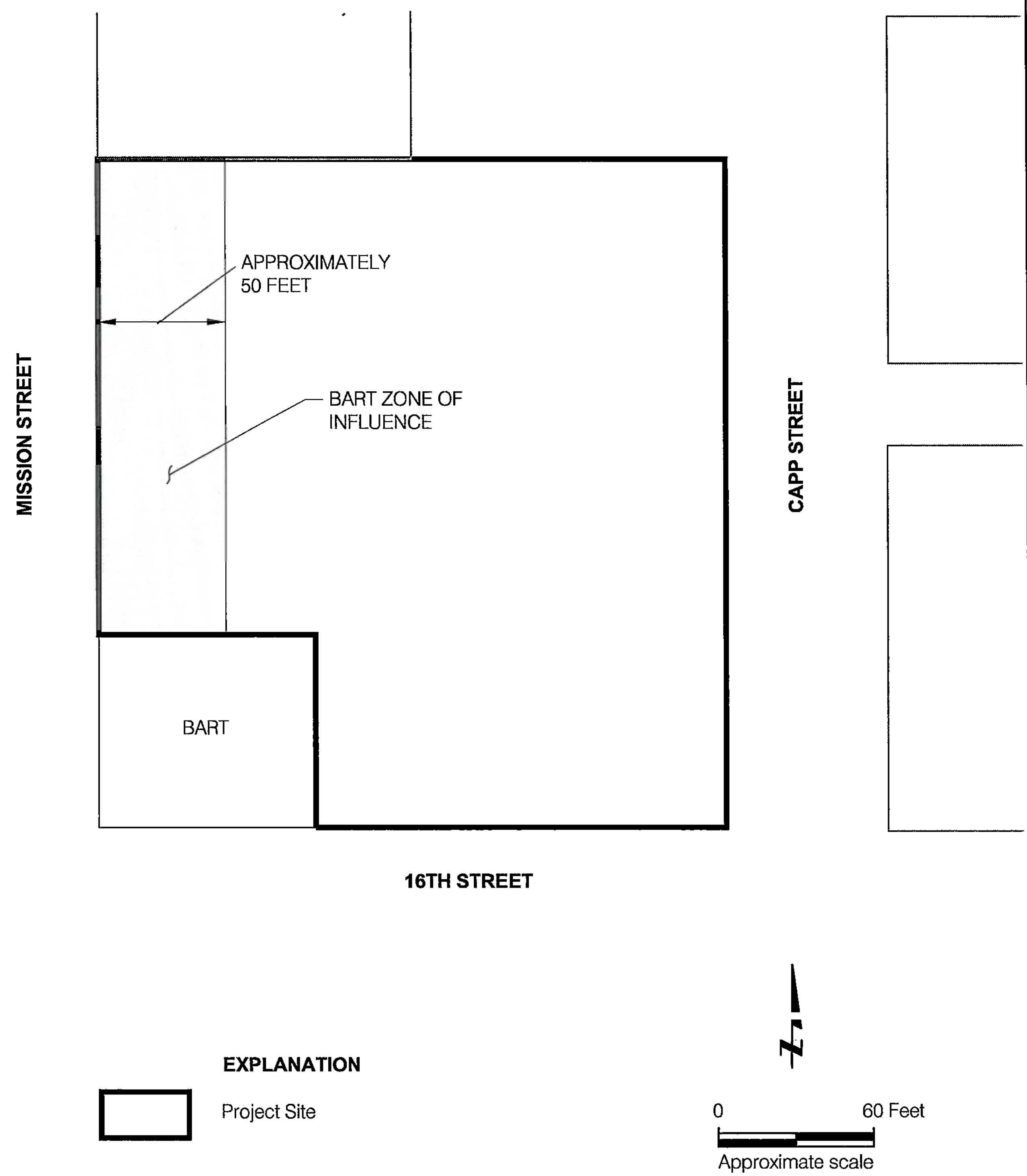

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San Francisco, California

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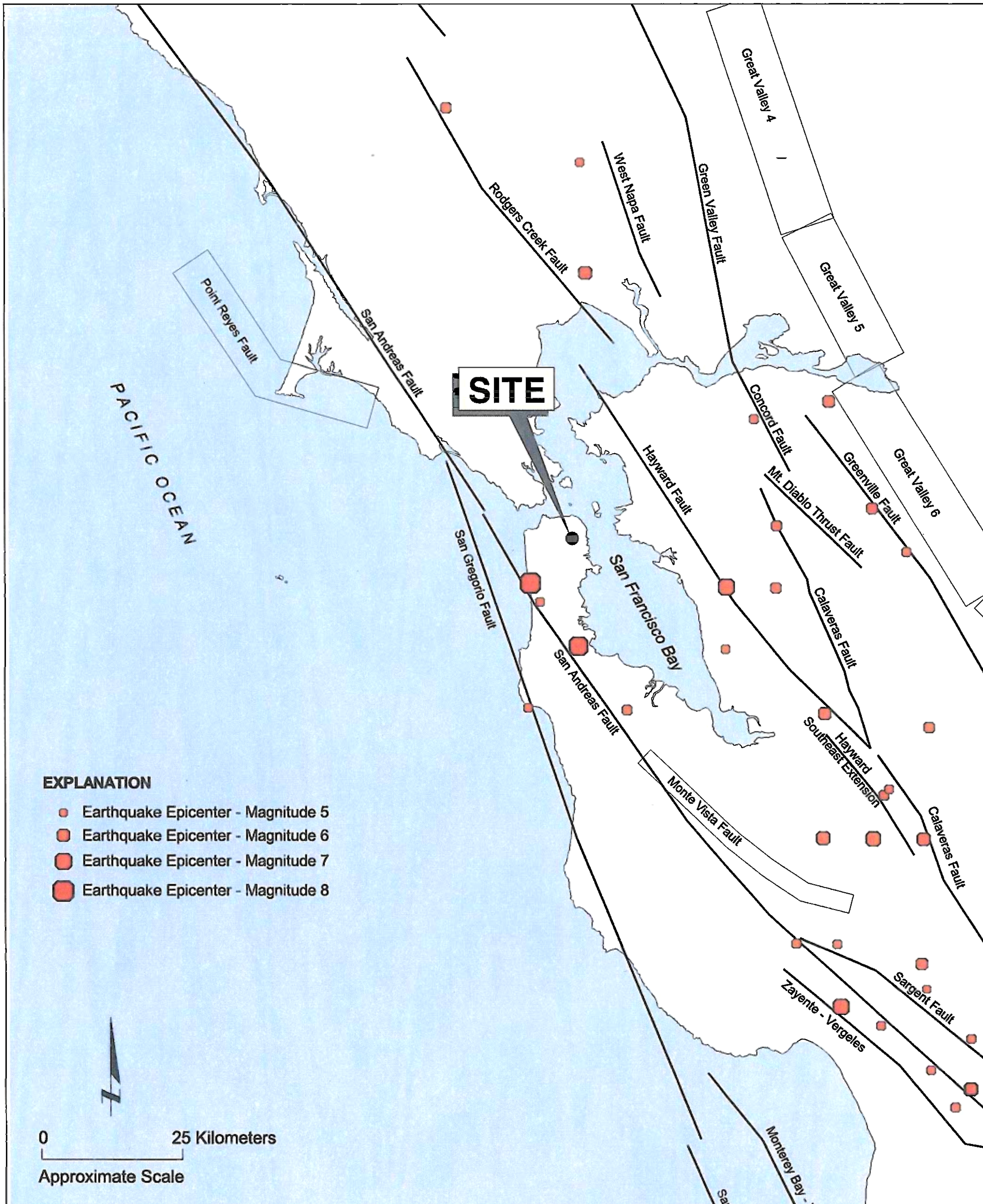
BART ZONE OF INFLUENCE

Date 01/14/13	Project No. 731607001	Figure 4



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

1979 MISSION STREET San Francisco, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
Treadwell&Rollo A LANGAN COMPANY	Date 01/14/13 | Project No. 731607001 | Figure 5

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.

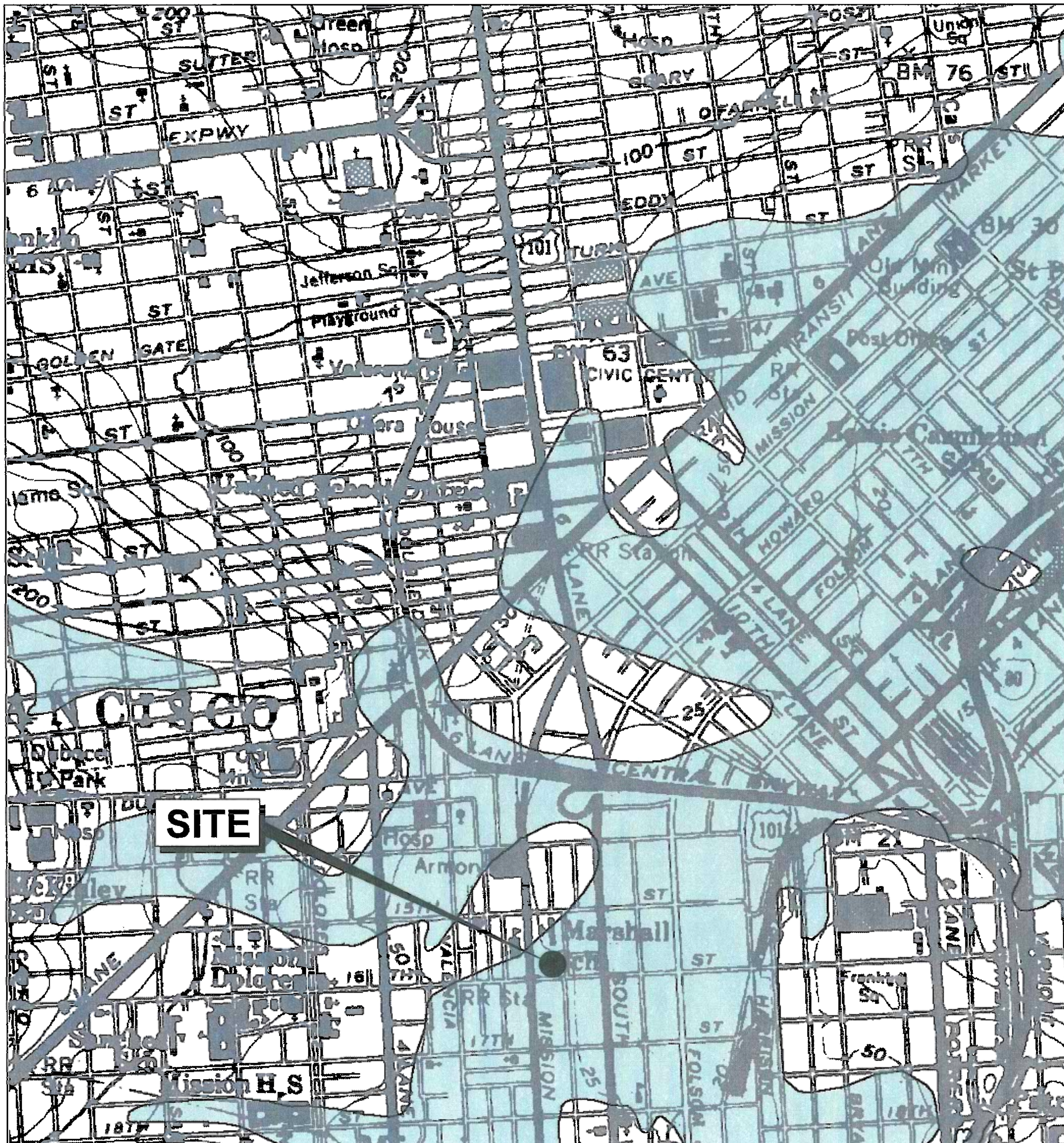
IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others 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.

1979 MISSION STREET San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**		
Treadwell&Rollo *A LANGAN COMPANY*	Date 01/14/13	Project No. 731607001	Figure 6



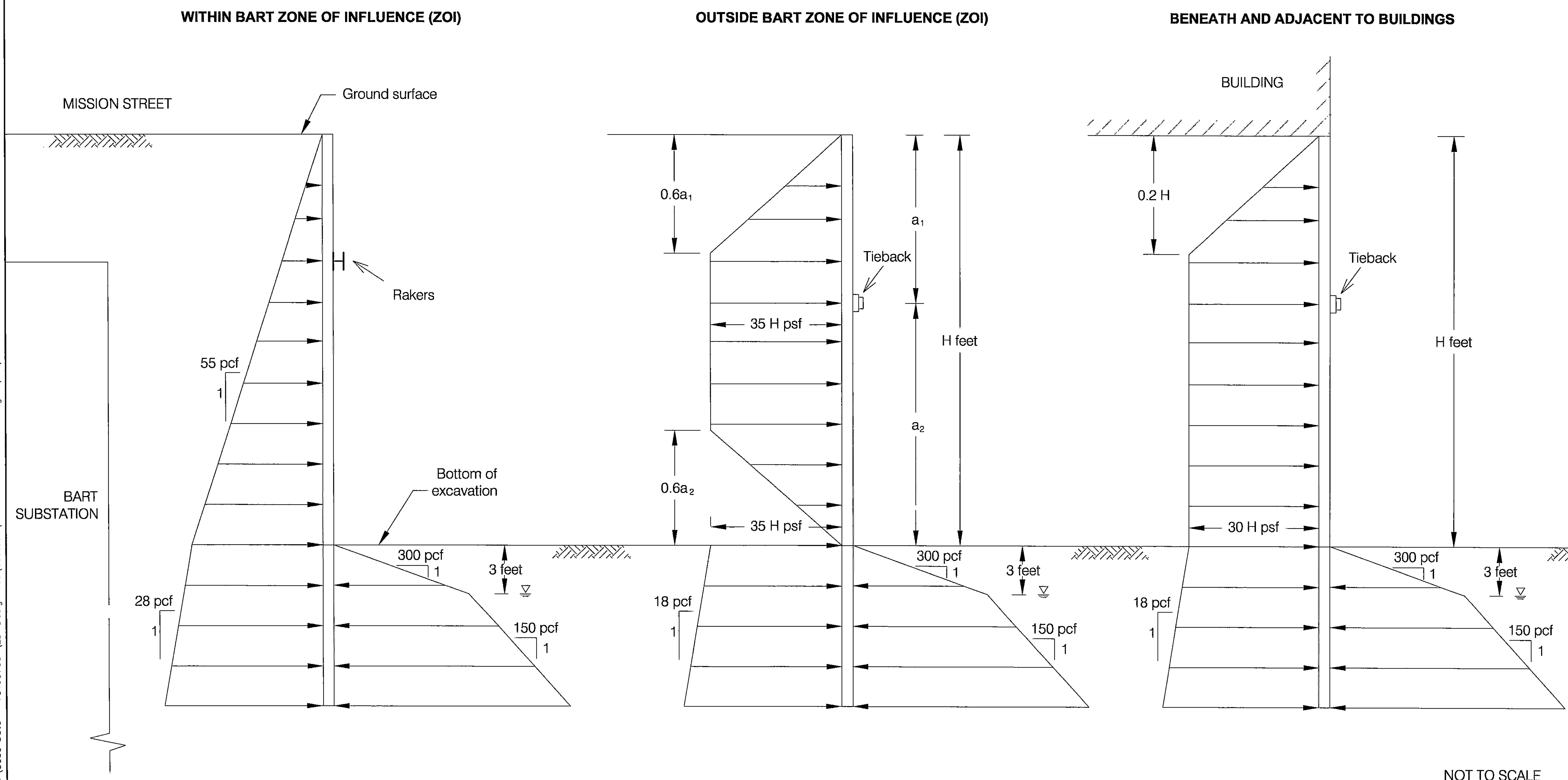
EXPLANATION

Zone of Liquefaction

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

0 1,000 2,000 Feet
Approximate scale

1979 MISSION STREET San Francisco, California	**REGIONAL SEISMIC HAZARD ZONE MAP**
Treadwell&Rollo *A LANGAN COMPANY*	Date 01/14/13 · Project No. 731607001 · Figure 7



Notes:

1. Passive pressure includes a factor of safety of 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters (assume structural concrete is used for backfill).
3. Active pressure below excavation level should be assumed to act over one pile diameter.
4. The shoring pressures do not include surcharge pressures from existing building, construction equipment, and traffic. These surcharge pressures should be added, as appropriate.

1979 MISSION STREET
San Francisco, California

TEMPORARY SHORING DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING SYSTEM

Date 01/28/13	Project No. 731607001	Figure 8

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APPENDIX A

Boring Logs and Classification Chart

PROJECT:	**1979 MISSION STREET** San Francisco, California	**Log of Boring TR-1** PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon

Date started: 1/4/13 | Date finished: 1/4/13

Drilling method: Rotary Wash

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

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

Ground Surface Elevation: 17 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	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						3.5-inches Asphalt Concrete (AC)						
0–1						3-inches Aggregate Base (AB)						
1–2.5					SM	SILTY SAND (SM) yellow-brown, moist (FILL)						
2.5–9					SP	SAND (SP) yellow-brown to olive-brown, very loose to loose, moist to wet, with mica particles						
5–6.5	S&H		1 2 3	4	SP							
7						∇ (1/4/2013, 8:15 AM)						
9–14.5					SC	CLAYEY SAND (SC) dark olive-gray to black, very loose, wet LL = 26, PL = 15, PI = 11, see Figure B-1						
10–11.5	S&H		0 1 2	2	SC					40.2	25.2	100
14.5–24.5					SP	SAND (SP) olive-brown, dense, wet, trace silt						
15–16.5	S&H		12 22 28	35	SP						19.6	105
20–21.5	SPT		14 17 23	48	SP	yellow-brown						
24.5–26.5					SC	CLAYEY SAND (SC) red-brown, medium dense, wet						
25–26.5	SPT		12 9 10	23	SC					45.1	17.0	
26.5–30					SC-SM	CLAYEY SILTY SAND (SC-SM) yellow-brown, medium dense, wet						

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Project No.: 731607001 | Figure: A-1a

PROJECT:	1979 MISSION STREET San Francisco, California	Log of Boring TR-1
		PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		6 9 13	15	SC-SM	CLAYEY SILTY SAND (SC-SM) (continued) LL = 21, PL = 17, PI = 4, see Figure B-1				36.8	17.7	
32–34												
35–36	SPT		17 24 40	77	SC	CLAYEY SAND (SC) orange-brown, very dense, wet				13.1	21.8	
37–39												
40–41	S&H		40 50/ 5.5"	35/ 5.5"	SC							
42–44												
45–46	SPT		19 23 26	59		olive-brown						
47												
48–49					SP-SM	SAND with SILT (SP-SM) yellow-brown, very dense, wet						
50–51	S&H		30 50/6"	35/6"								
52–60												

Boring terminated at a depth of 51 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 7 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on San Francisco City datum.

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Project No.: 731607001

Figure: A-1b

PROJECT:	**1979 MISSION STREET** San Francisco, California	**Log of Boring TR-2** PAGE 1 OF 4

Boring location:	See Site Plan, Figure 2
Date started:	1/3/13
Date finished:	1/3/13
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Magallon

MATERIAL DESCRIPTION — Ground Surface Elevation: 19 feet[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3-inches Asphalt Concrete (AC)						
					3-inches Aggregate Base (AB)						
1–3				SM	SILTY SAND (SM) brown, moist, trace brick and cloth debris (FILL)						
3				SP	SAND (SP) olive-brown, medium dense, moist, fine-grained, trace mica particles						
5–6.5	S&H	6 9 10	13	SP							
8				SP	▽ (1/3/2013, 9:18 AM)						
10–11.5	SPT	2 5 6	13	SP	wet, rapid dilatancy						
13.5				CL	CLAY with SAND (CL) olive gray, stiff, wet, fine grained sand, trace gravel						
15–16.5	SPT	0 2 7	11	CL							
16.5				SM	SILTY SAND (SM) olive-brown, medium dense, wet						
17				SM					17.7	23.9	
20				SM	SILTY SAND (SM) yellow-brown to orange-brown, medium dense, wet						
20–21.5	SPT	7 9 9	22	SM							
25–26.5	S&H	8 5 8	9	CL	SANDY CLAY (CL) red brown, stiff, wet, fine grained sand						
29.5–30				SM							

Project No.: 731607001

Figure: A-2a

TEST GEOTECH LOG 731607001.GPJ TR.GDT 1/25/13

PROJECT:	1979 MISSION STREET San Francisco, California	Log of Boring TR-2

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	23 40 50/6"	63	SM	SILTY SAND (SM) yellow-brown, very dense, wet						
36	SPT	15 20 33	64	SP-SM	SAND with SILT (SP-SM) yellow-brown mottled olive, very dense, wet						
41	SPT	19 33 43	91								
43				SM	SILTY SAND (SM) yellow-brown to brown, dense, wet						
46	S&H	37 50/6"	35/6"							19.9	107
51	SPT	25 27 32	71								
56	SPT	25 50/6"	60/6"								
60											

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Project No.: 731607001

Figure: A-2b

TEST GEOTECH LOG 731607001.GPJ TR.GDT 1/25/13

PROJECT:	1979 MISSION STREET San Francisco, California	Log of Boring TR-2 PAGE 3 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		27 42 50/5"	64/ 11"	SM	SILTY SAND (SM) (continued)						
62												
63												
64												
65–66	SPT		22 34 40	89		brown to olive-brown, decrease in grain size						
67					SP-SM	SAND with SILT (SP-SM) olive brown, very dense, wet						
68												
69												
70–71	SPT		28 50/6"	60/6"						7.9	23.0	
72												
73												
74												
75–76	S&H		50 50/ 4.5"	35/ 4.5"								
77												
78												
79												
80–81	SPT		28 50/6"	60/6"								
82												
83					SP	SAND (SP) olive, very dense, wet, trace silt						
84												
85–86	S&H		29 50/6"	35/6"								
87												
88												
89												
90												

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Project No.: 731607001

Figure: A-2c

PROJECT:	1979 MISSION STREET San Francisco, California	Log of Boring TR-2 PAGE 4 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	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
91	SPT		30 50/6"	60/6"	SP	SAND (SP) (continued)						
92												
93												
94												
95												
96												
97												
98												
99												
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 91 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 8.4 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on San Francisco City datum.

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Project No.: 731607001 | Figure: A-2d

TEST GEOTECH LOG 731607001.GPJ TR.GDT 1/25/13

PROJECT:	**1979 MISSION STREET** San Francisco, California	**Log of Boring TR-3** PAGE 1 OF 2

Boring location: See Site Plan, Figure 2		Logged by: S. Magallon
Date started: 1/4/13	Date finished: 1/4/13	
Drilling method: Rotary Wash		
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Automatic Hammer	LABORATORY TEST DATA
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)		

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 17.5 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					4-inches Asphalt Concrete (AC)						
1–4				SM	SILTY SAND (SM) brown to yellow-brown, moist, trace brick debris (FILL)						
4–6	S&H	4 5 6	8	SP	SAND (SP) yellow-brown, loose, moist						
8					▽ (1/4/2013, 13:30 PM)						
9–10	SPT	200-450 psi		ML	SANDY SILT (ML) dark olive brown, stiff, wet						
10–11	S&H	3 14 28	29	SM	SILTY SAND (SM) yellow-brown to olive-brown, medium dense, wet				20.0	15.1	
13	DIST										
14					trace gray, red and green cobbles						
16	S&H	15 19 24	30		olive-brown, trace silt						
20–21	SPT	5 3 2	6	CL	SANDY CLAY (CL) red-brown, medium stiff, wet						
23–26	SPT	7 9 12	25	SC-SM	CLAYEY SILTY SAND (SC-SM) yellow-brown mottled red-brown, medium dense, wet, fine-grained sand; LL = 25, PL = 18, PI = 5, see Figure B-1				21.3	21.5	
28–29				SP	SAND (SP) red-brown, very dense, wet, trace silt						
30											

PROJECT:	**1979 MISSION STREET** San Francisco, California	**Log of Boring TR-3** PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		17 53 44	54	SP	SAND (SP) (continued)						
32												
33					SP-SM	SAND with SILT (SP-SM) yellow-brown to orange-brown, very dense, wet						
34												
35–36	SPT		17 35 43	94								
37–39												
40–41	SPT		19 26 44	84								
42–44												
45	S&H		35 50/5"	35/5"		olive-brown, decrease silt						
46												
47					SP	SAND (SP) yellow-brown mottled orange-brown, very dense, wet						
48–49												
50–51	SPT		15 20 30	60								
52–60												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 8 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on San Francisco City datum.

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Project No.: 731607001

Figure: A-3b

TEST GEOTECH LOG 731607001.GPJ TR.GDT 1/25/13

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample, grab groundwater

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

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

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San Francisco, California

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CLASSIFICATION CHART

Date 01/10/13 | Project No. 731607001 | Figure A-4

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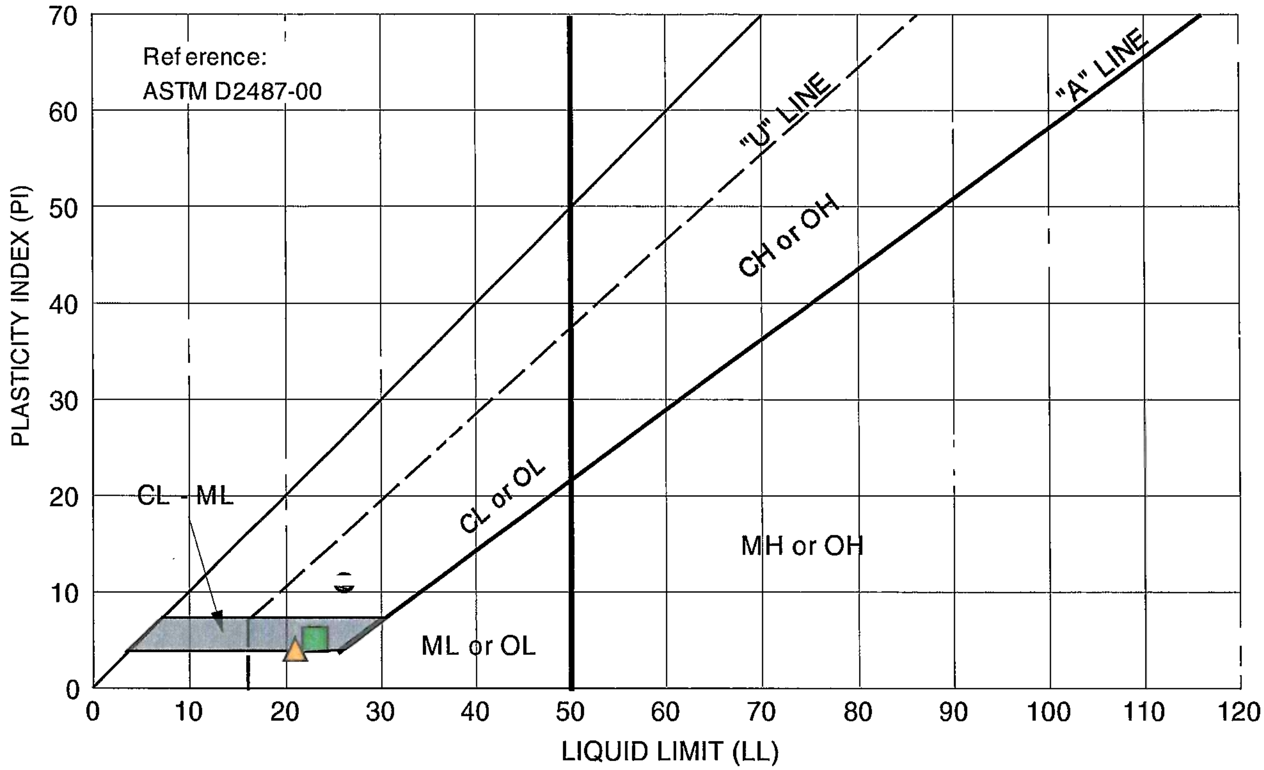
APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
● (blue circle)	TR-1 at 11 feet	CLAYEY SAND (SC), dark olive-gray to black	25.2	26	11	40.2
△ (orange triangle)	TR-1 at 31 feet	CLAYEY SILTY SAND (SC-SM), yellow brown	17.7	21	4	36.8
■ (green square)	TR-3 at 25 feet	CLAYEY SILTY SAND (SC-SM), yellow-brown mottled red-brown	21.5	23	5	21.3

1979 MISSION STREET
San Francisco, California

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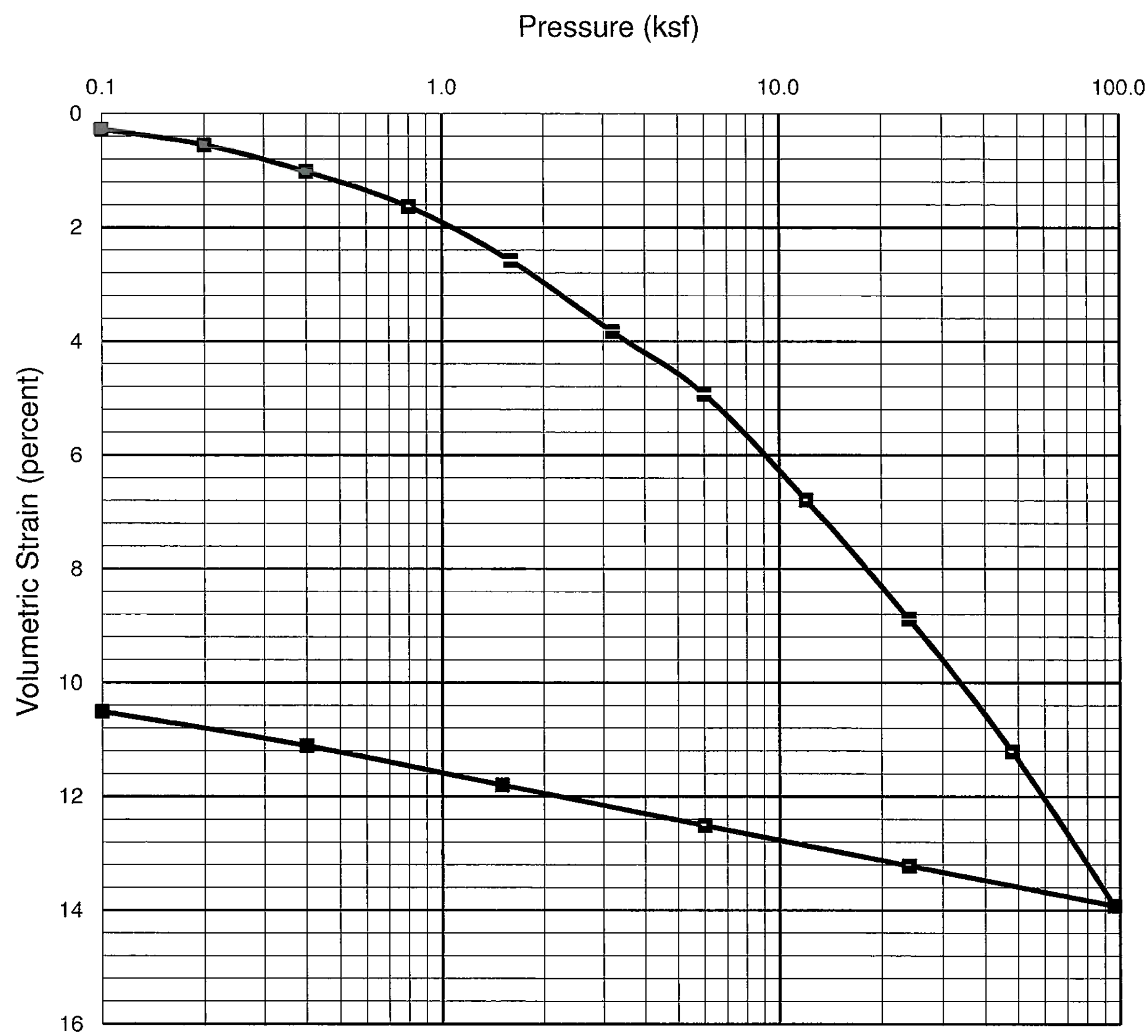
PLASTICITY CHART

Date 01/25/13	Project No. 731607001	Figure B-1



Sampler Type: Sprague & Henwood				Condition	Before Test			After Test	
Diameter (in) 2.42	Height (in)	1.00		Water Content	w_o	18.9 %		w_f	14.0 %
Overburden Pressure, p_o	2,030	psf		Void Ratio	e_o	0.53		e_f	0.37
Preconsol. Pressure, p_c	2,030	psf		Saturation	S_o	96 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$				Dry Density	γ_d	110 pcf		γ_d	123 pcf
LL	PL		PI			G_s	2.70	(assumed)	
Classification SANDY CLAY (CL), red brown				Source	TR-2 @ 26 feet				

1979 MISSION STREET
San Francisco, California

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CONSOLIDATION TEST REPORT

Date 01/30/13 | Project No. 731607001 | Figure B-2

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APPENDIX C

Laboratory Corrosion Test Results

California State Certified Laboratory No. 2153

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

28 January, 2013

Job No.1301059
Cust. No.10727

Ms. Kristen Lease
Treadwell & Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111

Subject: Project No.: 731607001
Project Name: 1979 Mission St., San Francisco
Corrosivity Analysis – ASTM Methods

Dear Ms. Lease:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on January 09, 2013. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentrations reflect none detected with a detection limit of 15 mg/kg.

The sulfate ion concentrations ranged from none detected to 23 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.5 to 7.7, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 440 to 490-mV, which are indicative of aerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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

Client: Treadwell & Rollo
Client's Project No.: 731607001
Client's Project Name: 1979 Mission St., San Francisco
Date Sampled: Not Indicated
Date Received: 9-Jan-13
Matrix: Soil
Authorization: Transmittal dated 01/08/13

Date of Report: 28-Jan-2013

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1301059-001	TR-1 @ 5.5'	440	7.7	-	5,200	-	N.D.	23
1301059-002	TR-2 @ 6'	490	7.5	-	3,700	-	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	21-Jan-2013 & 28-Jan-2013	18-Jan-2013	-	17-Jan-2013	-	18-Jan-2013	18-Jan-2013

Cheryl McMillen
Cheryl McMillen
Laboratory Director

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

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

APPENDIX D

San Francisco Bay Area Rapid Transit District – General Guidelines for Design and Construction over or adjacent to BART Subway Structures

SAN FRANCISCO BAY AREA RAPID TRANSIT DISTRICT

GENERAL GUIDELINES FOR DESIGN AND CONSTRUCTION OVER OR ADJACENT TO BART'S SUBWAY STRUCTURES

1. Structures over or adjacent to BART's subway structures shall be designed and constructed so as not to impose any temporary or permanent adverse effects on subway. The minimum clearance between any part of the adjacent structures to exterior face of substructures shall be 7'-6". Minimum cover of 8 feet shall be maintained wherever possible.

2. In general, cut-and-cover subway structures were designed with an area surcharge applied at the ground surface both over and adjacent to the structures. The area surcharge was considered static uniform load with the following value:

D (ft)	**Additional Average Vertical Loading (psf)**
D>20	0
5<D<20	800-40D
D<5	600

   Where **D** is the vertical distance from the top of the subway roof to the ground surface.

3. In general, steel-lined tunnels were designed to support the weight of 35 feet of earth above the roof of the tunnel. Whenever the actual depth of cover is less than this amount, construction may be added imposing an additional average vertical loading of 120 lbs. per square foot for each foot of depth of reduced cover. Where basements are excavated, the allowable additional average vertical loading can be increased to the extent that it is balanced by the weight of the removed material. The effects of soil rebound in such cases shall be fully analyzed.

4. Shoring is required for excavations in the Zone of Influence. Zone of Influence is defined as the area above a Line of Influence which is a line from the critical point of substructure at a slope of 1 ½ horizontal to 1 vertical (line sloping towards ground level).

5. Shoring shall be required to maintain at-rest soil condition and monitored for movement.

6. Soil redistribution caused by temporary shoring or permanent foundation system shall be analyzed.

7. Dewatering shall be monitored for changes in groundwater level. Recharging will be required if existing groundwater level is expected to drop more than 2 feet.

8. Piles shall be predrilled to a minimum of 10 feet below the Line of Influence. Piles shall be driven in a sequence away from BART structures. No pile will be allowed between steel-lined tunnels.

9. Subway structures shall be monitored for vibration during pile driving operations for all piles within 100 feet of the structures. Steel –lined tunnels shall also be monitored for movement and deformation. Requirements for monitoring will be provided upon request.

10. Excavation shall be done with extreme care to prevent damage to the waterproofing membrane and the structure itself. Hand excavation shall be performed for the final one foot above the subway roof.

The above shall be considered as general information only and is not intended to cover all situations. Notwithstanding these guidelines, pertinent design and construction documents shall be submitted to BART for review and approval. In addition, the following shall be submitted as applicable:

- Geologic Hazards Evaluation and Geotechnical Investigation reports. The reports shall include engineering geology map, site plan showing the location of subway structures, BART easement, soil reworking plan and the geological conclusion and recommendations.
- Dewatering monitoring and recharging plans.
- Vibration monitoring plan and/or movement and deformation monitoring plans for steel-lined tunnels. Plans shall include locations and details of instruments in subways.
- Foundation plan showing the anticipated total foundation loads.
- Excavation plan for area within the Zone of Influence showing excavation slope or shoring system.
- Procedures and control of soil compaction operation.

Treadwell&Rollo
A LANGAN COMPANY

DISTRIBUTION

3 copies: Mr. Seth Mallen
Principal
Maximus Real Estate Partners
345 Vidal Drive
San Francisco, California 94132