REVISED GEOTECHNICAL INVESTIGATION
MUSEUM PLACE
San Jose, California

Prepared For:
Insight Realty Company
333 W. Santa Clara Street, Suite 805
San Jose, California 95113

Prepared By:
Langan Treadwell Rollo
4030 Moorpark Street, Suite 210
San Jose, California 95117

REGISTERED PROFESSIONAL ENGINEER
GABRIEL J. ALCANTAR
No. C 83967
EXP. 09/30/17
CIVIL
STATE OF CALIFORNIA

Gabriel J. Alcantar

Gabriel J. Alcantar, P.E #83967
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
SERENA T. JANG
No. 2702
EXP 06/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

Serena Jang, G.E. #2282
Associate

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770627701

LANGAN TREADWELL ROLLO

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Langan Treadwell Rollo for the proposed Museum Place development at 180 Park Avenue in San Jose, California. The site is on the south side of Park Avenue between S. Almaden Boulevard and S. Market Street, as shown on Figure 1. It is bound by Park Avenue to the north, a two-story concrete parking garage and an open asphalt parking lot to the west, the City of San Jose's National Civic Center buildings to the south and the Tech Museum of Innovation (Tech Museum) building to the east. Currently, the site is occupied by a one-story structure designated as Parkside Hall and a pedestrian corridor (also formally known as S. Almaden Avenue), which are owned by the City of San Jose.

The proposed project will consist of constructing a 270-foot high mixed-use tower that will include retail, banquet, office, condominiums and hotel space above three basement levels. The basement levels and first level will be used mostly for parking, retail, lobby and banquet space. The proposed building will occupy the entire footprint of the project site, as shown on Figure 2.

On the basis of discussions with Steinberg Architects, the project architect, the proposed finish floor elevation of the ground floor will be at Elevation 88 feet[1]. Based on our review of structural plans prepared by Magnusson Klemencic Associates (MKA)[2], the project structural engineer, the proposed building's basement footprint will extend beyond the footprint of the ground floor level as shown on Figure 2. The basement will extend three-levels below grade with the lowest basement finish floor at Elevation 49 feet, corresponding to a depth of 39 feet below the proposed ground floor elevation. According to the structural drawings, the thickness of the proposed mat foundation is six feet, except in the deepened core areas shown on

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[1] All elevations reference NGVD 29.

[2] Sheets S2.P3A, S2.P3B, S2.P3C and S2.P3D, Foundation Plan – Level P3, Basement, Museum Place, File Name"2016 0525_50% Structural.pdf", by MKA

Figure 2, where the thickness of the mat is eight feet. In addition, a one foot thick working pad is planned. Therefore, the bottom of excavation will range from Elevation 40 to 42 feet, corresponding to a depth of 46 to 48 feet below the proposed ground floor elevation.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our revised proposal dated 23 November 2015. We reviewed available subsurface information for the site and vicinity from our files and further explored subsurface conditions at the site by drilling borings and advancing cone penetrometer tests (CPTs). We then conducted laboratory tests on samples recovered from the borings and used the results from our field exploration to perform engineering analyses and develop conclusions and recommendations regarding:

- anticipated subsurface conditions including estimates of groundwater levels;
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, and fault rupture;
- appropriate foundation type(s);
- design parameters for the recommended foundation type(s), including vertical and lateral capacities and associated estimated settlements;
- appropriate temporary shoring type and associated lateral earth pressures and tieback design criteria;
- subgrade preparation for slab-on-grade floors and exterior slabs and flatwork, including sidewalks;
- site preparation, grading, and excavation, including criteria for fill quality and compaction;
- 2013 California Building Code (CBC) site classification, mapped values S_S and S_1, modification factors F_a and F_v and S_{MS} and S_{M1};
- soil corrosivity with brief evaluation; and
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We began our investigation by reviewing previous geotechnical investigations performed in the vicinity of the site. To further investigate subsurface conditions at the site, we drilled three borings and advanced four Cone Penetration Tests (CPTs).

Prior to performing the field investigation, we:

- obtained drilling permits from the Santa Clara Valley Water District (SCVWD)
- notified Underground Service Alert (USA)
- checked the boring and CPT locations for underground utilities using a private utility locator
- participated in a pre-investigation site walk to coordinate site access with Team San Jose, The Tech Museum, Steinberg Architects and both drilling and CPT subcontractors.

Details of the field exploration activities and laboratory testing are described in the remainder of this section.

3.1 Previous Investigation

We reviewed existing subsurface information from the following reports:

- Geotechnical Investigation, 200 Park Avenue, San Jose, California," dated 13 April 1998, by Treadwell & Rollo, Inc.
- Geotechnical Investigation, 177 Park Avenue, San Jose, California," dated 15 October 2001, by Treadwell & Rollo, Inc.

We used the information provided on the boring and CPT logs from the above referenced reports to supplement the information developed from our exploration of the site. The approximate locations of the previously drilled borings and CPTs in the vicinity of the site are presented on Figure 2.

3.2 Borings

The test borings, designated B-1, B-2 and B-3, were drilled at the site at the approximate locations shown on Figure 2. The borings were drilled between 25 and 27 January 2016 by Pitcher Drilling Company of Palo Alto, California using a track-mounted drilling rig (Fraste Multidrill XL) with rotary wash equipment and a PD-81 automatic hammer. The borings were advanced to depths of 90 to 101½ feet bgs. During drilling, the soil was logged and samples of the material encountered were obtained for visual classification and laboratory testing. The boring logs are presented in Appendix A as Figures A-1 through A-3. The soil was logged in accordance with the soil classification system described on Figure A-4.

Soil samples were obtained using three different types of samplers: two driven split-barrel samplers and one pushed thin-walled sampler. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches

- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners

- Shelby Tube (ST) with a 3.0-inch outside diameter and a 2.875-inch inside diameter.

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

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.8 and 1.4, respectively, to account for sampler type and hammer energy[3], and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts.

The Shelby Tube sampler was pushed hydraulically into the soil; the pressure required to advance the sampler is shown on the boring logs, measured in pounds per square inch (psi).

Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD. The soil cuttings and drilling fluid from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and transported off-site for proper disposal.

3.3 Cone Penetrometer Tests (CPTs)

To supplement the soil boring information, four CPTs, designated CPT-1 through CPT-4, were performed on 27 January 2016 by CPT, Inc. of San Leandro, California at the locations shown on Figure 2. The CPTs were advanced to depths ranging from approximately 62 to 100 feet below existing finish floor of Parkside Hall. CPT-2, CP-3 and CPT-4 were terminated at depths of about 62, 95 and 92 feet, respectively where refusal was encountered.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe, with a projected area of 10 square centimeters, into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges or load cells within the cone continuously measured the cone tip resistance and frictional resistance during the entire depth of each probing. Accumulated data was processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance, side friction and friction ratio by depth, as well as interpreted SPT N-Values and interpreted soil classification, are presented in Appendix B on Figures B-1 through B-4. Soil types were estimated using the classification chart shown on Figure B-5.

Pore-pressure dissipation tests (PPDTs) were performed during the advancement of CPT-1 through CPT-4 at various depths. PPDTs were conducted to measure hydrostatic water pressures and to determine the approximate depth to groundwater. The variation of pore

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[3] Hammer energy is based on standard penetration energy measurements for automatic hammer, PD-81, provided by Pitcher Drilling. (Gregg Drilling and Testing, Inc., 2016)

pressure with time is measured behind the tip of the cone and recorded. For this investigation, the duration of the tests range from approximately 260 to 370 seconds. The results of the four PPDTs are presented in Appendix B on Figures B-6 through B-11.

After completion, the CPTs were backfilled with cement grout in accordance with SCVWD requirements.

3.4 Laboratory Testing

The soil samples collected from the field exploration program were reexamined in the office for soil classifications, and representative samples were selected for laboratory testing. The laboratory testing program was designed to evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, plasticity (Atterberg Limits), gradation, shear strength, compressibility, and corrosion characteristics, where appropriate. Results of the laboratory testing are included on the boring logs and in Appendix C on Figures C-1 through C-11.

3.5 Soil Corrosivity Testing

To evaluate the corrosivity of the soil near the foundation subgrade, we performed corrosivity tests on samples obtained from the upper three feet. The corrosivity of the soil samples was evaluated by CERCO Analytical using the following ASTM Test Methods:

- Redox – ASTM D1498
- pH – ASTM D4972
- Resistivity (100% Saturation) – ASTM G57
- Sulfide – ASTM D4658M
- Chloride – ASTM D4327
- Sulfate – ASTM D4327

The laboratory corrosion test results and a brief corrosivity evaluation by JDH Corrosion are presented in Appendix D.

4.0 SITE AND SUBSURFACE CONDITIONS

The existing site and subsurface conditions observed and encountered at the site, respectively, are discussed in this section.

4.1 Site Conditions

The site is currently occupied by a one-story structure (designated as Parkside Hall) and a public pedestrian corridor located west of Parkside Hall; both are owned by the City of San Jose. Built in the early 1970's, Parkside Hall is used as an exhibition hall with up to 30,000 square feet of floor space and a 24-foot high ceiling (W. Hedley Jr., 1975). A topographic site plan (Kier & Wright, 2016) indicates the finished floor elevation of the exhibit area within Parkside Hall is at Elevation 89.7 feet. The existing concrete slab for Parkside Hall was measured to be approximately seven inches thick in Borings B-2 and B-3; the slab is underlain by plastic sheeting.

The existing pedestrian corridor, also known as S. Almaden Avenue, is west of Parkside Hall and consists of concrete hardscapes and landscaping. Ground surface elevation along the pedestrian corridor ranges from Elevation 87.9 to 88.9 feet. To the west and southwest are three parcels consisting of asphalt parking, a two-story concrete public parking garage and Hyatt Place Hotel. To the south and southeast are two to three-story buildings known as the McCabe Hall and Montgomery Theatre (as known as the Civic Center) building. Currently, the foundations of the existing structures west of the site, McCabe Hall and Montgomery Theatre building are unknown.

The existing Tech Museum building is immediately east of the project site. Based on a review of available construction set drawings (The Steinberg Group/ Legorreta Arquitectos, 1996), the building was constructed in 1996 and consists of three levels with exhibit halls, recreational space and a basement level with the finish floor at Elevation 68.5 feet. The building is supported on a 2.75 to 3-foot thick reinforced concrete mat foundation. In addition, it is our understanding, a portion of west perimeter of the Tech Museum is at-grade supported by drilled piers (The Steinburg Group/Legorreta Arquitectos, 1996).

4.2 Subsurface Conditions

The site appears to be blanketed by 3 to 3½ feet thick of fill that consists of sand with gravel. The fill is underlain by alluvial deposits consistent with the geology of the region. The alluvial deposits generally consist of medium stiff to hard clays and silts with interbedded layers of medium dense to very dense sands and gravels to the maximum depth explored. At approximately 37 to 54 feet bgs (corresponding to Elevations 51.4 to 35.7 feet), a 20 to 32 feet thick layer of dense to very dense sand and gravel layer with varying amount of fines was encountered in all of the CPTs and borings.

Where tested, the clays above the sand and gravel layer are normally to overconsolidated[4] and have shear strengths ranging from about 720 pounds per square foot (psf) to 1,160 psf. Where tested, the clay encountered below the sand and gravel layer is normally consolidated with shear strengths ranging from 1,600 to 1,710 psf.

Based on our review of published maps (California Division of Mines and Geology, 2002) and nearby geotechnical investigations (Treadwell & Rollo, 1998, 2001), historic high groundwater in the project vicinity is approximately 15 to 20 feet bgs. During our current investigation, the groundwater levels measured in PPDTs ranged from approximately 29 to 31.7 bgs (corresponding to Elevations 59.5 to 60 feet). Groundwater was encountered at Boring B-2 at a depth of approximately 22 bgs, corresponding to Elevation 67.6 feet. However, this depth was measured during drilling and may not represent a stabilized ground water level. Groundwater levels may fluctuate due to seasonal rainfall.

5.0 REGIONAL SEISMICITY

The major active faults in the area are the San Andreas, Hayward, and San Gregorio faults. These and other faults of the region are shown on Figure 3. For each of the active faults within a distance of about 100 kilometers, the distance from the site and estimated mean characteristic Moment magnitude[5] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

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[4] A normally consolidated clay has completed consolidation under the existing load; and an overconsolidated clay has experienced a pressure greater than its current load.

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

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	11	Southwest	6.50
Total Calaveras	14	Northeast	7.03
Total Hayward	14	Northeast	7.00
Total Hayward-Rodgers Creek	14	Northeast	7.33
N. San Andreas – Peninsula	19	Southwest	7.23
N. San Andreas (1906 event)	19	Southwest	8.05
N. San Andreas – Santa Cruz	20	Southwest	7.12
Zayante-Vergeles	28	Southwest	7.00
Greenville Connected	36	East	7.00
San Gregorio Connected	43	West	7.50
Mount Diablo Thrust	45	North	6.70
Monterey Bay-Tularcitos	50	Southwest	7.30

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. 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 4) 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 Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 33 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 ($M_w = 6.2$).

The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 106 kilometers north of the site, with a M_W of 6.0.

The WGCEP (2008) 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. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2

WGCEP (2008) 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

6.0 SEISMIC 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[6], lateral spreading[7], and seismic densification[8]. Each of these conditions has been evaluated based on our literature review, field investigation, and analyses, and is discussed in this section.

6.1 Liquefaction and Associated Hazards

When a saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of shear strength as a result of a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The site is within a zone designated with the potential for liquefaction, as identified by the California Geological Survey (formerly the California Division of Mines and Geology), on map titled, State of California Seismic Hazard Zones, San Jose West 7.5-Minute Quadrangle, Santa Clara County prepared by the California Geologic Survey (7 February 2002) as shown on Figure 5. Specifically, the map shows the site is in an area "where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693 (c) would be required."

To evaluate the liquefaction potential at this site, we performed liquefaction analysis in accordance with the State of California Special Publication 117A, Guidelines for Evaluation and Mitigation of Seismic Hazards in California (2008) and followed the procedures presented in the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction

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[6] 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.

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

[8] Seismic densification (also referred to as Differential Compaction) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

Resistance of Soils (Youd and Idriss 2001). To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Tokimatsu and Seed (1987) for the borings and CPTs.

These analytical methods calculate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design level seismic event. Specifically, two distinct terms are used in the liquefaction triggering analyses:

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, relative density, depth of groundwater, earthquake magnitude, and overall soil behavior;
- Cyclic Stress Ratio (CSR), which quantifies the stresses that may develop during cyclic shaking.

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, we judge the soil layer may generate excess pore pressure and liquefy during a large seismic event.

The primary design parameters used in our liquefaction triggering calculations are summarized in Table 3.

TABLE 3
Values Used in Liquefaction Evaluation

Parameter	Value
Depth to groundwater (Historic depth to high groundwater)	Approximately 15 feet below ground surface (about Elevation 74.5 feet, NGVD29)
Peak Ground Acceleration (PGA_M)	0.5g
Predominant Earthquake Moment Magnitude (M_w)	8.0
Factor of Safety for Liquefaction Triggering	1.3
Conversion factor for SPT sampler blow count to SPT N-value	1.4
Conversion factor for S&H sampler blow count to SPT N-value	0.84
Hammer Efficiency	0.84 (Gregg Drilling and Testing, Inc., 2016)
CPT conversion factor for tip resistance to SPT N-value	4 to 5

In our analyses soil that has significant amount of plastic fines, I_c greater than 2.6 were considered too cohesive to liquefy; a corrected cone tip resistance q_{c1N} greater 160 tons per square foot (tsf) were considered too dense to liquefy. Because the predominant earthquake is a moment magnitude 8.0, the cyclic resistance ratio (CRR) has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Idriss (Youd and Idriss, 2001).

As discussion in Section 1.0, the proposed building will have a three-level basement and be supported by a thick concrete mat foundation. The thickest basement floor foundation slab has a thickness of eight feet; the excavation for the building's basement including the mat and protection slab is estimated to be approximately Elevation 40 feet (corresponding to a depth of 48 feet bgs). We understand State guidelines (SCEC, 1999) recommend a minimum depth of 50 feet below lowest proposed bottom of excavation grade for evaluation of liquefaction potential. Boring B-2 and CPT-1, which were performed to depths of approximately 100 to 101½ feet bgs, meet the State's guidelines. CPT-2, CPT-3 and CPT-4 encountered refusal at a depth of about 62, 95 and 92 feet bgs, respectively in a very dense sand with gravel layer, which is indicative that the potential for liquefaction below these depths are not an issue.

Layers of medium dense saturated sand, silty sand, sandy silt, and gravel with sand, varying in thickness from approximately ½ to 6½ feet, were encountered below the historic high groundwater level to approximately 64 feet bgs.

On the basis of the results of our analyses, we conclude several of these layers could potentially liquefy during a major earthquake and may experience liquefaction-induced settlement. A summary of our boring and CPT subsurface data for the exploration points, as well as other pertinent parameters regarding liquefaction triggering and associated settlement, are presented in Tables 4 and 5.

TABLE 4
Summary of Liquefaction Potential and Estimate Settlement from Logs of Boring Data

Boring No.	Approx. Depth (feet, bgs)	Elevation of top of layer (feet)	Layer Thickness (feet)	$(N_1)_{60\text{-}cs}$	CSR_{EQ}	$CRR_{7.5}$	Factor of Safety	Volumetric Strain ε_v (%)	Estimated Vertical Settlement (in)
B-1	29.5	60.2	1.0	9	0.34	0.10	0.30	2.65	0.3
	32.8	57.7	2.5	21	0.35	0.22	0.64	1.50	0.5
	51.0	40.7	5.0	25	0.35	0.29	0.84	1.10	0.7
	56.0	35.7	5.0	27	0.34	0.33	0.99	0.60	0.4
Total Settlement at B-1									**1.8**
B-2	26.3	66.6	6.5	23	0.35	0.26	0.74	1.35	1.1
	32.5	58.1	2.0	12	0.35	0.13	0.37	2.25	0.5
	46.3	46.6	6.5	22	0.36	0.25	0.69	1.25	1.0
Total Settlement at B-2									**2.6**
B-3	17.0	74.1	3.0	16	0.36	0.17	0.48	1.80	0.6
	25.8	66.1	4.5	25	0.35	0.28	0.82	1.10	0.6
	29.8	61.6	3.5	10	0.34	0.12	0.34	2.65	1.1
	34.3	58.1	5.5	13	0.36	0.14	0.39	2.10	1.4
	52.4	40.1	5.8	26	0.35	0.31	0.91	1.00	0.7
	58.1	34.4	5.8	25	0.33	0.30	0.89	1.00	0.7
Total Settlement at B-3									**5.1**

TABLE 5
Summary of Liquefaction Potential and Estimate Settlement from CPT Results

CPT Number	Approx. Depth (feet)	Elevation of top of layer (feet)	Layer Thickness (feet)	I_c	$(q_{c1N})_{cs}$ (tsf)	CSR_{EQ}	$CRR_{7.5}$	Factor of Safety	Volumetric Strain ε_v (%)	Estimated Vertical Settlement (in)
CPT-1	26.4	63.1	2.5	2.2	117	0.47	0.23	0.49	1.40	0.4
	32.0	57.5	0.3	2.3	84	0.49	0.14	0.27	1.80	0.1
	34.6	54.9	0.3	2.5	73	0.49	0.12	0.23	2.00	0.1
	43.0	46.5	3.8	1.9	127	0.37	0.28	0.14	1.20	0.5
Total Settlement at CPT-1										**1.1**
CPT-2	25.8	63.8	0.3	2.5	100	0.46	0.18	0.38	1.65	0.1
	26.9	62.7	1.5	2.0	113	0.47	0.22	0.46	1.40	0.2
Total Settlement at CPT-2										**0.3**
CPT-3	26.1	63.5	1.5	1.9	110	0.46	0.20	0.44	1.50	0.3
	33.0	56.6	0.8	1.9	82	0.49	0.13	0.26	1.80	0.2
	46.8	42.8	2.3	1.9	120	0.37	0.26	0.13	1.35	0.4
	50.4	39.2	1.8	2.0	118	0.36	0.25	0.13	1.35	0.3
Total Settlement at CPT-3										**1.2**
CPT-4	20.5	68.0	1.5	2.5	126	0.42	0.27	0.64	1.20	0.2
	37.1	51.4	0.3	2.1	115	0.49	0.24	0.47	1.35	0.1
	41.5	47.0	5.0	1.7	128	0.47	0.30	0.57	1.20	0.7
	55.0	33.5	1.8	1.8	131	0.42	0.32	0.65	1.10	0.2
Total Settlement at CPT-4										**1.2**

We conclude several layers are potentially liquefiable during a major earthquake. The excavation for three basement levels will remove most of the layers; however, we estimate liquefaction-induced settlements up to 1 inch could occur beneath the building with differential settlements of about ½ inch in 30 feet. Where there will not be any basement excavations, we conclude about one to five inches of seismically induced-settlement could occur. This settlement is expected to be erratic.

The potentially liquefiable layers encountered beneath the planned basement foundation appear to be relatively thin, discontinuous, and are separated by layers of relatively plastic clay. Therefore, we judge that the potentially liquefiable material does not pose a hazard for loss of foundation support of a foundation subgrade between Elevations 42 feet and 40 feet.

6.2 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open slope cut, or in the direction of a regional slope or gradient. The potential for lateral spreading to occur at a site is typically evaluated using an empirical relationship developed by Youd, Hansen, and Bartlett (2002). This relationship incorporates the thickness, fines content, mean grain-size diameter, and relative density of the liquefiable layer, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions (such as a free face or edge of shoreline), to estimate the horizontal ground movement.

The site and surrounding area are generally flat. The nearest free face (Guadalupe River) is over 1,000 feet east of the site. We used the results of the laboratory tests performed on soil samples from the borings, the CPT data and the Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacements (Youd et al. 2001) to evaluate the potential for lateral spreading. These regression equations indicate that sandy soil layers with $(N_1)_{60}$ values greater than 15 blows per foot may be moderately susceptible to soil liquefaction, but are sufficiently dense to resist the potential for lateral spreading (Youd et al 2001). In addition the potentially liquefiable layers are generally discontinuous and the proposed basement levels should key the building below any zone where lateral spreading could occur.

During the 1906 earthquake, significant lateral spreading was observed around Coyote Creek which is about 1¼ miles to the northeast of the site; however, lateral spreading was not observed along the Guadalupe River (Lawson, 1908). Also, the site and surrounding area are generally flat. Considering these conditions, we judge the potential for lateral spreading is low.

6.3 Sand Boils

We estimated the potential for sand boils using the Ishihara (1985) and Youd and Garris (1995) method using the non-liquefiable soil cover thickness, liquefiable sand thickness, and maximum acceleration. Layers at the site that may potentially liquefy are thin and have sufficient non-liquefiable soil cover; therefore, we conclude that the potential for sand boils to manifest at the ground surface or below the three basement levels is low.

6.4 Seismic Densification

Seismic densification (also referred to as cyclic densification and differential compaction) can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. The borings and CPTs indicate that the materials above the water table are granular deposits. Therefore, where there will not be any basement excavations (i.e. surrounding sidewalks and improvements at-grade), we conclude about ½ inch of seismic densification may occur.

6.5 Fault Rupture

Historically, ground surface fault rupture closely follows the traces 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 rupture 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 low.

7.0 DISCUSSION AND CONCLUSIONS

We conclude the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the project plans and implemented during construction. An excavation up to 48 feet below existing grades will be required to achieve the proposed subgrade for the proposed foundation system. Temporary shoring will be required to brace the excavations. The primary geotechnical issues for this project include:

- selection of an appropriate foundation system to support the building loads and accommodate estimated static and seismic settlements

- dewatering and support for proposed excavations and adjacent structures during construction
- providing a stable subgrade and adequate working surface at the base of the excavation.

Our conclusions regarding these and other geotechnical issues are discussed in the remainder of this section.

7.1 Foundations and Settlement

We understand the design team is proposing a mat foundation. We conclude that a mat foundation is feasible for the building. Mat foundations are discussed in the following subsections.

7.1.1 Mat Foundation

Because the proposed excavation for the basement levels will result in a net decrease in overburden pressure under the basement footprint, we do not anticipate excessive settlements in the clay layers below. In addition, the basements will extend below the groundwater level and foundation system/floor will need to resist hydrostatic uplift pressures and span between columns. Considering these issues, we conclude the building can be supported on a mat foundation.

The mat should also be designed to tolerate liquefaction-induced settlements and static settlement as discussed in Sections 6.1 and 7.1.2, respectively. Because the basement walls and mat will extend below the groundwater level, they should be waterproofed.

7.1.2 Foundation Settlement Characteristics

The building will settle moderately due to recompression of the soil under the building loads. We anticipate the bottom of the excavation will predominantly consist of dense sands and gravels. We estimate total static settlement under an estimated allowable building load of 4,000 psf of the basement portion of the building will be about 1 inch. Differential settlement will depend on the rigidity of the mat. The majority of the anticipated settlement should occur during construction.

As discussed in Sections 1.0 and 6.1, after the 46-foot and 48-foot deep excavation, an additional 1 inch of seismically-induced total settlement should also be expected within the basement footprint, with a differential settlement of about ½ inch between columns. These estimated differential settlements do not take into account the rigidity of the mat, and the actual differential should be less.

These settlements estimates are preliminary. We should the mat pressures once they are developed by the structural engineer and we will revise the modulus of subgrade reaction as necessary.

7.2 Groundwater

During our investigation, groundwater levels were encountered between Elevations 67.6 to 59.5 feet, however the historic high groundwater is about 15 feet bgs corresponding to Elevation 75 feet. Because ground water levels may fluctuate seasonally, we recommend using a design groundwater of Elevation 75 feet.

7.3 Dewatering, Shoring, Underpinning and Excavation

Our conclusions regarding dewatering, shoring, underpinning, and excavation are discussed in the following subsections.

7.3.1 Dewatering

To construct the basement of the building, the groundwater will need to be temporarily lowered to a depth of at least three feet below the bottom of the planned excavation. Several sandy layers are present within the proposed depth of excavation. Some of these sandy layers may act as a conduit for water to flow into the excavation from the sides.

Based on experience, we consider dewatering of the excavation to be of extreme importance to the performance of the shoring and maintaining a stable subgrade for construction of the foundation. A well-designed, installed and operated dewatering system is therefore essential. It may be necessary to dewater the sand layers near the bottom of the proposed excavation to relieve the hydrostatic pressure on the overlying clay layer and reduce the possibility of blowing out the bottom of the excavation. Wells extending into this sand layer may be necessary to maintain stability. However, special care should be taken to minimize the removal of fines from the granular layers. This could be done by placing a tap on each well and monitoring the amount of fines removed. The dewatering should be maintained until sufficient weight and/or tiedown capacity is available to resist the hydrostatic uplift forces on the bottom of the foundation.

Variables that influence the performance of the dewatering system and the quantity of water produced include the number of wells, the 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 successfully dewater the site. The site dewatering should be designed and implemented by an experienced dewatering contractor. However, we should review the dewatering system proposed by the contractor prior to installation. Because of the size of the site, a system of perimeter wells may not sufficiently dewater it. Interior wells may also be needed to adequately dewater the site and minimize disturbance to the subgrade.

A working pad, as discussed in Sections 7.4 and 8.1, can also be used as a temporary drainage blanket in addition to wells. Perforated pipes may be placed in the gravel to collect water and conduct it to a sump. The sump and collector pipes should be decommissioned once they are no longer needed.

Dewatering the site should remain as localized as possible. Widespread dewatering could result in subsidence of the area around the site due to increases in effective stress in the soil. Nearby streets and other improvements should be monitored for vertical movement and groundwater levels outside the excavation should be 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. A recharge program should be submitted as part of the dewatering plan.

If the excavation is supported by a cutoff wall shoring system (such as a deep soil mix wall), we anticipate only dewatering within the site will be required, and there should be no significant lowering of the groundwater level outside of the excavation. In this case we would not anticipate significant settlement of the surrounding improvements associated with the required dewatering.

7.3.2 Shoring Considerations

An excavation up to approximately 48 feet deep will be required to accommodate the below grade levels. The adjacent sidewalks, streets and utilities along the sides of the site should be retained by temporary shoring until the permanent basement walls have been constructed.

There are several key considerations in selecting a suitable shoring system. Those we consider to be of primary concern are:

- control of movements
- constructability
- dewatering
- cost

There are several possible methods of providing lateral support for the excavation, including:

- Soldier Pile and Lagging: A soldier-pile-and-lagging system consists of concrete encased steel H-beams placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds.
- Soldier Pile Tremie Concrete (SPTC) walls: A SPTC wall consists of soldier piles installed in a slurry trench and the tremie concrete is placed to form the concrete wall. The steel wide-flange piles form the primary support system for the wall and the concrete is designed to act as "lagging" spanning between the structural steel members.
- Soil-Cement-Mixed walls. Soil-cement-mixed walls are installed by advancing hollow-stem augers and pumping cement slurry through the tips of the augers during auger penetration. In one type of soil-cement-mixed walls, the walls are constructed by excavating grooves with a moving chain-saw cutter. The soil is mixed with the cement slurry in situ, forming continuous overlapping soil-cement columns or continuous walls. Steel beams are placed in the soil-cement columns or walls at pre-determined spacing to provide rigidity. Soil-cement walls are considered temporary; permanent walls are usually built in front of the walls.

All systems would require tiebacks or internal bracing for lateral support.

A soldier pile and lagging retaining system is a more flexible system and is not as impervious as either an SPTC or soil-cement-mixed wall. The latter two types of walls would be relatively rigid and could significantly limit lateral deflections and ground subsidence related to the shoring; therefore, we conclude soldier-pile and lagging is unsuitable for this site.

We judge a soldier pile tremie concrete (SPTC) walls or soil-cement-mixed walls are the most suitable for the soil conditions encountered at this site. The disadvantages of these systems are cost and space requirements; they will require two to three feet around the perimeter of the site. Also existing basement walls and footings that interfere with the shoring system would need to be removed prior to installing the shoring. A combination of these systems could be used depending on the performance desired along the various excavation faces. Where movements could be detrimental to adjacent existing improvements or it is not practical to install underpinning the stiffer shoring systems could be used.

The shoring system selected should be designed by a civil engineer knowledgeable in the specific type of construction.

The adjacent property owners should be notified of the planned excavation and consulted regarding any special requirements they may have for construction. It may be difficult to obtain permission to install tiebacks on their property.

7.3.3 Underpinning

Where the proposed excavation extends deeper than the foundations of adjacent buildings (existing Tech Museum and Montgomery Theatre) or where adjacent foundations are above an imaginary 1:1 (horizontal to vertical) line extending up from the base of the excavation, underpinning should be provided to support the adjacent building loads or the shoring should be deepened to support the surcharges due to the foundations. Underpinning could consist of steel piles installed in slant-drilled shafts (slant piles) or intermittent hand-excavated piers that extend at least two feet below the planned bottom of excavation. The excavation face between the underpinning piles/piers should be retained using lagging, provided the existing footing can span between piers. The underpinning piles/piers should be designed to resist vertical building loads, vertical tieback loads (if tiebacks are used), and lateral earth pressures. Depending on the selected dewatering system, underpinning installation may require local dewatering. Hand excavated underpinning piers are usually about 30 by 48 inches in plan and are reinforced with steel and filled with concrete; slant piles are generally 30 to 48 inches in diameter. The piers/piles should be pre-loaded by jacking against the foundation, and the top of the pier/pile dry-packed to fit tightly with the base of the underpinned foundation. Underpinning piers should act in end bearing in the bearing strata below the depth of the proposed excavation, while slant piles gain their capacity in friction along the sides of the shaft.

7.3.4 Excavation and Monitoring

The site is occupied by an existing building, which will be demolished, and surrounding hardscape and asphalt pavement, which will be excavated. The soil at the site consists mainly of sand and clay that can be excavated with conventional earth-moving equipment such as loaders and backhoes. Brick, building rubble, and possibly concrete slabs, foundations, and walls of previous structures may be encountered in the fill. Handling and disposal of the fill material should be performed in accordance with a mitigation plan that includes health and safety criteria.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle. The magnitude of shoring movements and resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in installing the shoring. Typical maximum movement for a properly designed and constructed shoring system should be within about one inch. Considering the size and depth of the excavation and the presence of adjacent improvements, a monitoring program should be established to evaluate the effects of the construction on the surrounding area.

7.4 Mat Subgrade

The soil exposed at the bottom of the proposed excavations should consist predominantly of dense sands and gravels with varying amounts of fines in the majority of the site. Because the bottom of the basement levels will be below the groundwater level, the soil at subgrade level will be near saturation even after dewatering. Additionally, the subgrade exposed at the foundation level will be susceptible to disturbance under construction equipment loads. To help protect the soil subgrade a working pad should be constructed. The pad should consist of open graded crushed rock that is placed on a geotextile fabric. This layer of crushed rock can also be used as part of the dewatering system. A 3- to 4-inch thick mud slab can be placed on the crushed rock and then the waterproofing can be installed and the mat constructed. The need for the working pad and the thickness of rock section should be evaluated when the bottom of the excavation is reached; however, for budgeting purposes an allowance for the working pad should be included.

If any potentially liquefiable material is exposed at the mat subgrade, it should be over excavated and replaced with either lean concrete or engineered fill.

7.5 Exterior Slabs and Utilities

Exterior slabs, driveways, utilities, and utility connections at the building interface should be designed to accommodate potential differential settlement of up to five inches where the improvements settle relative to the buildings as a result of liquefaction. Entrances should be designed to accommodate the differential settlement, and flexible connections should be used where utilities enter the buildings.

7.6 Corrosion Potential

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed to evaluate the corrosivity of the near surface soil.

CERCO Analytical performed tests on soil samples to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Table 6 and Appendix D.

TABLE 6
Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfates (mg/kg)	Resistivity (ohms-cm)	Redox (mV)	Chlorides (mg/kg)
B-1	18	8.08	440	890	430	N.D
Composite of B-2 and B-3	6	8.27	70	3,500	450	N.D

N.D. = None Detected

Based upon resistivity measurements, the soil samples tested are classified as "moderately corrosive" to "corrosive" to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. The chemical analysis indicates reinforced concrete and cement mortar coated steel, will be affected by the corrosivity of the soil. To protect reinforcing steel from corrosion, adequate coverage should be provided as required by the building code. Corrosivity test results are presented in Appendix D.

8.0 RECOMMENDATIONS

Recommendations for site preparation, foundation support, slabs-on-grade, shoring design, below-grade walls, seismic design and other issues are presented in the following sections of this report.

8.1 Site Preparation and Grading

Existing pavements, old building foundations, abandoned utilities and other obstructions should be removed from areas to receive improvements. We anticipate the excavation for this project can be made using conventional earth-moving equipment except where old foundations and other buried obstructions are encountered. These may require hoe rams or jackhammers to remove. Any portions of existing buried foundations or basement walls that could interfere with the proposed improvements should be broken off and removed.

Where utilities to be removed extend off site, they should be capped or plugged with grout at the property line. It may be feasible to abandon utilities in-place, outside the proposed building footprint provided they will not interfere with future utilities or building foundations. If utilities are abandoned in-place, they should be completely filled with flowable cement grout over their entire length. Existing utility lines, where encountered, should be addressed on a case-by-case basis.

8.1.1 Mat Subgrade Preparation

Because the excavation for the basement levels will extend below the groundwater level, the soil at subgrade level will be near saturation even after dewatering. To protect the subgrade, we recommend heavy construction equipment not be allowed within three feet of subgrade elevation and that final excavation be made with excavators or backhoes with smooth buckets. Without an extended period for drying, we judge the subgrade may not support even light equipment and foot traffic without experiencing excessive disturbance.

To help protect the subgrade we recommend overexcavating the site and constructing a working pad on which to construct the foundation. We anticipate an overexcavation of about 12 inches should suffice if used in conjunction with a woven reinforcing fabric (geotextile), such as Mirafi 600x or equivalent. After placing the reinforcing fabric on the exposed subgrade, the overexcavation should be backfilled with clean one-inch minus crushed rock or similar material.

Because the proposed basement foundation will be below the groundwater level, waterproofing the base of the foundation is recommended. As discussed in Section 7.2, we recommend a design groundwater elevation of 75 feet be used. The waterproofing should be placed directly on the crushed rock (if allowed by the manufacturer) or on a mud slab (thin layer of lean concrete) and be covered by a mud slab. The mud slab covering should reduce the potential for damage to the waterproofing and provide a firm, smooth surface on which to place the reinforcing steel for the mat. We recommend the waterproofing be placed in accordance with the manufacturer's specifications. If they differ from our recommendations, the manufacturer's specification should be followed to preserve their warranty.

As discussed in Section 7.4, depending on the amount of water at the subgrade elevation, it may be desirable to use the crushed rock as a temporary drainage blanket. To drain the crushed rock, four-inch diameter perforated PVC pipe should be placed near the bottom of the rock, spaced every 30 feet, to direct water trapped in the rock to a sump. The sump should be properly abandoned before the completion of construction.

The soil subgrade at the base of the excavation should be free of standing water, debris, and disturbed materials prior to placing the reinforcing fabric and crushed rock. If loose or soft disturbed material is observed in the excavation, it should be overexcavated to firm, competent material and replaced with crushed rock or lean concrete. We should check the exposed subgrade after cleaning, but prior to placement of the working pad, mud slab or waterproofing.

8.1.2 At-Grade Improvements

Other areas that will receive improvements (e.g. sidewalks and exterior concrete flatwork) should be stripped of existing improvements. The surface exposed by stripping should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction[9]. If soft or loose soil is encountered, the unsuitable material should be removed and be replaced with suitable fill material that is properly compacted and moisture conditioned. The exposed ground surface should be kept moist during subgrade preparation.

New fill should be free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, have low corrosion potential[10] and be approved by the geotechnical engineer. All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The subgrade

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[9] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-07 laboratory compaction procedure.

[10] Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

surface should be rolled to a dense, non-yielding surface. If the compacted subgrade is disturbed during utility trench excavations, the subgrade should be re-rolled to provide a smooth, firm surface for concrete slab support.

We recommend new sidewalks and concrete flatwork be underlain by at least four inches of Class 2 aggregate base material (or the minimum thickness per City of San Jose Standards) that is compacted to at least 95 percent relative compaction.

8.2 Mat Foundation

On the basis of the results of our analyses, we conclude the proposed building with three basement levels may be supported on a mat foundation bottomed on dense sand and gravel.

To design the mat using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 45 kips per cubic foot (kcf) for the mat foundation. The modulus values are representative of the anticipated settlement under the building loads. After the mat analysis is completed, we should review the computed settlement and bearing pressure profiles to check that the moduli values are appropriate. The moduli are applicable for allowable dead plus live loads up to 4,000 psf, and for total loads (including wind and/or seismic) of 6,000 psf. In addition, the mat should be designed for an additional ½-inch of differential settlement between columns during a major earthquake.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. To calculate the passive resistance, we recommend using an equivalent fluid weight of 160 pounds per cubic foot (pcf). Frictional resistance should be computed using a base friction coefficient of 0.15 to 0.2; this friction value assumes a waterproofing membrane is placed below the mat. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

We recommend a design groundwater level of Elevation 75 feet be used to evaluate hydrostatic uplift pressure. Because the mat will be below the design groundwater level, we recommend that it be waterproofed. A waterproofing consultant should be retained to provide recommendations for the type of waterproofing and its installation.

The exposed subgrade for the mat/structural slab should be free of standing water, debris, and disturbed materials prior to constructing a working pad. We should check the subgrade after cleaning, but prior to placement of waterproofing mud slab, crushed rock or reinforcing steel to confirm bearing and that loose and disturbed material has been removed. If loose or disturbed material is observed at the bottom of the excavation, it should be overexcavated to firm, competent soil and be replaced with crushed rock or lean concrete.

8.3 Basement Wall Design

We recommend all basement walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. Because the site is in a seismically active area, the design should also be checked for seismic conditions. Under seismic loading conditions, there will be a seismic pressure increment that should be added to active earth pressures (Sitar et al., 2012). We used the procedures outlined in Sitar et al. (2012) and the peak ground acceleration based on the Design Earthquake (DE) ground motion level to compute the seismic pressure increment. Basement walls should be designed for the equivalent fluid weights presented in Table 7.

TABLE 7
Lateral Earth Pressures

	Static Conditions		**Seismic Conditions**
	Unrestrained Walls	**Restrained Walls**	**Total – Active Plus Seismic Pressure Increment**
Above the water table[1]	35 pcf	55 pcf	61 pcf
Below the water table	80 pcf	90 pcf	92 pcf

Note: 1. Design groundwater level is Elevation 75 feet, NGVD29 Datum.
2. pcf = pounds per cubic foot.

The walls should be designed for the more critical loading condition of static or seismic conditions. If surcharge loads occur above an imaginary 45-degree line projected up from the bottom of a below-grade wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the surcharge pressure on a case-by-case basis.

Where traffic will pass within 10 feet of basement 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.

If the basement walls are designed to resist lateral forces such as wind or earthquake loading they should be checked using passive pressures. To calculate the passive resistance against the below-grade walls, we recommend using a maximum uniform pressure of 1,300 psf. This value contains a factor of safety of 1.5.

The lateral earth values given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend to the design groundwater elevation (Elevation 75 feet). A four-inch diameter perforated collection pipes should be installed at the bottom of the drainage panel and discharge to a suitable outlet. The pipe should be wrapped in Mirafi 140NC fabric and surrounded on all sides by 4-inches of Class 2 permeable rock. We should check the manufacturer’s specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls and according to manufacturer’s specifications.

8.4 Excavation, Temporary Slopes and Shoring

We anticipate the proposed 46- to 48-foot excavation can be made using conventional earth moving equipment. Removal of existing on-site improvements, including the foundations of the existing building, may require equipment capable of breaking concrete. The excavation contractor should note that previous foundations, building debris, and other obstructions may be encountered during shoring installation and excavation. These obstructions may have to be partially removed before the shoring can be installed.

If temporary slopes are used they should not be steeper than 1.5:1 (horizontal to vertical) for slopes up to 15 feet in height. Slopes greater than 15 feet in height should be analyzed on a case by case basis. If there is insufficient space to slope the excavation, then shoring will be required can be used to retain the sides of the excavations.

8.4.1 Shoring

We estimate excavations for the proposed basement will extend approximately 46 to 48 feet below the existing ground surface. The selection, design, construction, and performance of the temporary shoring system should be the responsibility of the contractor.

As discussed, we conclude an impervious shoring wall system such as an SPTC wall or soil-cement mixed wall, or combination of these walls along with tiebacks or internal bracing can be used to retain the excavation. A SPTC wall or soil-cement mixed wall should be designed using the lateral earth pressures presented on Figure 6. If the shoring will be designed to provide lateral support of adjacent structures, then the pressures presented on Figure 6 will need to be revised to account for building surcharges.

If traffic is 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 materials will be within a distance equal to the shoring depth in feet. 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 of the temporary shoring and possible damage to adjacent improvements. Penetration of a tied-back shoring system should be sufficient to achieve lateral stability and resist the downward loading of the tiebacks.

8.4.2 Tiebacks

Temporary tiebacks may be used to restrain the shoring. The vertical load from the temporary tiebacks should be accounted for in the design of the vertical elements. Design criteria for tiebacks are presented on Figure 6.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks with bar and strand tendons should have a minimum unbonded length of 10 and 15 feet, respectively. The unbonded length should be created by placing an oversized rigid smooth plastic casing (i.e. PVC pipe) over the bars or strands; flexible plastic does not provide adequate bond-break for the unbonded zone. All tiebacks should have a minimum bonded length of 15 feet and be spaced at least six times the grouted diameter of the bonded zone or four feet, whichever is greater. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, post grouting, and workmanship. The use of solid-flight augers to install tiebacks in sand and the fill can result in loss of soil and settlement of structures or the ground surface located above the tiebacks. Therefore, solid flight augers or Titan type anchors should not be used for tieback installation. We recommend a smooth cased tieback installation method (such as a Klemm type rig) be used. For estimating purposes, we recommend using the allowable skin friction values for post-grouted tiebacks shown on Figure 6; these values include a factor of safety of at least 1.5. Higher allowable skin friction values may be used, if verified by load tests.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth and water pressures imposed on the temporary shoring systems. Determination of the tieback length should be based on the contractor's familiarity with their installation method. The computed bond length should be confirmed by a performance- and proof-testing program under our observation. Replacement tiebacks should be installed for tiebacks that fail the load tests.

8.4.3 Tieback Testing

Each tieback should be tested. 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. 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.

During testing 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.

8.4.3.1 Performance Tests

The performance tests 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 movement, and to check that the designed unbonded 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, 4, 5, 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.

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. Creep tests should be performed in accordance with the provisions of "Recommendations for Prestressed Rock and Soil Anchors" of Post-Tensioning Institute.

8.4.3.2 Proof Tests

A proof test is a simple test which is used to measure the total movement of the tiebacks 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.

8.4.3.3 Acceptance Criteria

The geotechnical engineer 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 unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with a creep rate that does not exceed 0.08 inch/log cycle of time, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

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

8.5 Underpinning

Where foundations for adjacent buildings are above an imaginary 1:1 line drawn up from the bottom of the proposed excavation, they should be underpinned, or the shoring should be designed to provide vertical and lateral support for adjacent structures. Although portions the Tech Museum Building to the south may be pile supported, underpinning piers/piles may be required to support the loads of the adjacent building.

If underpinning is required, we judge hand-excavated underpinning piers or slant piles will be acceptable methods to underpin adjacent structures. The underpinning piles/piers should be designed to resist the vertical building loads (surcharge pressures from adjacent buildings), vertical tieback loads (if tiebacks are used), and at-rest lateral earth pressure calculated using 55 pounds per cubic foot (pcf) above the groundwater table and 90 pcf below the groundwater table. Lateral pressures may be resisted by passive resistance against the embedded portion of the piers/piles. Passive resistance is also presented on Figure 6. This value includes a safety factor of about 1.5.

Underpinning pits should extend at least two feet below the bottom of the planned excavation and may be designed using an allowable bearing pressure of 4,000 psf for dead plus live loads, provided they are embedded in the dense sand/gravel layer.

To reduce movement and provide adequate foundation support during installation of the underpinning piers, adjacent piers should not be excavated concurrently. We recommend underpinning piers should be preloaded prior to dry packing. We should observe the bottoms of the underpinning pier excavations to check that the exposed soil can support the design bearing pressure.

If slant piles are used as underpinning, they should be designed by the underpinning contractor, and the design reviewed by Langan. The slant pile designer should evaluate the required penetration depth of the slant piles. The slant piles should have sufficient axial capacity to support the vertical load acting on the piles. To compute the axial capacity of the piles, we recommend using an allowable friction of 1,000 psf in the soil on the perimeter of the piles below the excavation level. To compute the allowable skin friction (against the back side of the slant piles) above the excavation level, we recommend an allowable friction coefficient of 0.3 times the horizontal component of the tieback force. End bearing of the slant piles should be neglected. The slant pile designer should also check that the slant piles are sufficiently stiff to avoid twisting from the test loads of the tieback. If the slant piles twist it may be necessary to stiffen them by welding beams or plates in the field or adding walers.

8.6 Dewatering

Prior to and during excavation, the groundwater should be drawn down so that the groundwater level on the inside of the excavation is at least three feet below the bottom of the 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 based on the design groundwater elevation. Elevator and sump pits can be locally dewatered. Adjacent improvements should be monitored for vertical movement caused by the dewatering.

The structural engineer should evaluate and provide recommendations when the dewatering system can be turned off. The number and depth of dewatering wells should be determined by an experienced specialty dewatering contractor. The volume of water discharged should be monitored and a record of the amount should be submitted to the owner.

8.7 Tiedown Anchors

Tiedown anchors may be used where the uplift pressure will exceed the anticipated building loads. Tiedown anchors typically consist of relatively small-diameter, drilled, concrete or grout filled shafts with steel bars or tendons embedded in the concrete or grout. The anchors develop their uplift resistance from friction between the sides of the shaft and the surrounding soil.

Tiedown anchors should be spaced at least six shaft diameters apart or 4 feet, whichever is greater. The ultimate bond strength between the anchor and soil will depend on the installation procedure. The actual bond strength should be estimated by the designer. For planning purposes, however, we recommend using an ultimate skin friction of 2,000 psf for post-grouted tiedowns. Higher values may be obtained depending upon the techniques employed by the contractor and the results of pullout tests. A safety factor of 1.5 and 2.0 should be used for temporary loads such as seismic or wind and permanent loads such as hydrostatic, respectively.

Special attention should be given to waterproofing the connections between the tiedown anchors and the foundation. Because tiedowns will be permanent, we recommend they be double corrosion protected. The tiedowns will be installed below the water table; therefore, the contractor should use an installation method that prevents the holes from caving. If water is present in the shaft, concrete should be placed using a tremie system. High strength bars or strands may be used as tensile reinforcement in the anchors. A minimum stressing length (free length) of 10 and 15 feet should be provided for bar and strand tendons, respectively. If strands are used, a significant lock-off load will be required (roughly 50 to 75 percent of the design load), to limit deformation of the tiedown under the hydrostatic loading. If a steel bar is used, a lower lock-off load will be required.

The bond length should be at least 15 feet. The design capacity of the tiedowns should be confirmed by a performance- and proof-test program conducted under our observation. We recommend the first two production tiedowns and two percent of the remaining tiedowns be performance-tested to 1.5 times the design load. All other tiedowns should be proof-tested to 2.0 times the design load. The test procedure and acceptance criteria described in Section 8.4.3 for tieback testing should also be used for tiedowns. After testing, all anchors should be loaded and locked off to a portion of their design load as determined by the structural engineer and indicated on the structural drawings and/or in the specifications.

8.8 Seismic Design

For seismic design in accordance with the provisions of 2013 California Building Code (CBC) we recommend the following:

- Site Class D
- Risk Targeted Maximum Considered Earthquake (MCE_R) S_S and S_1 of 1.500g and 0.600g, respectively.
- Site Coefficients F_a and F_v of 1.0 and 1.5, respectively
- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.500g and 0.900g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.000g and 0.600g, respectively.
- PGA_M of 0.5g

8.9 Utilities

Utilities should be designed to accommodate the predicted settlement. Flexible connections which allow for differential movement (static and earthquake-induced settlement) should be used as needed.

The corrosivity results provided in Appendix D of this report should be reviewed and corrosion protection measures used if needed. We recommend a corrosion engineer be retained when detailed corrosion protection recommendations are needed.

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

Backfill for utility trenches should be compacted according to the recommendations presented for the general site fill. Jetting of trench backfill is not permitted. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in lifts of eight inches or less, moisture-conditioned, and compacted to at least 90 percent relative compaction. If sand or gravel with less than 10 percent fines (particles passing the No. 200 sieve) is used, it should be compacted to 95 percent relative compaction.

Special care should be taken in controlling utility backfilling in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to exterior improvements.

8.10 Construction Monitoring

The conditions of existing buildings and other improvements within 100 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction.

To monitor ground movements, groundwater levels, and shoring movements, we recommend installing survey points on the adjacent buildings and streets that are within 100 feet of the site. In addition, survey points should be installed at the tops of the shoring walls at 20-foot-spacing.

The survey points should be read regularly and the results should be submitted to us in a timely manner for review. For estimating purposes, assume that the survey points will be read as follows:

- prior to any dewatering or shoring work at the site
- weekly once dewatering begins
- after installing soldier piles
- weekly during excavation work
- after the excavation reaches the planned excavation level
- every two weeks until the street-level floor slab is constructed

9.0 SERVICES DURING DESIGN AND CONSTRUCTION

We should review the final project plans and specifications to check that they are in general conformance with the intent of our recommendations. During construction, we should observe site preparation, grading, placement and compaction of fill, installation of building foundations, shoring and underpinning, and testing of tiebacks and tiedowns. These observations will allow us to compare the actual with the anticipated soil and bedrock conditions and to check that the contractors' work conforms to the geotechnical aspects of the plans and specifications.

10.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings and CPTs. Actual subsurface conditions could vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others. Any proposed changes in structures, depths of excavation, or their locations should be brought to Langan Treadwell Rollo’s attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan Treadwell Rollo's attention for evaluation, as they may affect our recommendations.

This report has been prepared to assist the Owner, architect, and structural engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities on adjacent properties which are beyond the limits of that which is the specific subject of this report.

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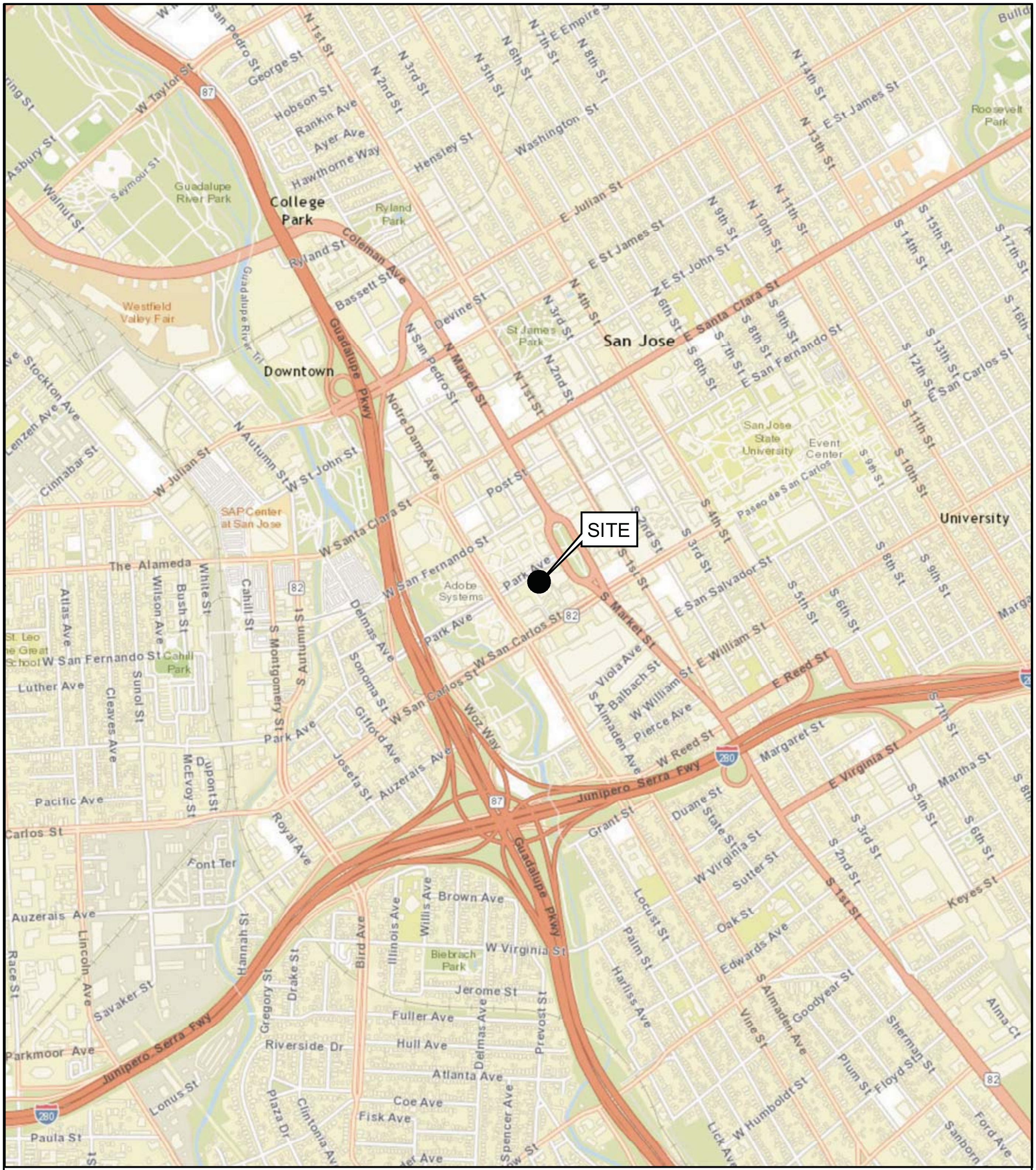
FIGURES



NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

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MUSEUM PLACE
San Jose, California

SITE LOCATION MAP

LANGAN TREADWELL ROLLO

Date 01/29/16 | Project No. 770627701 | Figure 1

Path: \\langan.com\data\SJO\data7\770627701\ArcGIS\ArcMap_Documents\Site Location Map.mxd User: cyoung

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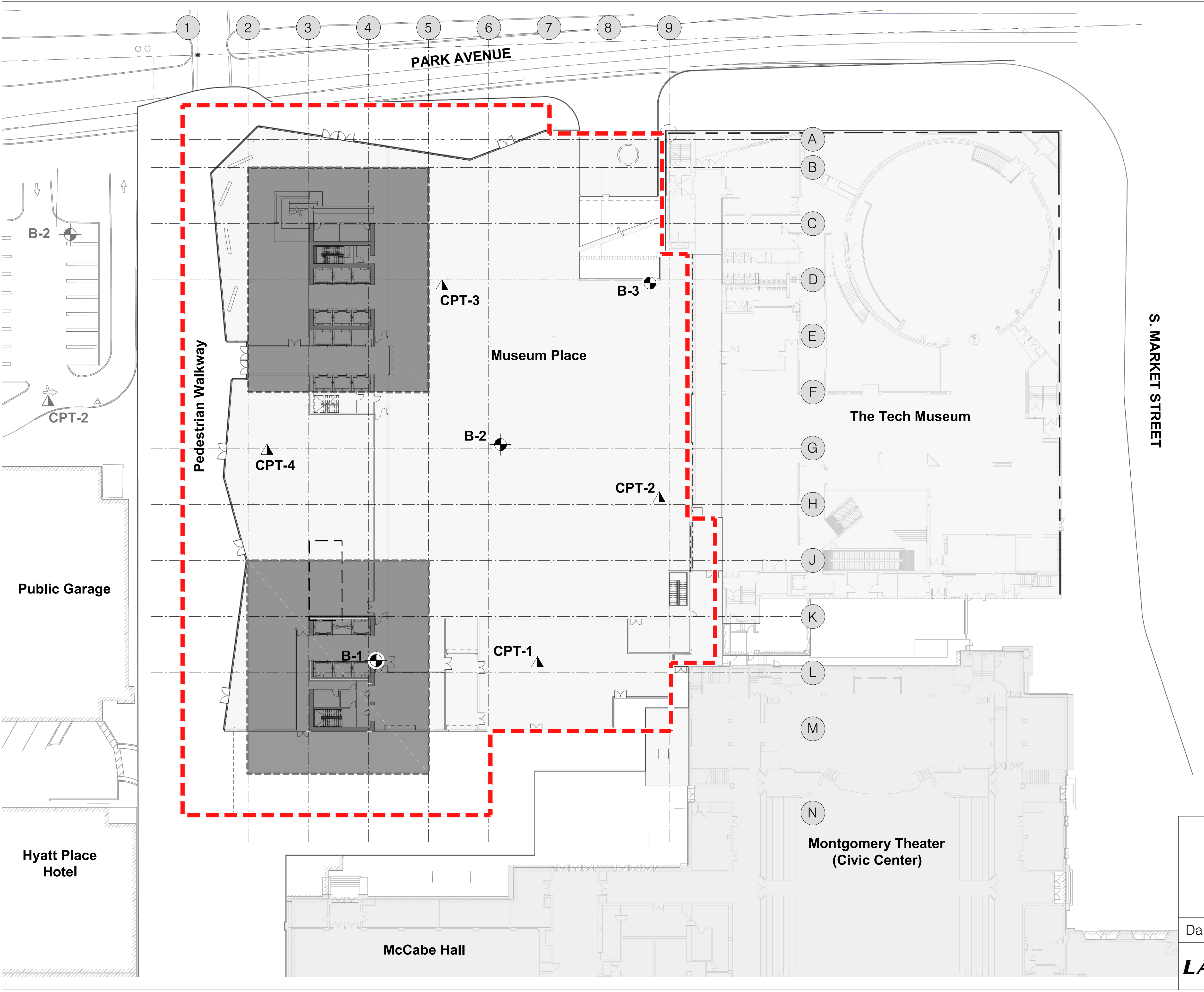


EXPLANATION

B-1 ⊕	Approximate location of boring by Langan Treadwell Rollo, January 2016
CPT-1 ▲	Approximate location of Cone Penetration Test (CPT) by Langan Treadwell Rollo, January 2016
B-2 ⊕	Approximate location of boring by Treadwell & Rollo, Inc., April 1998
CPT-2 ▲	Approximate location of Cone Penetration Test (CPT) by Treadwell & Rollo, Inc., April 1998
▨	Approximate area of proposed 8-foot thick mat foundation; remaining footprint a proposed 6-foot thick mat foundation
🟥	Proposed footprint of basement excavation

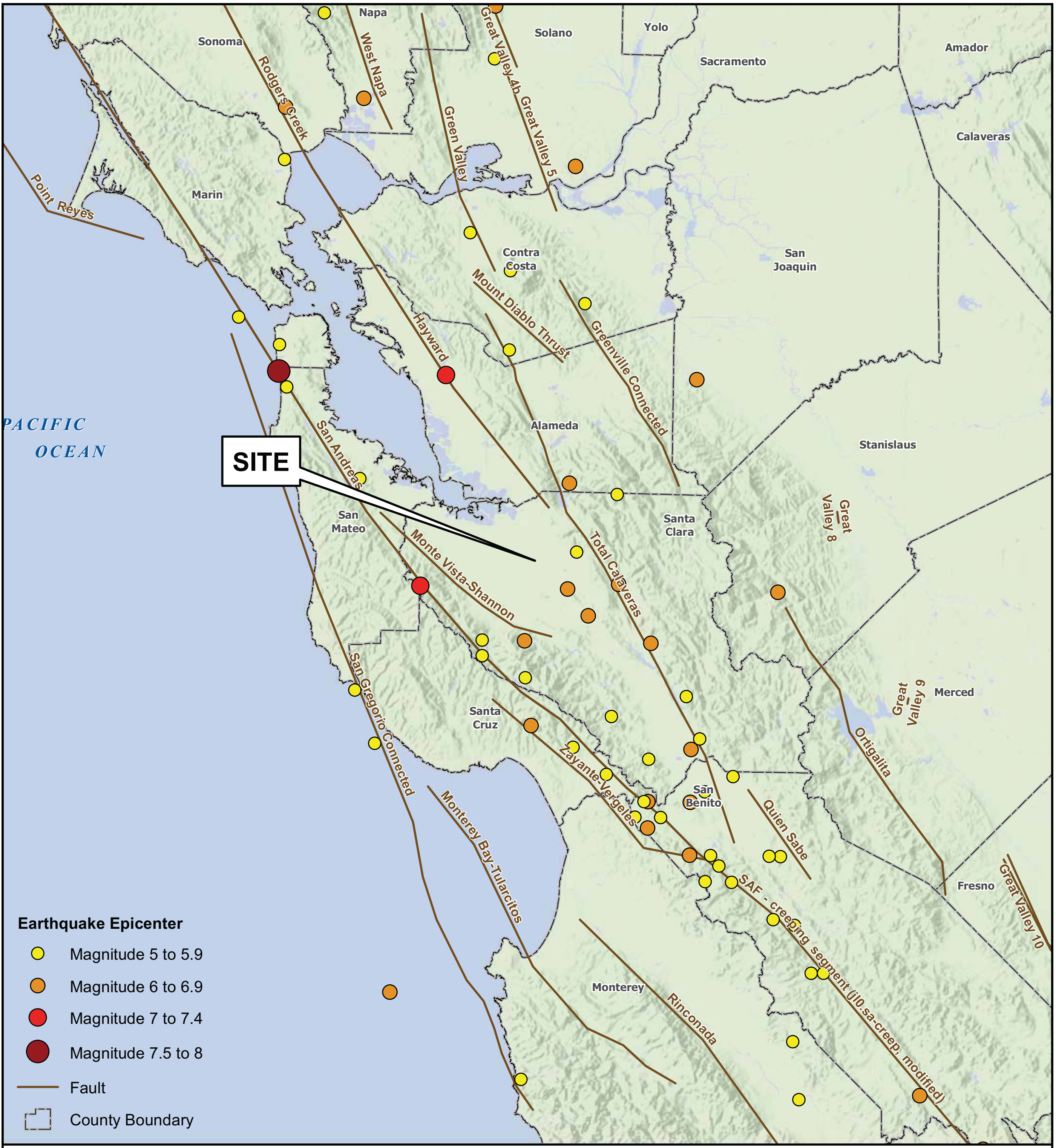
Reference: Base map from Architectural Site Plan, titled "Museum Place," Sheet 3.00," by Steinberg Architects, dated 06/30/16. Grid lines from Sheets S2.P3A, S2.P3B, S2.P3C and S2.P3D, Foundation Plan - Level P3, Basement, Museum Place, File Name"2016 0525_50% Structural.pdf", by MKA.

MUSEUM PLACE
San Jose, California

SITE PLAN

Date 07/19/16 | Project No. 770627701 | Figure 2

LANGAN TREADWELL ROLLO



\\langan.com\data\SJ\data7\770627701\ArcGIS\ArcMap_Documents\Fault Map.mxd User: agekas

Notes:

1. Quaternary fault data displayed are based on a generalized version of U.S Geological Survey (USGS) Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the USGS and is current through 08/26/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

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

Date 2/3/2016 | Project No. 770627701 | Figure 3

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.

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

MODIFIED MERCALLI INTENSITY SCALE

Date 02/03/16	Project No. 770627701	Figure 4

EXPLANATION

🟩 Zone of Liquefaction

Reference:
State of California "Seismic Hazard Zones"
San Jose West Quadrangle, Santa Clara County
Released on February 07, 2002

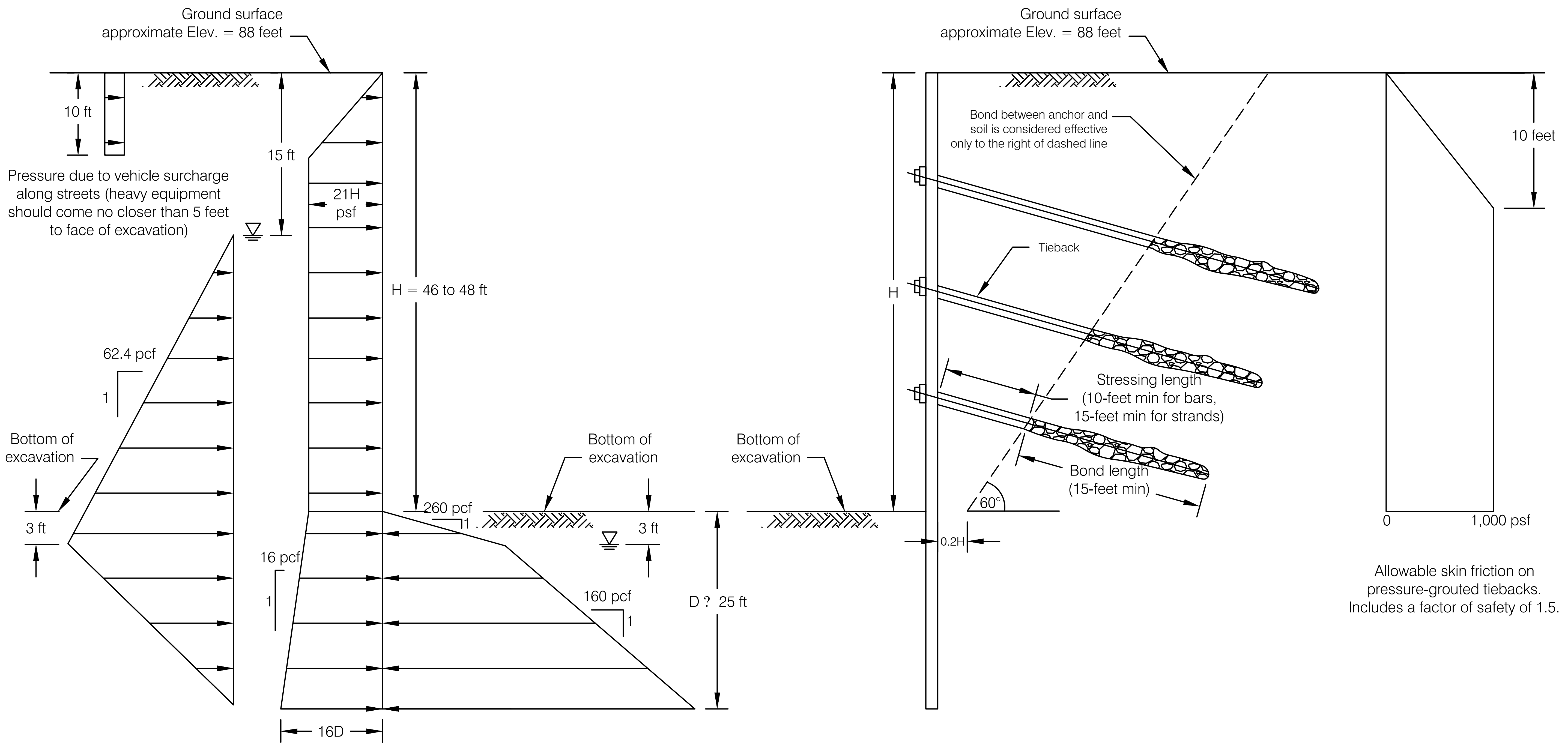
0 2,000 Feet
Approximate scale

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

LIQUEFACTION MAP
SAN JOSE WEST QUADRANGLE

Date 04/01/16	Project No. 770627701	Figure 5



NOT TO SCALE

Notes:
1. These shoring pressures area applicable to SPTC and soil-cement mixed walls with an excavation height of 46 to 48 feet.
2. Assumes groundwater within the site will be lowered to at least 3 feet below the excavation level.
3. Passive pressure values include a factor of safety of 1.5.
4. The tip elevation of the shoring wall should be determined by the shoring designer; however a minimum embedment of 25 feet (from the bottom of excavation) is recommended to tip the SPTC or soil-cement column into the clay beneath the dense sand/gravel layer.
5. Assumes shoring will be braced or tied back.
6. Where the shoring is adjacent to existing buildings, the shoring should be designed for the building surcharge load.

MUSEUM PLACE
San Jose, California

DESIGN PARAMETERS FOR
TEMPORARY IMPERVIOUS SHORING WALL

Date 07/20/16 | Project No. 770627701 | Figure 6

LANGAN TREADWELL ROLLO

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APPENDIX A

LOGS OF BORINGS

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-1

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2
Logged by: S. Magallon
Date started: 1/27/16
Date finished: 1/27/16
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 89.2 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
						8.5-inch thick unreinforced concrete slab with moisture barrier						
1						SAND with GRAVEL (SP) brown, moist, fine- to coarse-grained, fine subangular gravel FILL						
2					SP							
3						SILTY SAND (SM) brown to yellow-brown, loose, moist, fine-grained, with some clay						
4												
5												
6	S&H	▨	1 2 5	6						46.6	12.8	
7												
8												
9					SM							
10												
11	SPT	◩	2 4 5	13		medium dense						
12												
13												
14												
15						CLAY (CL) olive-brown with occasional orange oxidation staining, medium stiff, moist						
16	SPT	◩	1 3 3	8	CL							
17												
18						CLAY with SAND (CL) olive-brown, stiff, moist to wet, fine sand						
19	ST	◨		400 psi								
20												
21												
22					CL							
23												
24												
25												
26	S&H	▨	3 5 7	10		SANDY CLAY (CL) olive-brown to olive-gray, stiff, wet, fine sand, with silt TxUU Test, see Figure C-4	TxUU	3,100	1,160		26.8	96
27					CL							
28												
29					SP							
30												

LANGAN TREADWELL ROLLO

Project No.: 770627701
Figure: A-1a

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	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		6 4 5	7	CL	SAND (SP) olive-gray, loose, wet, trace silt CLAY with SAND (CL) olive-gray, medium stiff, wet, fine sand, with silt						
32	SPT		4 6 8	20								
33					SM	SILTY SAND (SM) olive-gray, medium dense, wet, fine-grained Non-Plastic, see Figure C-11				35.8	22.7	
34												
35						CLAY (CL) gray to dark gray, medium stiff, wet, trace fine sand						
36	S&H		9 7 4	9	CL	TxUU Test, see Figure C-5	TxUU	3,700	820		32.0	90
37												
38												
39						CLAY (CL) yellow-brown, medium stiff to stiff, wet, trace fine sand and silt						
40												
41	S&H		0 5 5	8	CL	Consolidation Test, see Figure C-1	PP		1,000		26.7	97
42												
43												
44						SANDY CLAY (CL) yellow-brown with orange mottling, medium stiff, wet, fine-grained sand, trace fine gravel						
45												
46	S&H		0 0 4	3	CL	TXUU Test, see Figure C-6	TxUU	4,300	720		21.3	106
47												
48												
49						SAND with SILT (SP-SM) olive-brown, medium dense, wet, medium-grained, trace fine subangular gravel						
50												
51	SPT		6 10 10	28	SP-SM					6.4	23.3	
52												
53												
54						SAND with GRAVEL (SP) brown, dense, wet, fine- to coarse-grained, fine to coarse subangular gravel up to 1-1/2 inches in diameter						
55												
56	SPT		14 14 15	41	SP							
57												
58												
59					GP	GRAVEL with SAND (GP) brown, very dense, wet, fine- to coarse subangular gravel up to 1-1/4 inches in diameter						
60												

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-1

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	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	◩	10 20 17	52		GRAVEL with SAND (GP) (continued) fine to coarse sand						
62												
63												
64												
65					GP							
66												
67												
68												
69												
70						CLAY with SAND (CL)						
71	S&H	■	3 8 14	18		gray, stiff, wet, fine sand, with abundant silt					25.5	102
72	SPT	◩	3 3 6	13		decrease in silt content						
73												
74												
75					CL							
76												
77												
78												
79						CLAY (CL)						
80						olive-gray with occasional light gray mottling, stiff, wet, trace fine sand, trace rootlets						
81	S&H	■	10 11 12	18		TXUU Test, see Figure C-7	TxUU	6,500	1,600		22.8	105
82												
83												
84												
85					CL							
86												
87												
88												
89	S&H	■	5 8 12	16		very stiff						
90												

Boring terminated at a depth of 90 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.4, respectively to account for sampler type and hammer energy.
[2] Elevations based on NGVD 29 datum.

LANGAN TREADWELL ROLLO

Project No.: 770627701

Figure: A-1c

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-2

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 1/25/16
Date finished: 1/25/16
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: S. Magallon

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 89.6 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						7-inch thick unreinforced concrete slab with moisture barrier						
1–5					SP	SAND with GRAVEL (SP) brown to yellow-brown, moist, fine- to coarse-grained, subrounded fine to coarse FILL						
5–8	S&H		7 10 10	16	SP	SAND with GRAVEL (SP) brown to yellow-brown, medium dense, moist, fine- to coarse-grained, subrounded fine to coarse gravel up to 1 1/4 inches in diameter, with trace silt						
9–13	S&H		8 15 18	26	GP	GRAVEL with SAND (GP) brown to yellow-brown, medium dense, moist, fine- to coarse-grained, subangular to subrounded gravel up to 1 1/4 inches in diameter, fine sand, trace silt						
14–16	SPT		3 3 3	8	CL	CLAY (CL) dark brown to black, medium stiff, moist						
17–18	ST			500 psi								
18–21					CL	SANDY CLAY (CL) yellow-brown, very stiff, moist to wet, fine sand	PP		4,000			
22						⊻ (01/26/16, 7:50 a.m.)						
23–25					SP-SM	SAND with SILT (SP-SM) yellow-brown with olive mottling, medium dense, wet, fine-grained						
26	S&H		10 9 12	17		SILTY SAND (SM) olive-gray, medium dense, wet, fine-grained, trace fine gravel						
27–30	SPT		5 8 8	22	SM					17.7	16.8	

LANGAN TREADWELL ROLLO

Project No.: 770627701
Figure: A-2a

TEST GEOTECH LOG 770627701.GPJ TR.GDT 5/2/16

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-2

PAGE 2 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
31	ST			200-500 psi	CL	SANDY CLAY (CL) dark blue-gray, wet, fine sand						
32						SAND with SILT (SP) olive-gray to gray, wet						
33					SP							
34						CLAY (CL) olive-gray to dark gray, stiff, wet						
35												
36	SPT		1 3 4	10	CL							
37	S&H		3 5 8	10								
38						SANDY CLAY (CL) yellow-brown with yellow mottling, stiff, wet, fine-grained sand, trace subrounded coarse gravel						
39					CL							
40						GRAVEL with SILT and SAND (GP-GM) yellow-brown, dense, wet, fine- to coarse-grained, subrounded to subangular gravel up to 1-inch in diameter, fine to coarse sand Sieve Analysis, see Figure C-10						
41	SPT		15 15 18	46						8.2	8.1	
42												
43												
44												
45					GP-GM							
46	SPT		10 11 11	31								
47												
48												
49												
50						SANDY CLAY with GRAVEL (CL) yellow-brown, very stiff, wet, fine to medium sand, fine subangular to subrounded gravel						
51	SPT		8 7 6	18								
52					CL							
53												
54						GRAVEL with SAND (GP) yellow-brown, very dense, wet, fine grained and subangular, fine to coarse sand, trace silt						
55												
56	SPT		10 23 23	60								
57					GP							
58												
59												
60												

TEST GEOTECH LOG 770627701.GPJ TR.GDT 5/2/16

LANGAN TREADWELL ROLLO

Project No.: 770627701

Figure: A-2b

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-2

PAGE 3 OF 4

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	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
						GRAVEL with SAND (GP) (continued)						
61												
62												
63												
64												
65					GP							
66	SPT	⊡	10 10 25	49		with cobbles in cuttings						
67												
68												
69												
70						SANDY CLAY (CL) yellow-brown, hard, wet, fine sand, trace coarse gravel up to 1-1/2 inches in diameter						
71	SPT	◩	10 18 20	53								
72					CL							
73												
74						SANDY CLAY (CL) olive-gray to gray, very stiff, wet, fine-grained sand, with silt, trace rootlets						
75												
76	S&H	■	9 10 11	17		Consolidation Test, see Figure C-2	PP		3,000		26.7	95
77												
78												
79												
80					CL							
81												
82												
83												
84												
85						stiff						
86	S&H	■	6 14 17	25		TxUU Test, see Figure C-8	TxUU	6,800	1,710		20.2	108
87					SC	CLAYEY SAND (SC) olive-gray to gray with occasional yellow-brown mottling, medium dense, wet, fine- to medium-grained				38.7	17.1	116
88												
89					CL	SANDY CLAY (CL) yellow-brown with orange mottling, very stiff, wet, trace fine and subangular gravel						
90												

LANGAN TREADWELL ROLLO

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Figure: A-2c

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-2

PAGE 4 OF 4

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	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
91	S&H	■	10 15 20	28	CL	SANDY CLAY (CL) (continued)				16.9	13.6	120
92					SC	CLAYEY SAND (SC) yellow-brown with orange mottling, medium dense, wet, fine- to medium-grained, trace coarse subrounded gravel up to 1-1/2 inches in diameter						
93												
94												
95												
96												
97					CL	SANDY CLAY (CL) olive-gray to gray, hard, wet, fine sand, trace fine subrounded gravel						
98												
99												
100												
101	SPT	◩	7 10 13	32								
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 101.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater ecountered at 22 feet below ground surface.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.4, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD 29 datum.

LANGAN TREADWELL ROLLO

Project No.: 770627701

Figure: A-2d

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2
Logged by: S. Magallon
Date started: 1/26/16
Date finished: 1/26/16
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 89.6 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
						7-inch thick unreinforced concrete slab with moisture barrier						
1					SP	SAND with GRAVEL (SP) brown to yellow-brown, moist FILL						
2												
3												
4					ML	SANDY SILT (ML) yellow-brown, stiff, moist, fine sand						
5												
6	S&H		4 5 7	10								
7												
8												
9												
10												
11	S&H		4 6 8	11	SM	SILTY SAND (SM) yellow-brown and olive with dark orange oxidation staining, loose to medium dense, moist, fine-grained						
12	SPT		3 3 2	7								
13					CL	CLAY (CL) yellow-brown and olive with occasional orange, stiff, moist TxUU Test, see Figure C-9						
14	ST			50 psi			TxUU	1,600	1,170		35.5	82
15												
16					SM	SILTY SAND (SM) yellow-brown and olive, moist, fine-grained, with some clay						
17												
18												
19					CL	CLAY with SAND (CL) olive with yellow-brown mottling, very stiff, moist, occasional light brown cemented sand clasts						
20												
21	S&H		5 11 11	18								
22												
23												
24					SM	SILTY SAND (SM) gray to olive-gray with yellow-brown mottling, medium dense, wet, fine-grained, occasional rootlets						
25												
26	S&H		7 11 18	15						23.7	15.5	107
27												
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 770627701
Figure: A-3a

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-3

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	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		4 7 4	9	SM	SILTY SAND (SM) (continued)						
32	SPT		0 2 4	8		SILTY CLAYEY SAND (SC-SM) gray, loose, wet, fine-grained sand LL = 24, PI = 4, see Figure C-11					19.9	
33												
34	ST			300 psi	SC-SM	increase in silt content						
35						Consolidation Test, see Figure C-3					30.8	91
36												
37						SANDY CLAY (CL) brown to olive-brown with yellow-brown mottling, very stiff, wet, trace fine subrounded gravel						
38												
39												
40					CL							
41	SPT		2 5 10	21								
42												
43						SAND with GRAVEL (SP) yellow-brown, dense, wet, fine-to coarse-grained, fine to coarse subangular gravel up to 1-1/4 inches in diameter						
44												
45												
46	SPT		7 12 17	41	SP							
47												
48												
49												
50						GRAVEL with SILT and SAND (GP-GM) gray-brown, dense, wet, fine to coarse grained, subangular, fine to coarse sand						
51	SPT		11 12 10	31						5.7	10.2	
52												
53												
54												
55					GP-GM							
56												
57												
58												
59						grades with increase in gravel content						
60												

LANGAN TREADWELL ROLLO

Project No.: 770627701

Figure: A-3b

PROJECT: **MUSEUM PLACE**
San Jose, California

Log of Boring B-3

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
61	SPT	◩	8 8 12	28		GRAVEL with SILT and SAND (GP-GM) (continued)						
62					CL	CLAY with SAND (CL) yellow-brown with orange mottling, very stiff, wet, fine-grained sand				72.0	28.4	
63						GRAVEL with CLAY and SAND (GP-GC) yellow-brown, very dense, wet, fine- to coarse-grained, subangular gravel up to 1-inch in diameter, fine to coarse sand						
64												
65												
66	SPT	◩	14 28 30	81	GP-GC							
67												
68												
69												
70						CLAY with SAND (CL) gray with yellow-brown and olive mottling, very stiff, wet, fine sand, with some silt						
71	S&H	⊙	11 18 16	26								
72												
73					CL							
74												
75												
76												
77						CLAY with SAND (CL) olive to olive-gray with yellow-brown mottling, very stiff, wet, fine sand						
78												
79												
80												
81	S&H	■	11 16 19	28								
82												
83					CL							
84												
85												
86												
87												
88												
89	SPT	◩	16 8 8	21		yellow-brown with dark orange oxidation staining						
90												

Boring terminated at a depth of 90 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.4, respectively to account for sampler type and hammer energy.
[2] Elevations based on NGVD 29 datum.

LANGAN TREADWELL ROLLO

Project No.: 770627701
Figure: A-3c

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 coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

SAMPLER TYPE

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

MUSEUM PLACE
San Jose, California

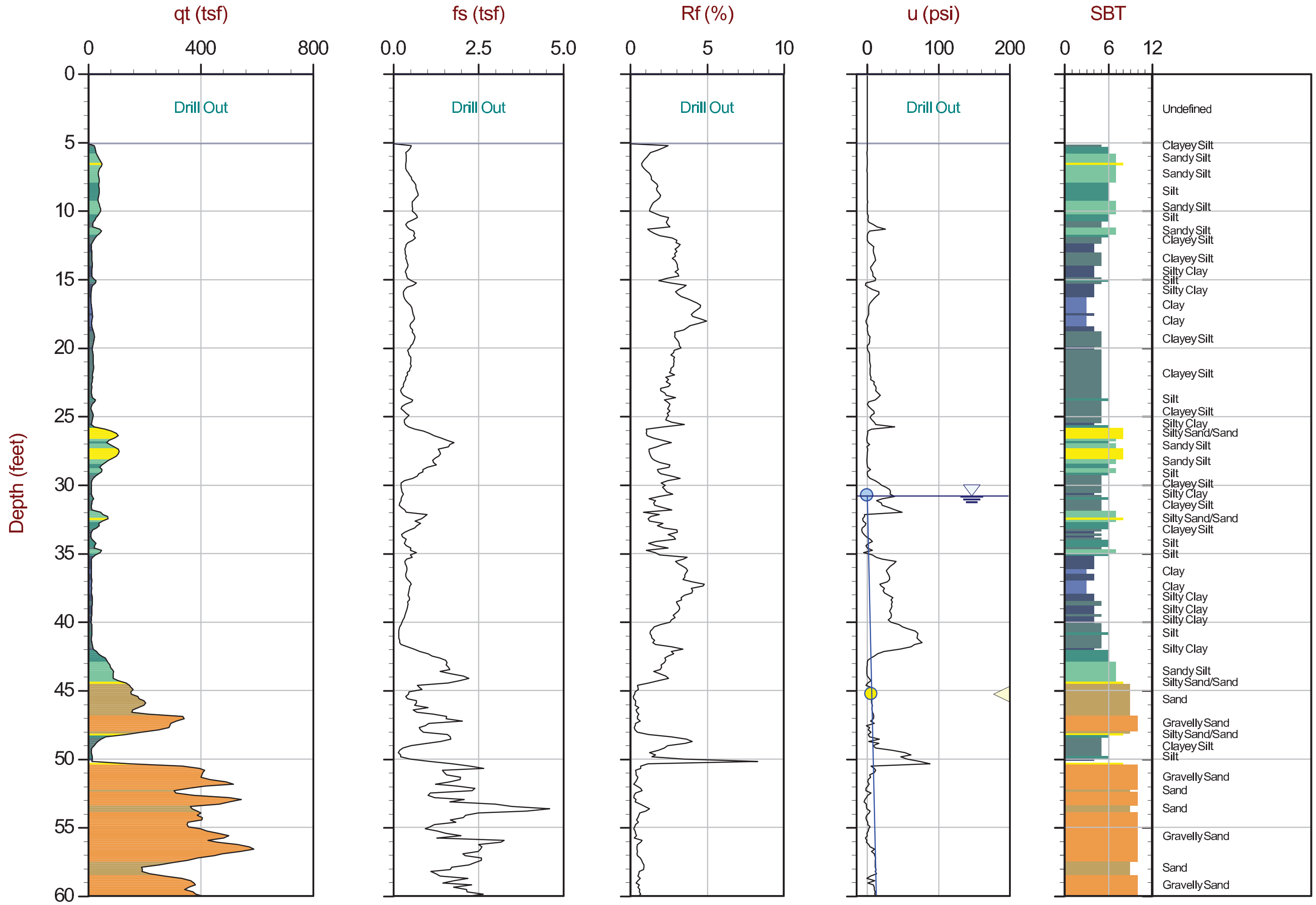
LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 03/08/16 | Project No. 770627701 | Figure A-4

APPENDIX B

CONE PENETROMETER TEST REPORT



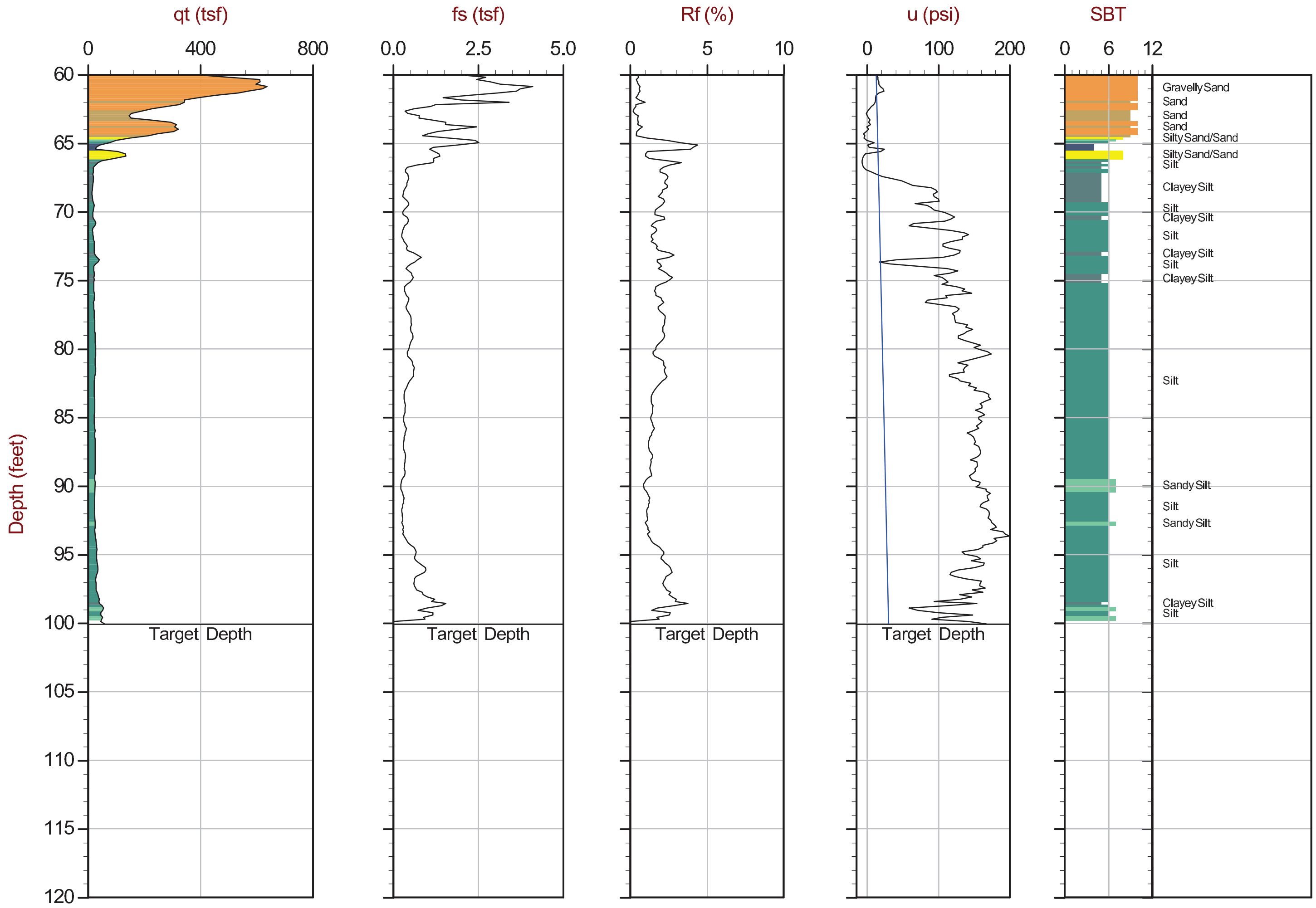
(continue on Figure B-1b)

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 03/08/16	Project No. 770627701	Figure B-1a

LANGAN TREADWELL ROLLO



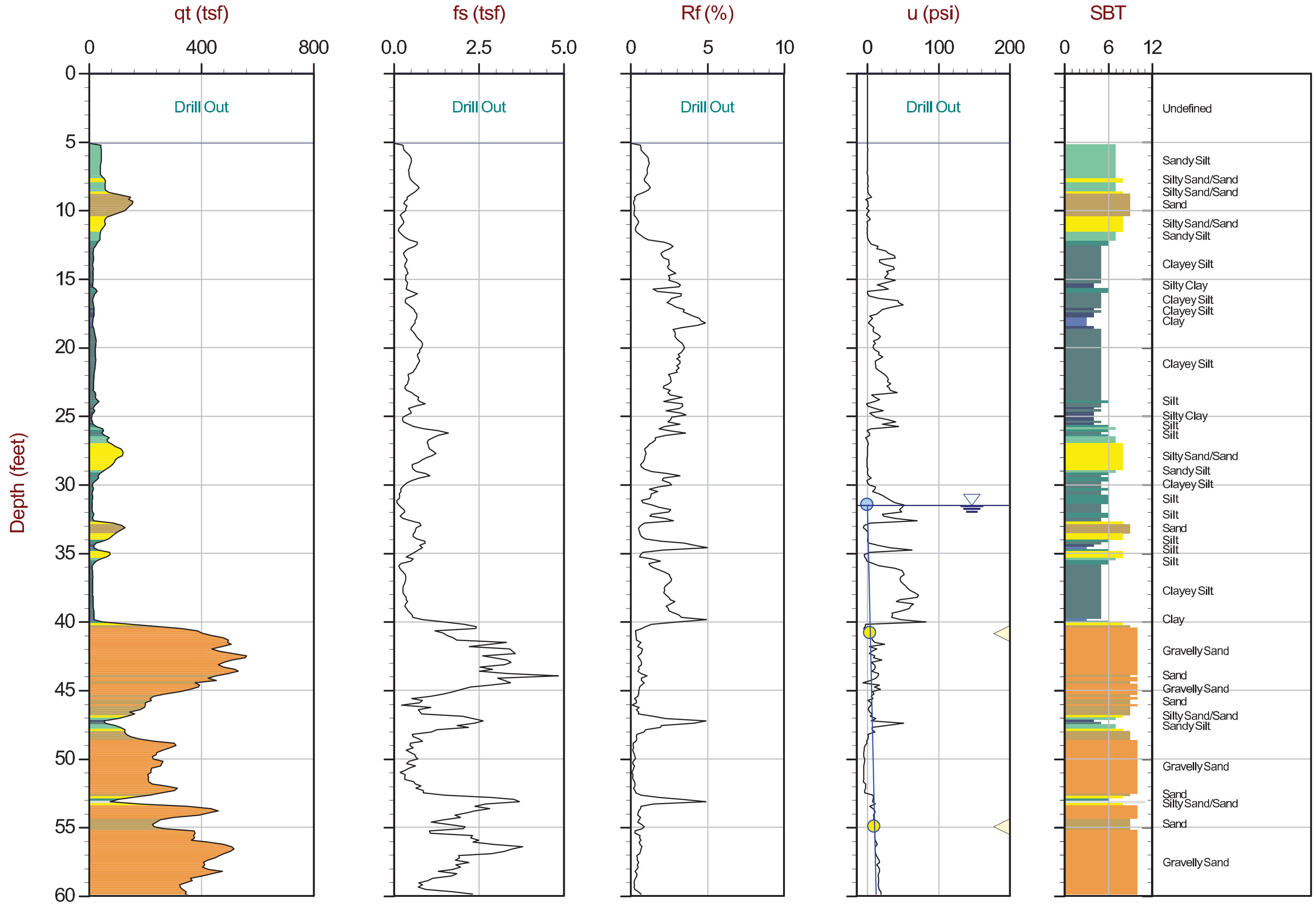
Terminated at 100.1 feet.
Groundwater encountered at 30.8 feet bgs (see PPDT on Figure B-6).
Date performed 01/27/16.
Ground surface elevation: 89.5 feet, NGVD 29 Datum.
Hand augered first 5 feet for utility clearance.

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-1 (CONTINUE)

Date 03/08/16	Project No. 770627701	Figure B-1b

LANGAN TREADWELL ROLLO



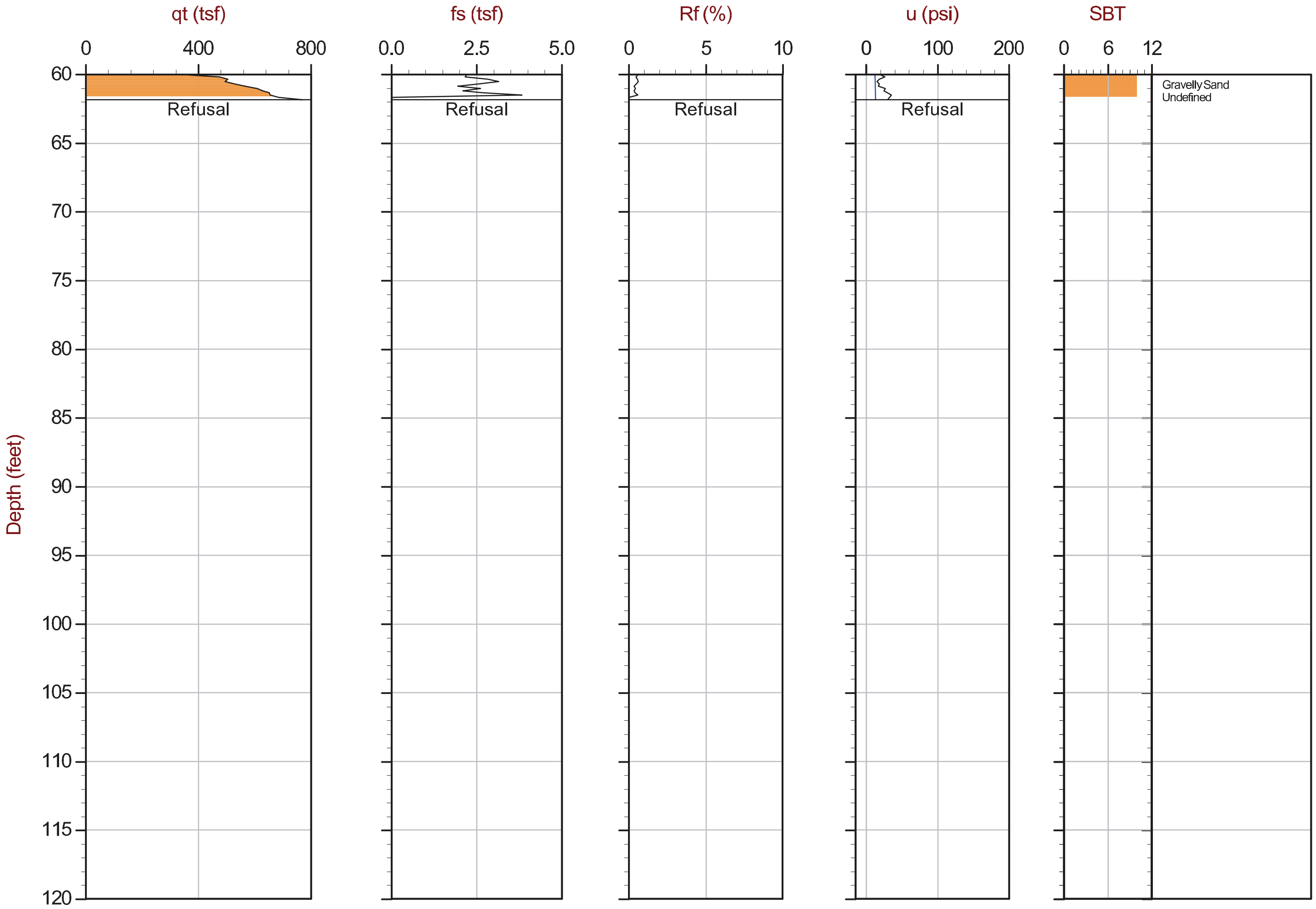
(continue on Figure B-2b)

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 03/08/16	Project No. 770627701	Figure B-2a

LANGAN TREADWELL ROLLO



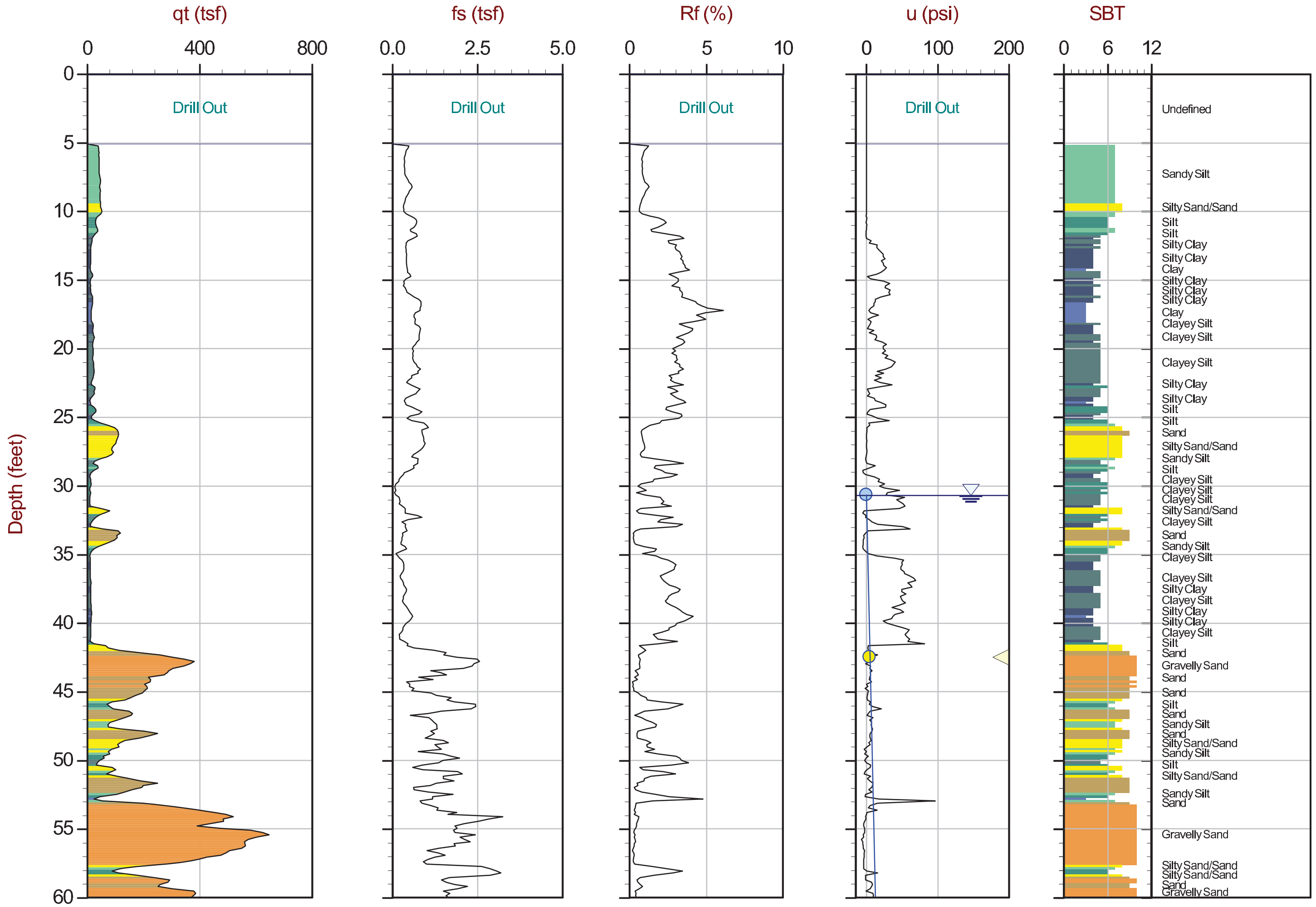
Terminated at 61.8 feet.
Groundwater encountered at 31.4 feet bgs (see PPDTs on Figures B-7 and B-8).
Date performed 01/27/16.
Ground surface elevation: 89.6 feet, NGVD 29 Datum.
Hand augered first 5 feet for utility clearance.

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-2 (CONTINUE)

Date 03/08/16	Project No. 770627701	Figure B-2b

LANGAN TREADWELL ROLLO



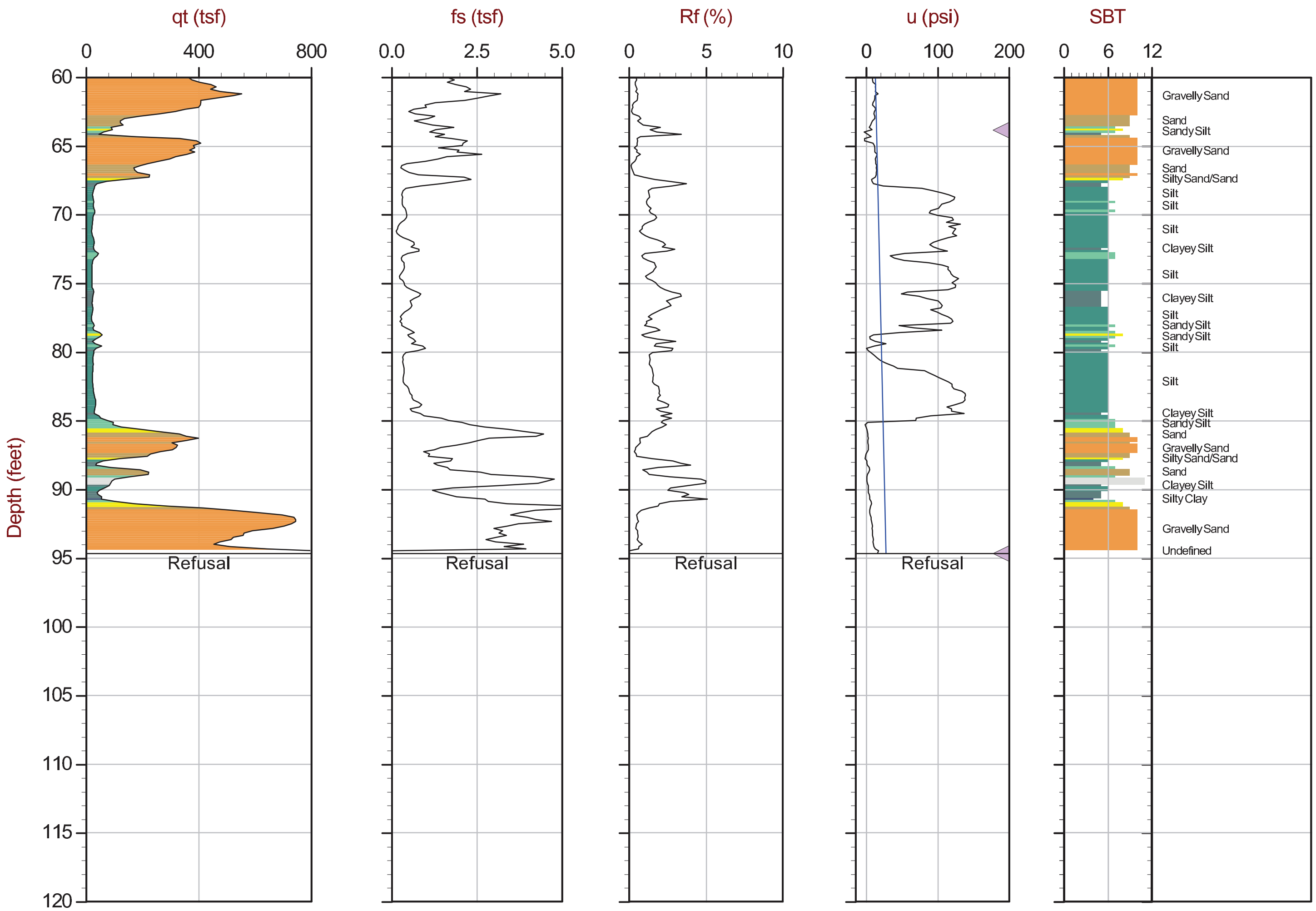
(continue on Figure B-3b)

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 03/08/16	Project No. 770627701	Figure B-3a

LANGAN TREADWELL ROLLO



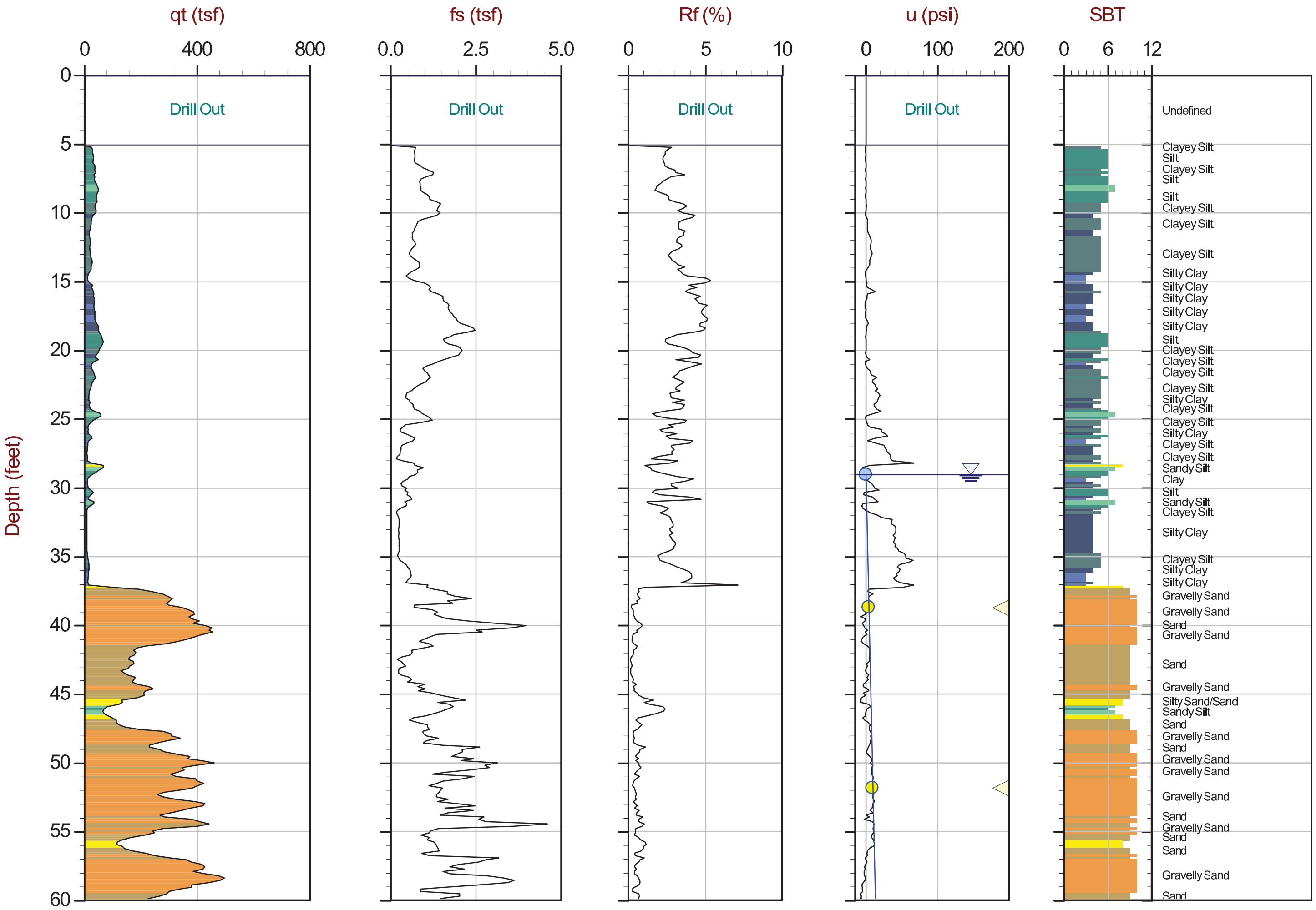
Terminated at 94.7 feet.
Groundwater encountered at 30.7 feet bgs (see PPDT on Figure B-9).
Date performed 01/27/16.
Ground surface elevation: 89.6 feet, NGVD 29 Datum.
Hand augered first 5 feet for utility clearance.

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-3 (CONTINUE)

Date 03/08/16	Project No. 770627701	Figure B-3b

LANGAN TREADWELL ROLLO



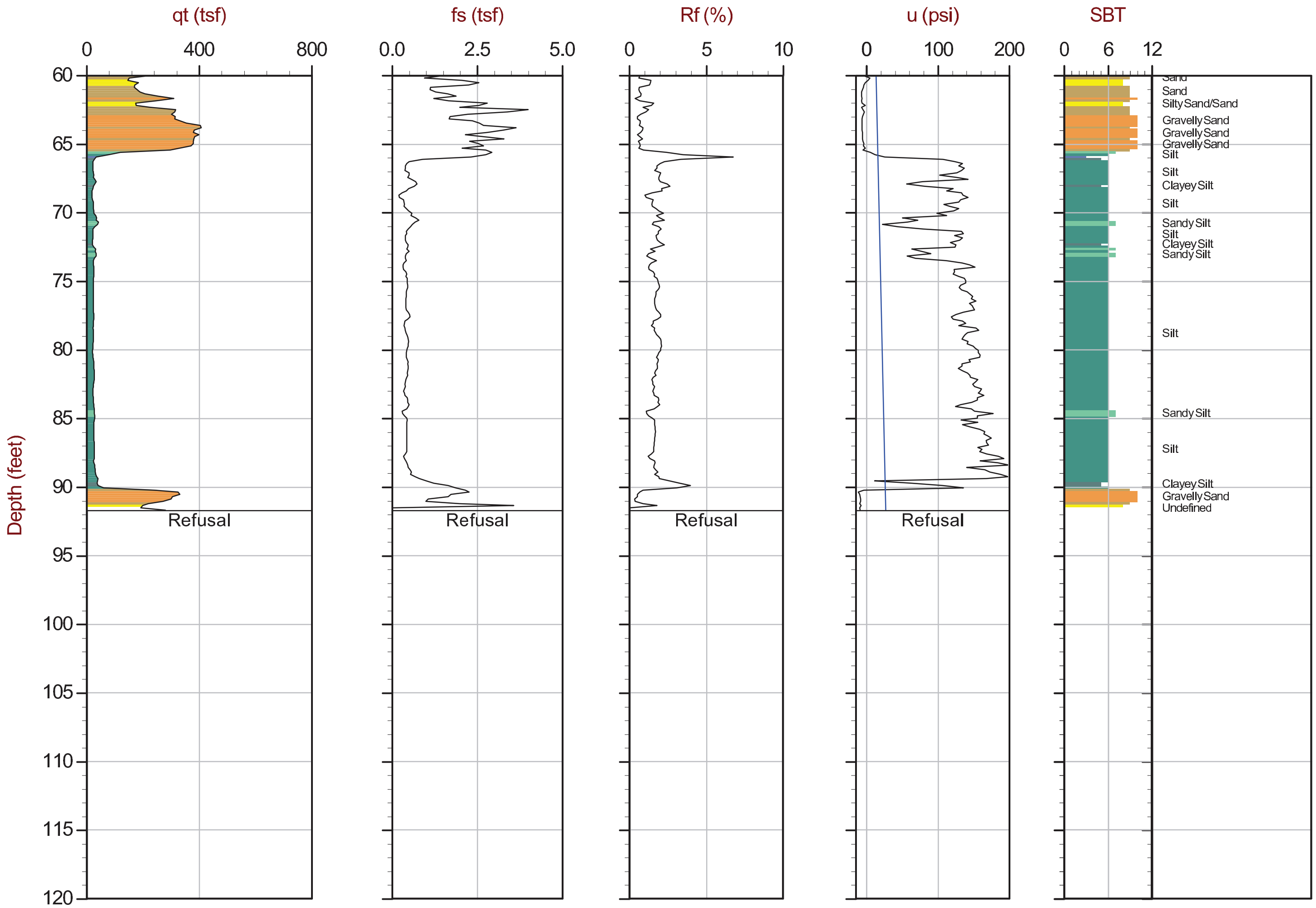
(continue on Figure B-4b)

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-4

Date 03/08/16	Project No. 770627701	Figure B-4a

LANGAN TREADWELL ROLLO



Terminated at 91.7 feet.

Groundwater encountered at 29 and 29.6 feet bgs (see PPDTs on Figures B-10 and B-11).

Date performed 01/27/16.

Ground surface elevation: 88.5 feet, NGVD 29 Datum.

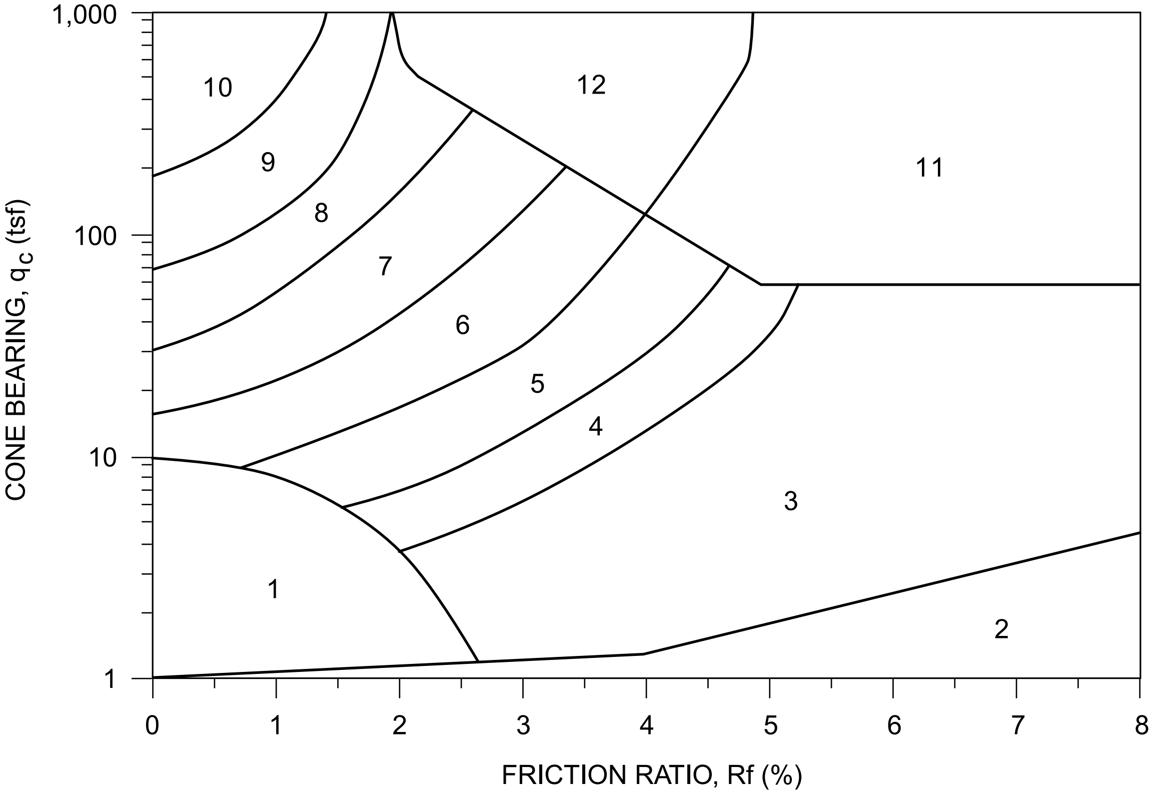
Hand augered first 5 feet for utility clearance.

MUSEUM PLACE
San Jose, California

CONE PENETRATION TEST RESULTS
CPT-4 (CONTINUE)

Date 03/08/16	Project No. 770627701	Figure B-4b

LANGAN TREADWELL ROLLO



ZONE	q_c/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for $q_c \leq 9$ tsf)	Sensitive Fine-Grained
2	1	15 (10 for $q_c \leq 9$ tsf)	Organic Material
3	1	15 (10 for $q_c \leq 9$ tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
q_c = Tip Bearing
f_s = Sleeve Friction
Rf = f_s/q_cx 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

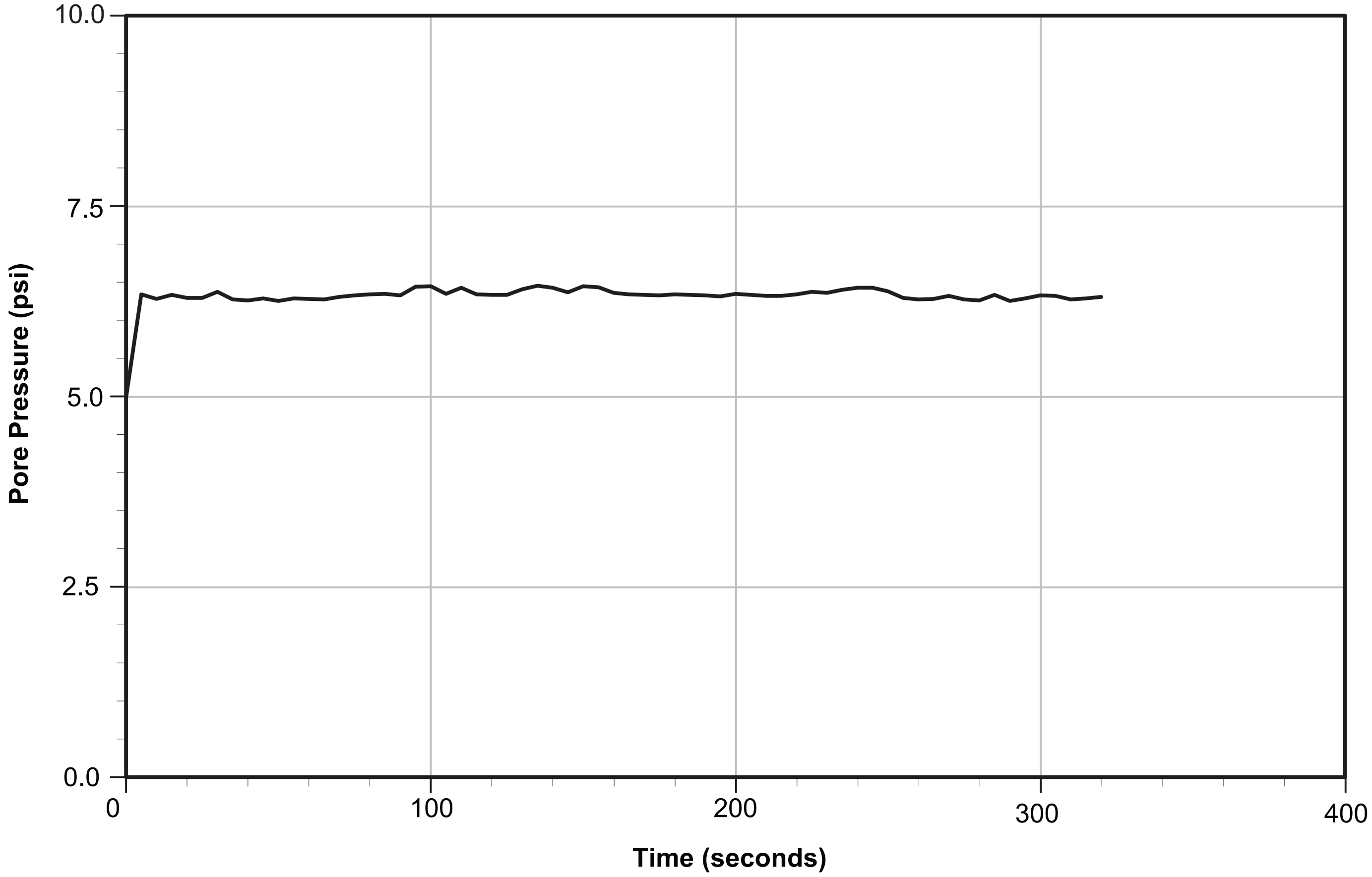
References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud $q_c \leq 9$).
Estimated from local experience (fine-grained soils $q_c > 9$).

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 03/02/16	Project No. 770627701	Figure B-5



CPT	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-1	45.3	6.3	14.5	30.8	58.7

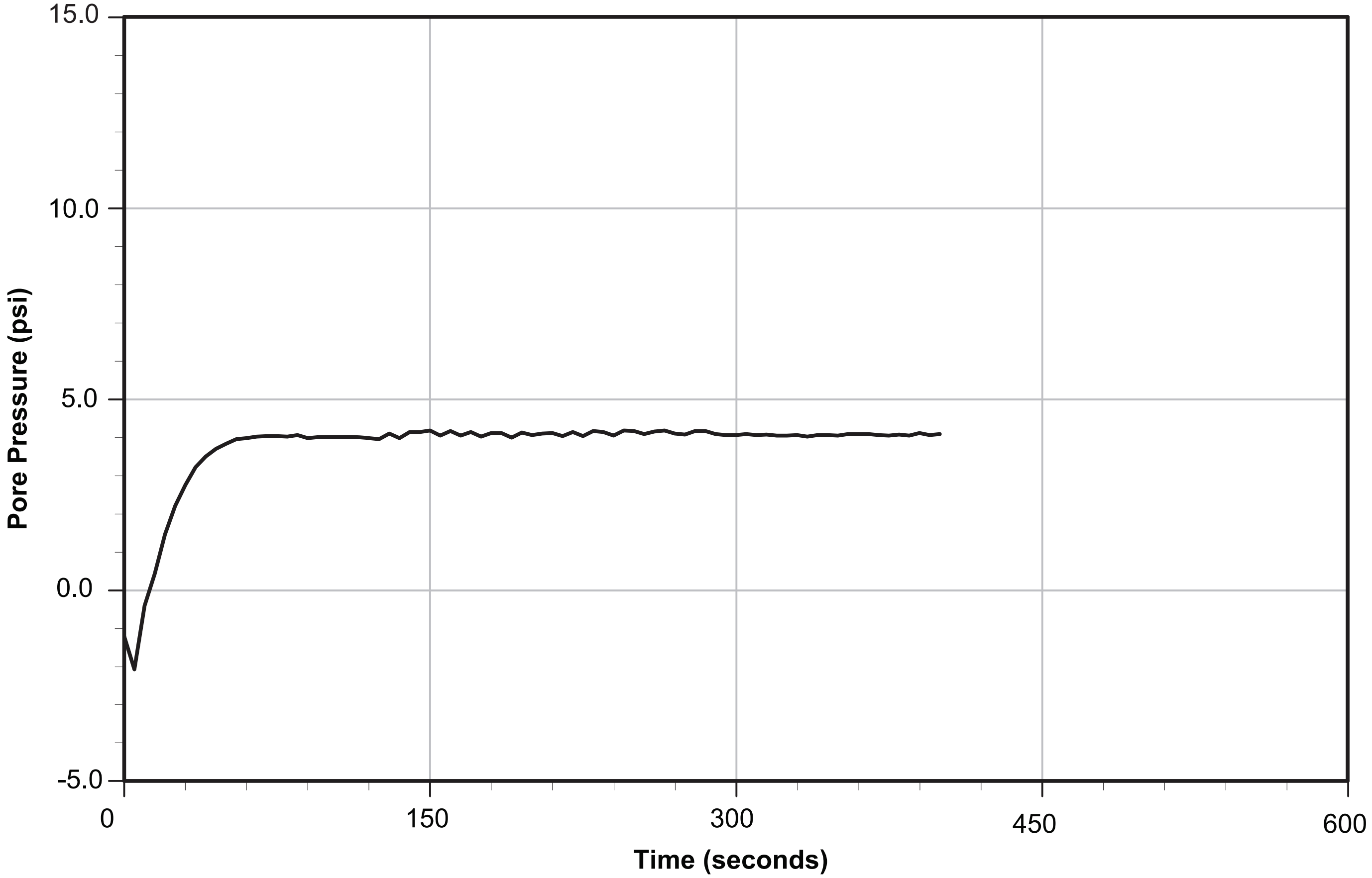
Note:
1. Ground Surface Elevation: 89.5 feet, NGVD 29 Datum.
2. PPDT performed on 01/27/16.

MUSEUM PLACE
San Jose, California

PORE PRESSURE DISSIPATION TEST
CPT-1 AT DEPTH OF 45 FEET

Date 03/08/16 | Project No. 770627701 | Figure B-6

LANGAN TREADWELL ROLLO



CPT	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-2	40.9	4.1	9.5	31.4	58.2

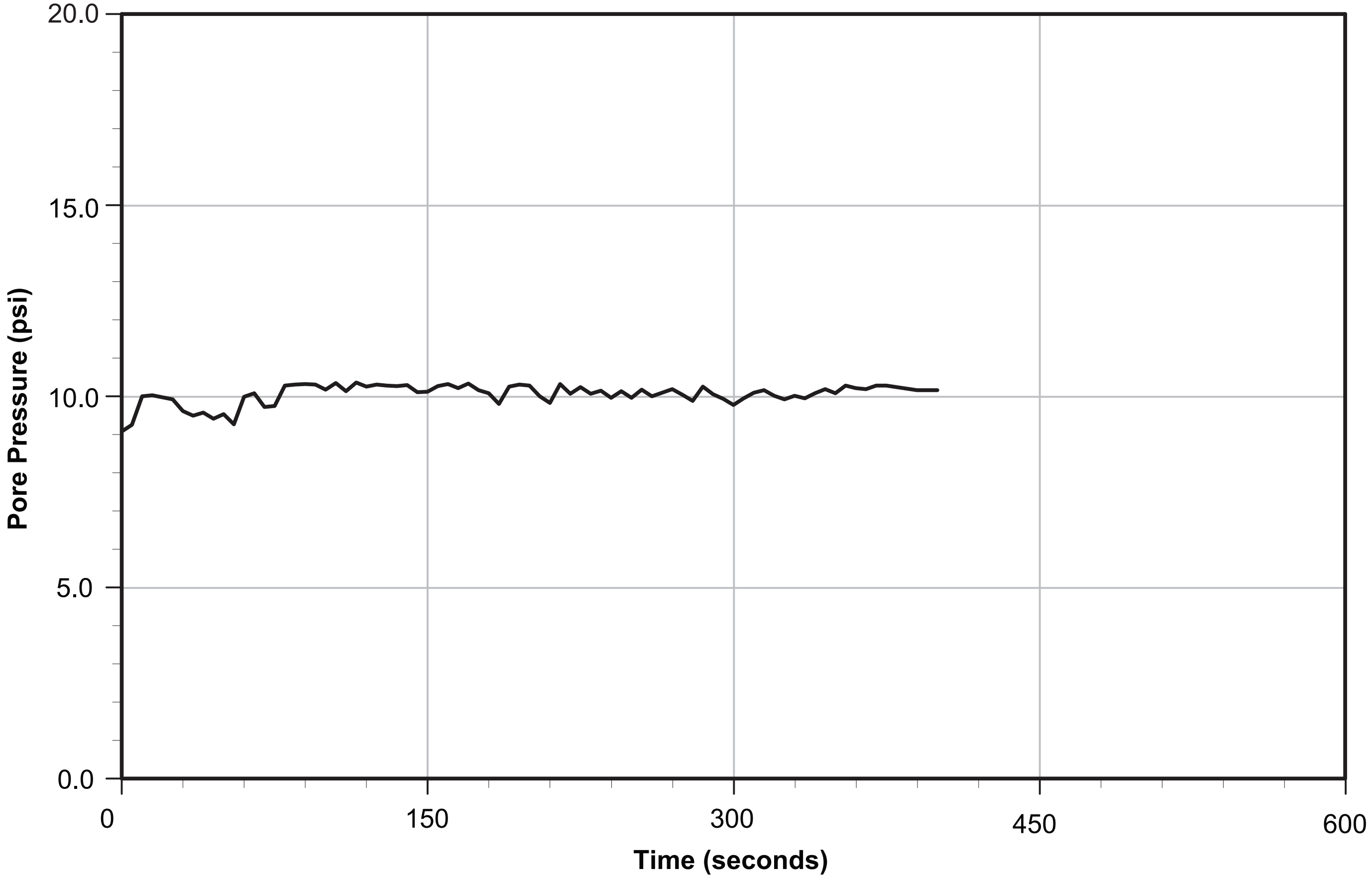
Note:
1. Ground Surface Elevation: 89.6 feet, NGVD 29 Datum.
2. PPDT performed on 01/27/16.

MUSEUM PLACE
San Jose, California

PORE PRESSURE DISSIPATION TEST
CPT-2 AT DEPTH OF 41 FEET

Date 03/08/16 | Project No. 770627701 | Figure B-7

LANGAN TREADWELL ROLLO



CPT	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-2	55	10.1	23.3	31.7	57.9

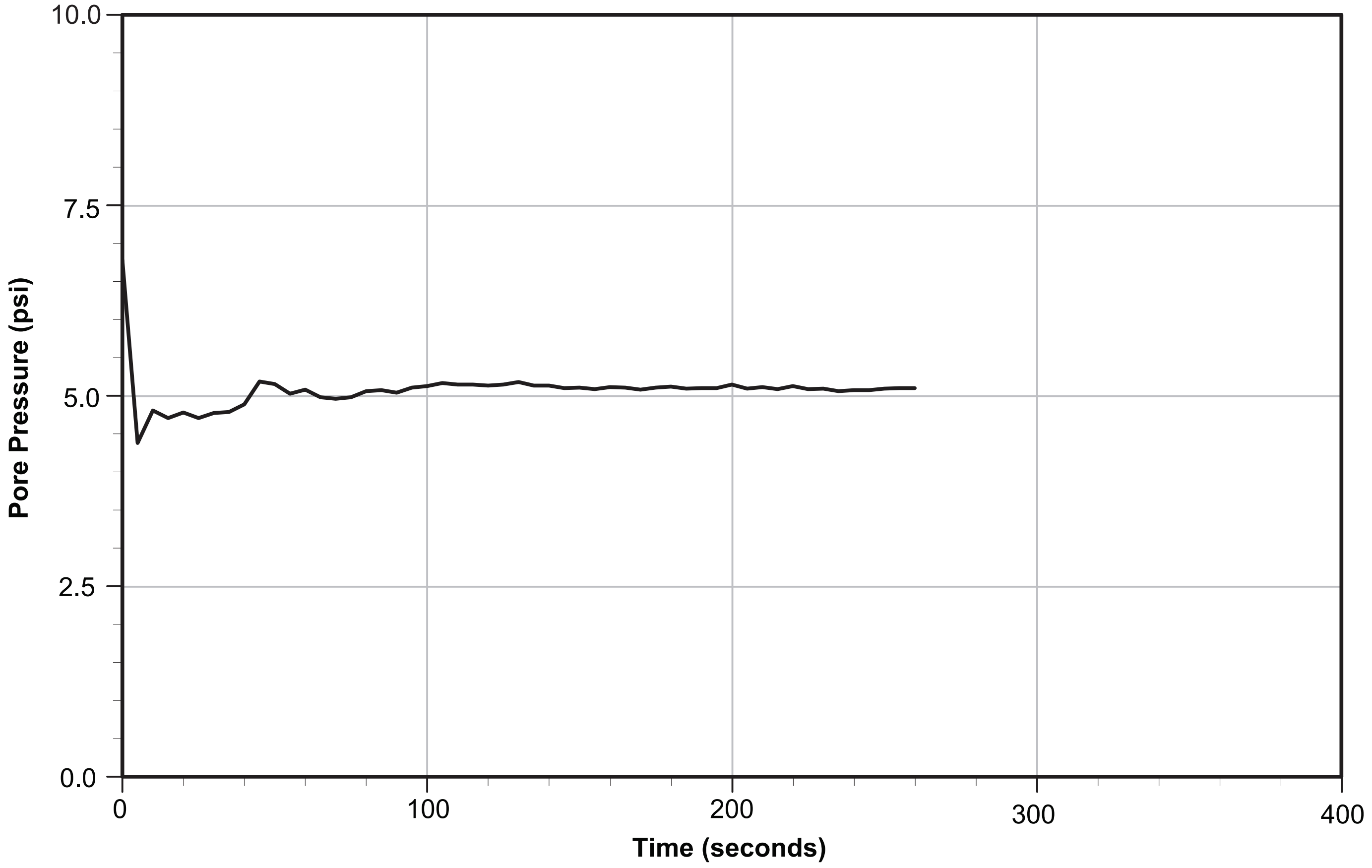
Note:
1. Ground Surface Elevation: 89.6 feet, NGVD 29 Datum.
2. PPDT performed on 01/27/16.

MUSEUM PLACE
San Jose, California

PORE PRESSURE DISSIPATION TEST
CPT-2 AT DEPTH OF 55 FEET

Date 03/08/16 | Project No. 770627701 | Figure B-8

LANGAN TREADWELL ROLLO



CPT	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-3	42.5	5.1	11.8	30.7	58.9

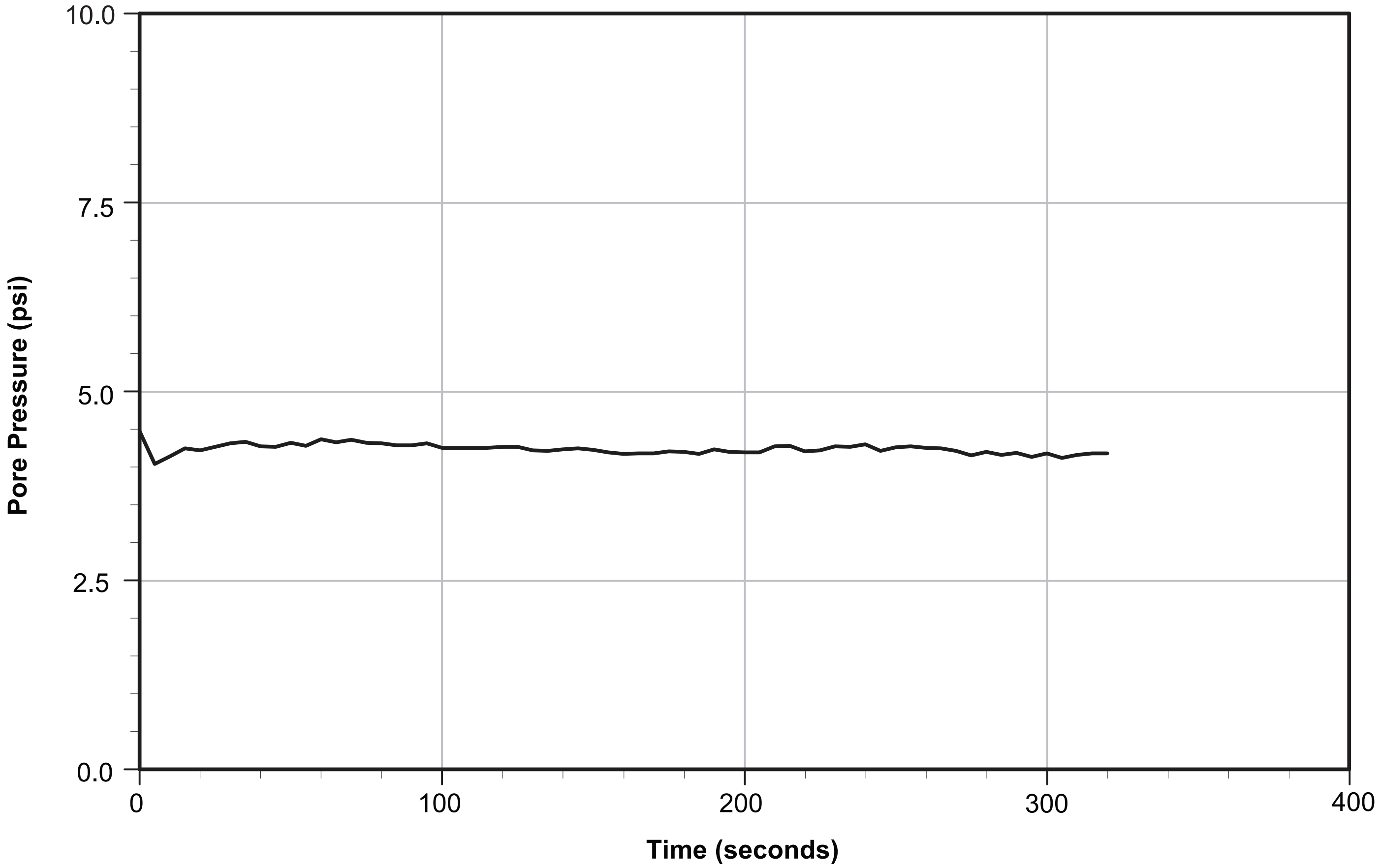
MUSEUM PLACE
San Jose, California

PORE PRESSURE DISSIPATION TEST
CPT-3 AT DEPTH OF 42.5 FEET

Date 03/08/16 | Project No. 770627701 | Figure B-9

LANGAN TREADWELL ROLLO

Note:
1. Ground Surface Elevation: 89.6 feet, NGVD 29 Datum.
2. PPDT performed on 01/27/16.



CPT	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-4	38.7	4.2	9.7	29	59.5

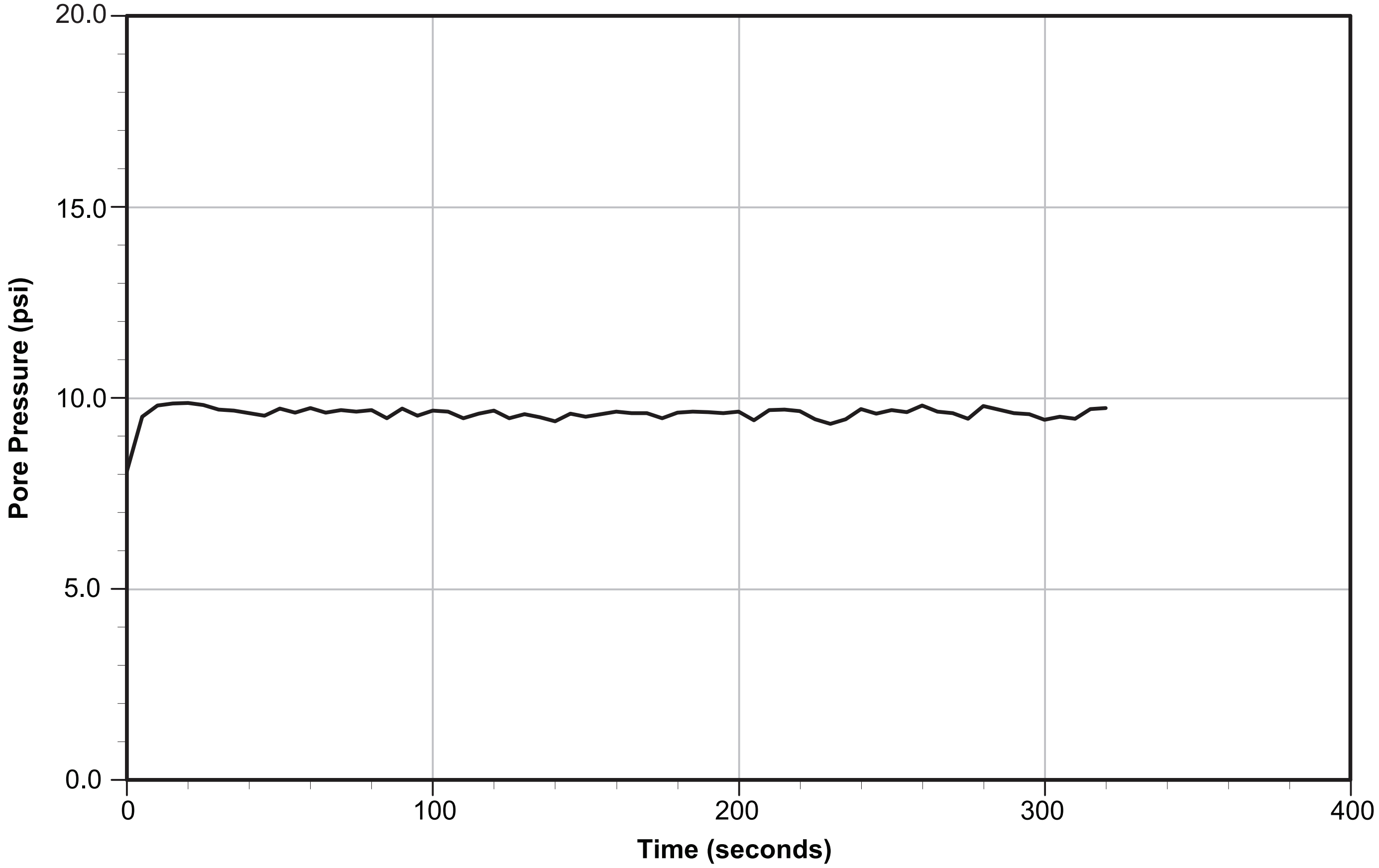
Note:
1. Ground Surface Elevation: 88.5 feet, NGVD 29 Datum.
2. PPDT performed on 01/27/16.

MUSEUM PLACE
San Jose, California

PORE PRESSURE DISSIPATION TEST
CPT-4 AT DEPTH OF 39 FEET

Date 03/08/16 | Project No. 770627701 | Figure B-10

LANGAN TREADWELL ROLLO



CPT	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-4	51.8	9.6	22.2	29.6	58.9

Note:
1. Ground Surface Elevation: 88.5 feet, NGVD 29 Datum.
2. PPDT performed on 01/27/16.

MUSEUM PLACE
San Jose, California

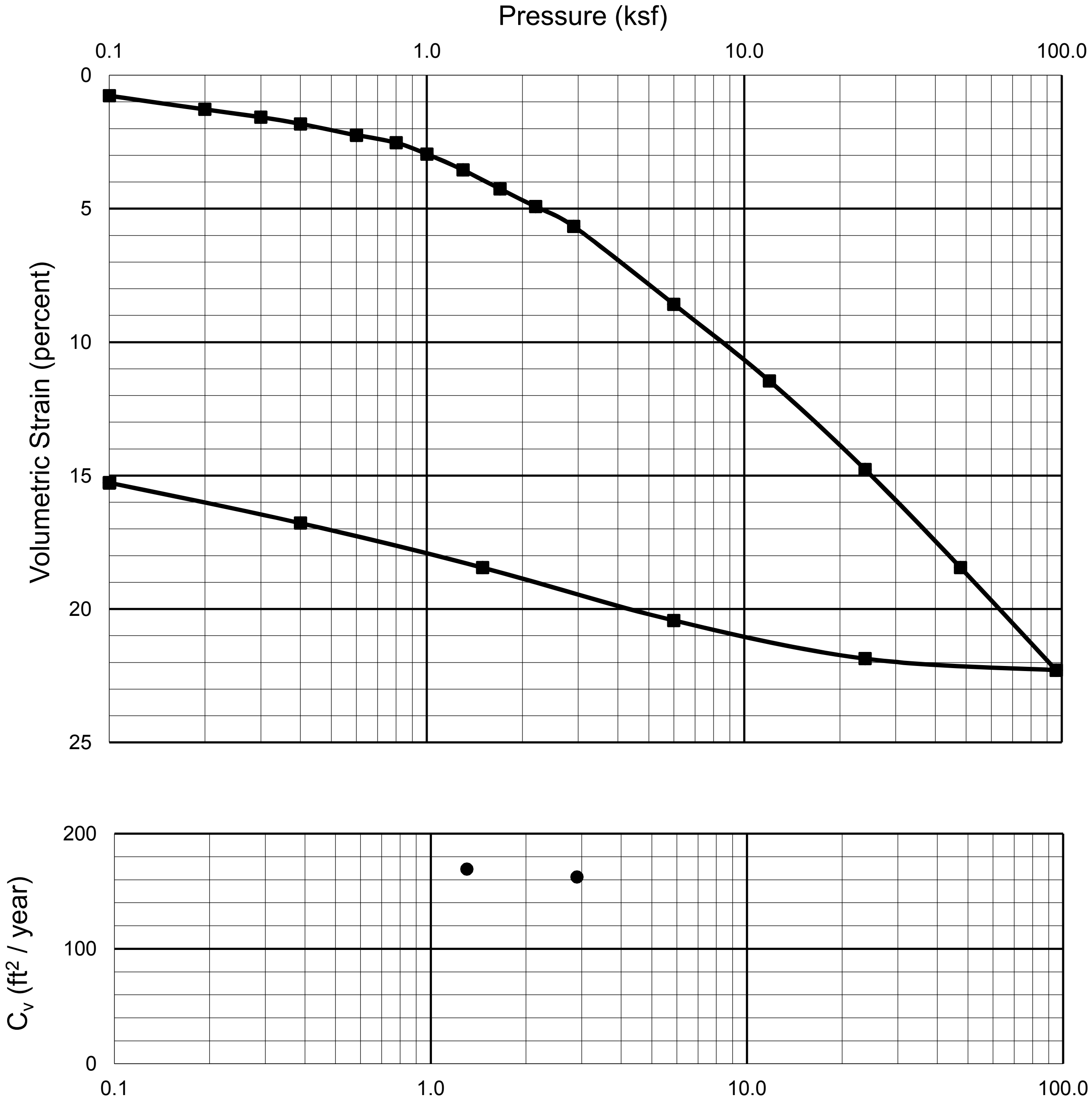
PORE PRESSURE DISSIPATION TEST
CPT-4 AT DEPTH OF 52 FEET

Date 03/08/16 | Project No. 770627701 | Figure B-11

LANGAN TREADWELL ROLLO

APPENDIX C

LABORATORY TEST RESULTS



Sampler Type: Sprague & Henwood		Condition		Before Test		After Test
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	26.7 %	w_f	17.7 %
Overburden Pressure, p_o	3,200 psf	Void Ratio	e_o	0.74	e_f	0.48
Preconsol. Pressure, p_c	3,200 psf	Saturation	S_o	97 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.12	Dry Density	γ_d	97 pcf	γ_d	114 pcf

LL - -	PL - -	PI - -	G_s 2.70 (assumed)

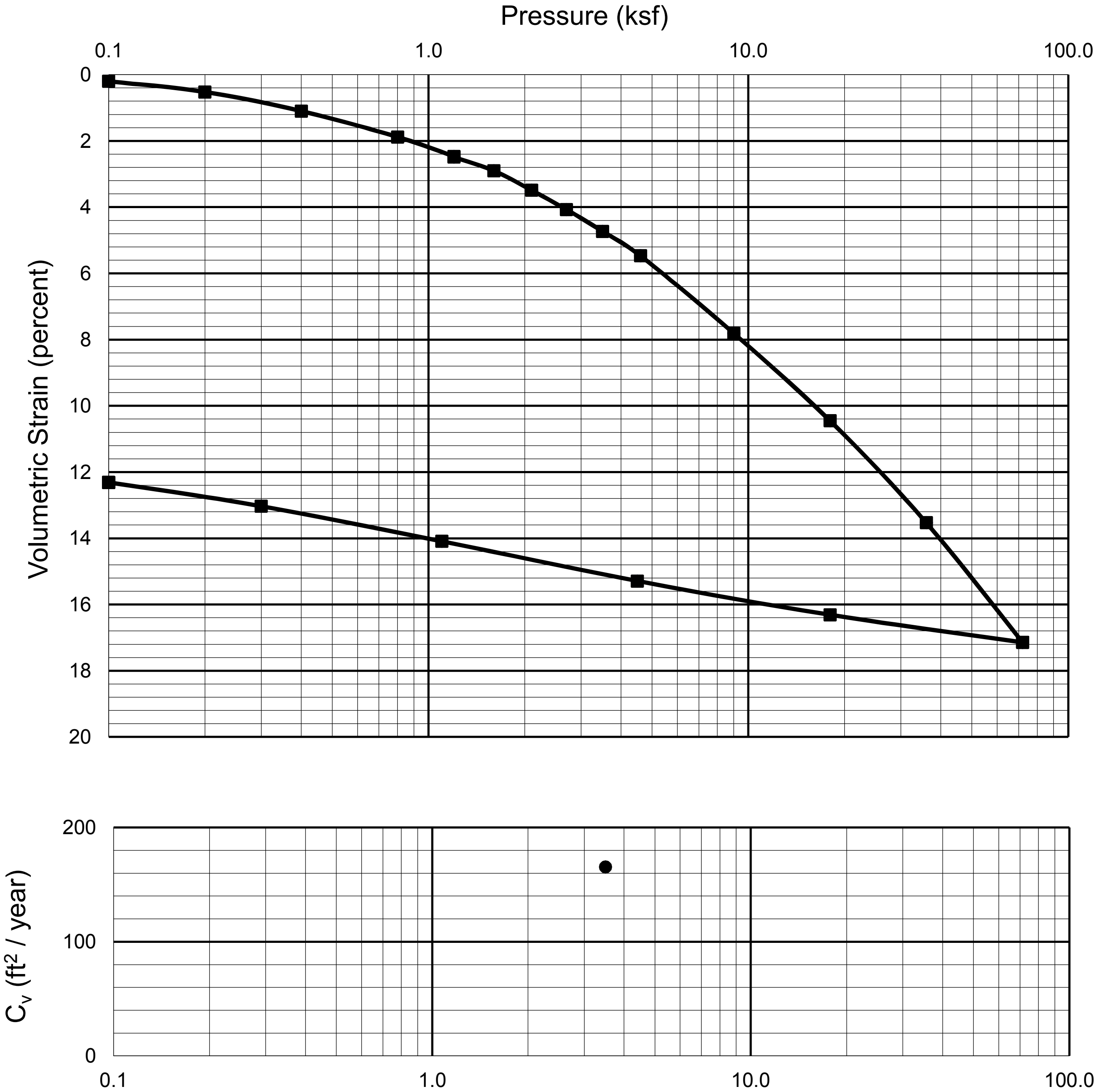
Classification CLAY (CL), yellow-brown Source B-1 at 41 feet

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date 03/01/16 Project No. 770627701 Figure C-1



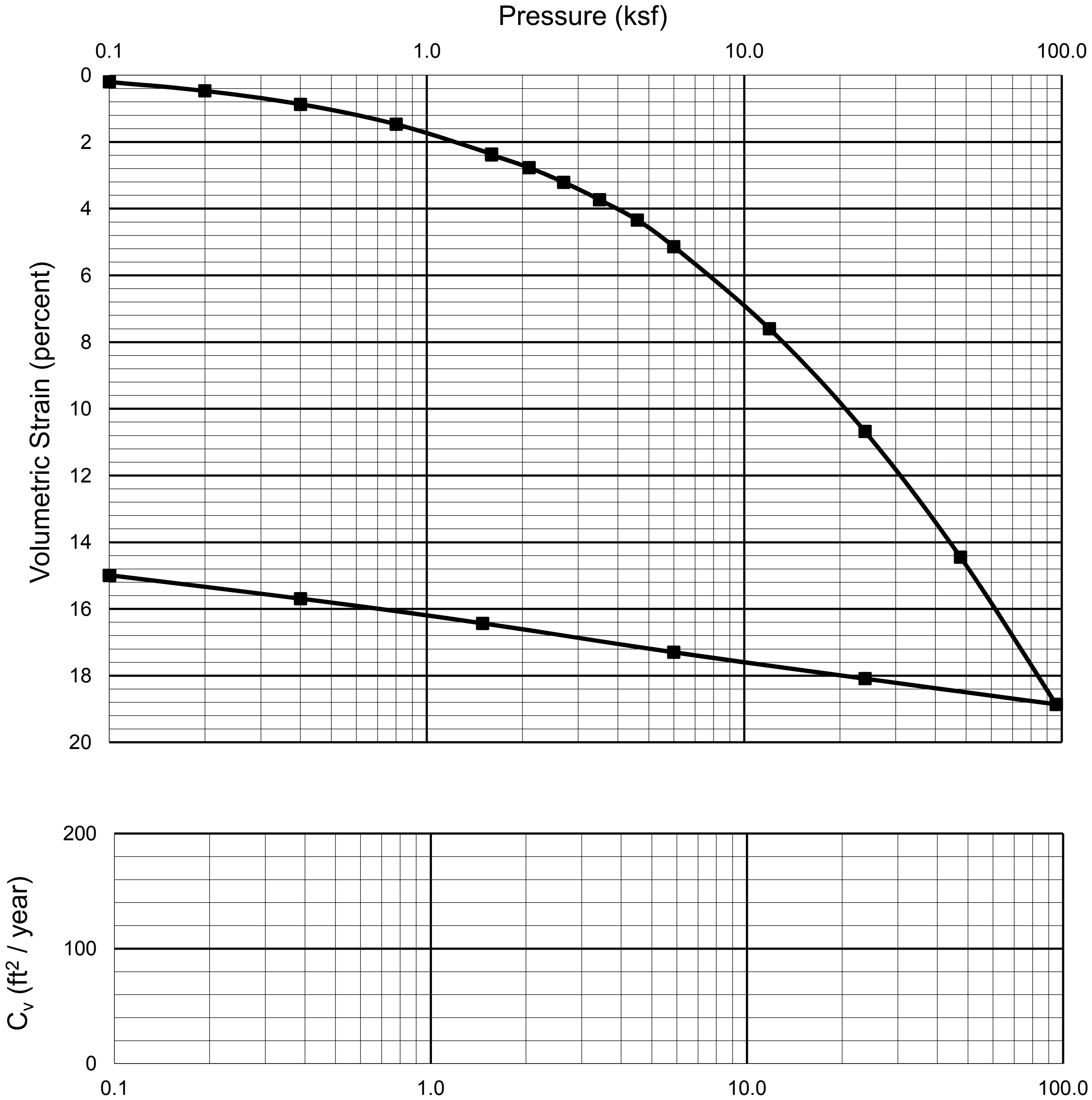
Sampler Type: Sprague & Henwood			Condition	Before Test			After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	26.7 %		w_f	19.9 %
Overburden Pressure, p_o	5,300	psf	Void Ratio	e_o	0.78		e_f	0.56
Preconsol. Pressure, p_c	5,300	psf	Saturation	S_o	93 %		S_f	96 %
Compression Ratio, $C_{\varepsilon c}$	0.12		Dry Density	γ_d	95 pcf		γ_d	108 pcf
LL - -	PL - -		PI - -		G_s	2.70	(assumed)	
Classification	SANDY CLAY (CL), olive-gray to gray		Source	B-2 at 76 feet				

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date 03/01/16 Project No. 770627701 Figure C-2



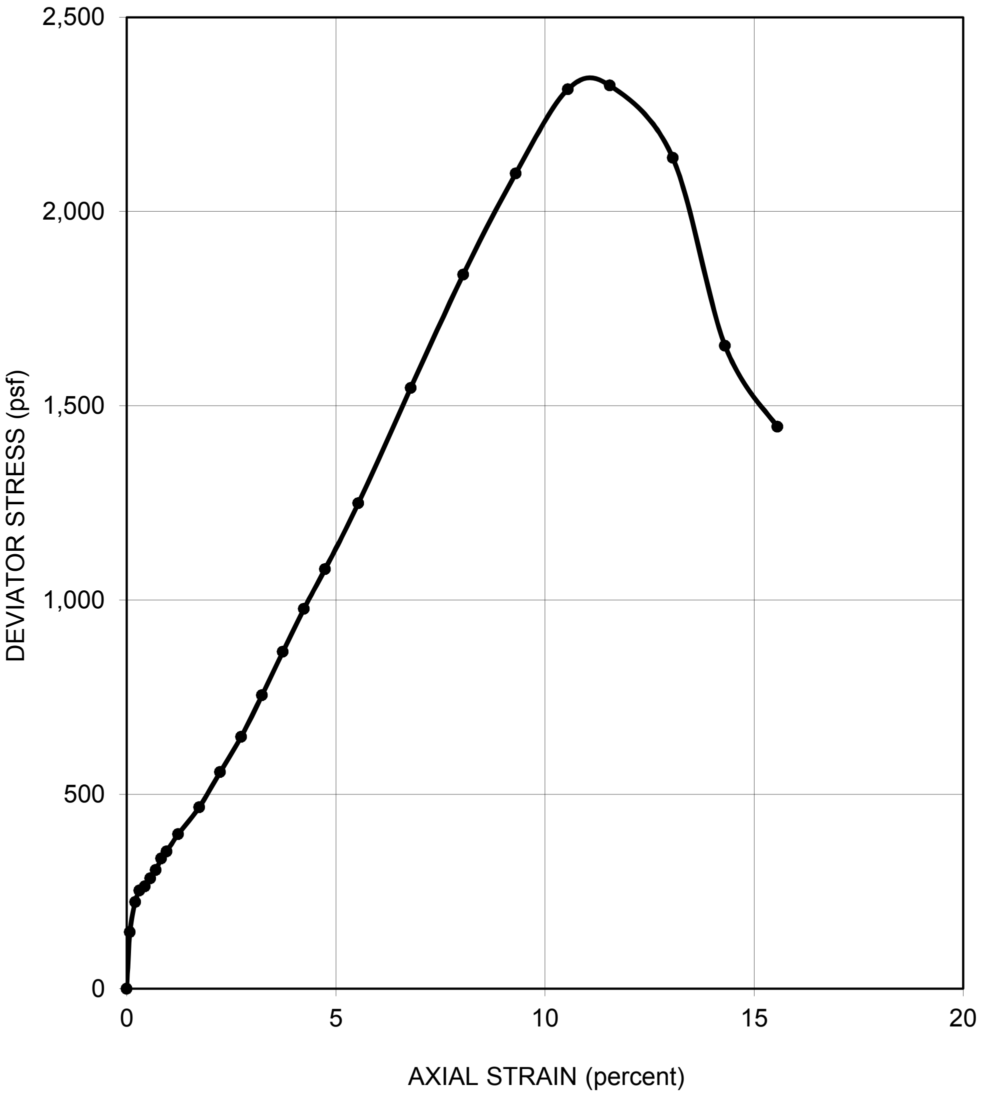
Sampler Type: Shelby Tube				Condition	Before Test			After Test		
Diameter (in) 2.42	Height (in)	1.00		Water Content	w_o	30.8	%	w_f	21.6	%
Overburden Pressure, p_o	2,800	psf		Void Ratio	e_o	0.86		e_f	0.58	
Preconsol. Pressure, p_c	9,300	psf		Saturation	S_o	96	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.12			Dry Density	γ_d	91	pcf	γ_d	107	pcf
LL - -	PL - -		PI - -			G_s	2.70	(assumed)		
Classification SILTY CLAYEY SAND (SC-SM), gray			Source	B-3 at 33 feet						

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date 03/01/16 | Project No. 770627701 | Figure C-3



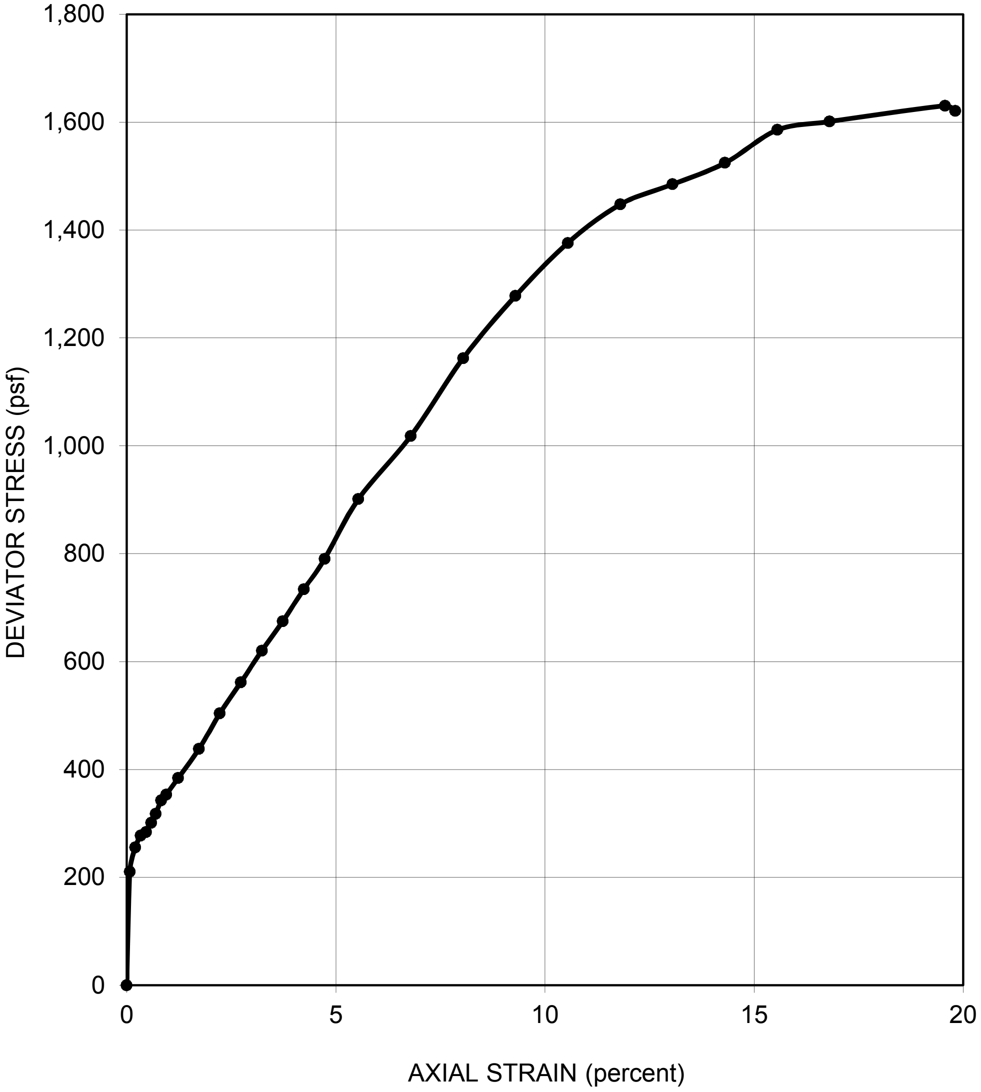
SAMPLER TYPE	Sprague & Henwood				SHEAR STRENGTH	1,160	psf
DIAMETER (in.)	2.42	HEIGHT (in.)	5.64		STRAIN AT FAILURE	11.6	%
MOISTURE CONTENT			26.8	%	CONFINING PRESSURE	3,100	psf
DRY DENSITY			96	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), olive-brown to olive-gray				SOURCE	B-1 at 26 feet	

MUSEUM PLACE
San Jose, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 03/01/16 Project No. 770627701 Figure C-4



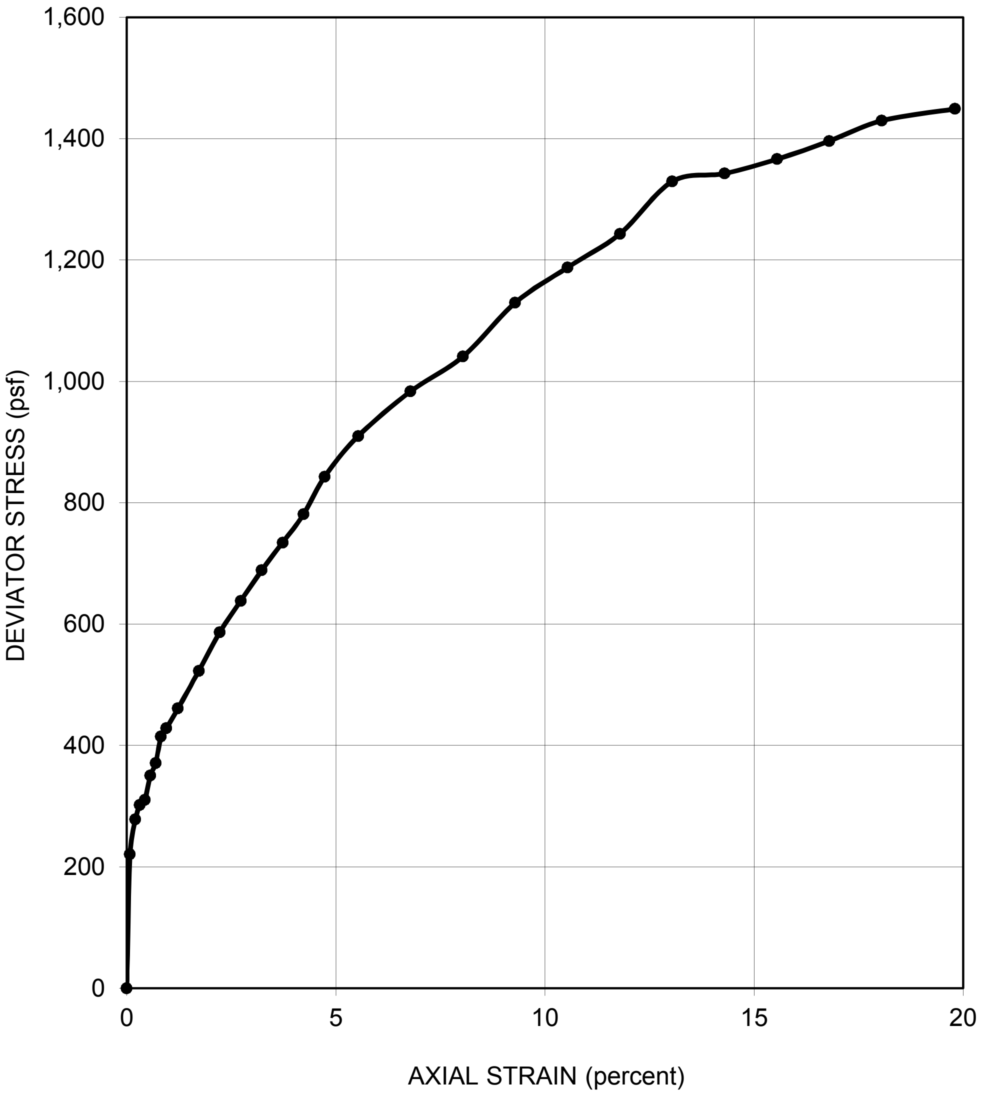
SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	820	psf
DIAMETER (in.)	2.42	HEIGHT (in.) 5.73	STRAIN AT FAILURE	19.6	%
MOISTURE CONTENT	32.0	%	CONFINING PRESSURE	3,700	psf
DRY DENSITY	90	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), gray to dark gray		SOURCE	B-1 at 36 feet	

MUSEUM PLACE
San Jose, California

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 03/01/16 | Project No. 770627701 | Figure C-5



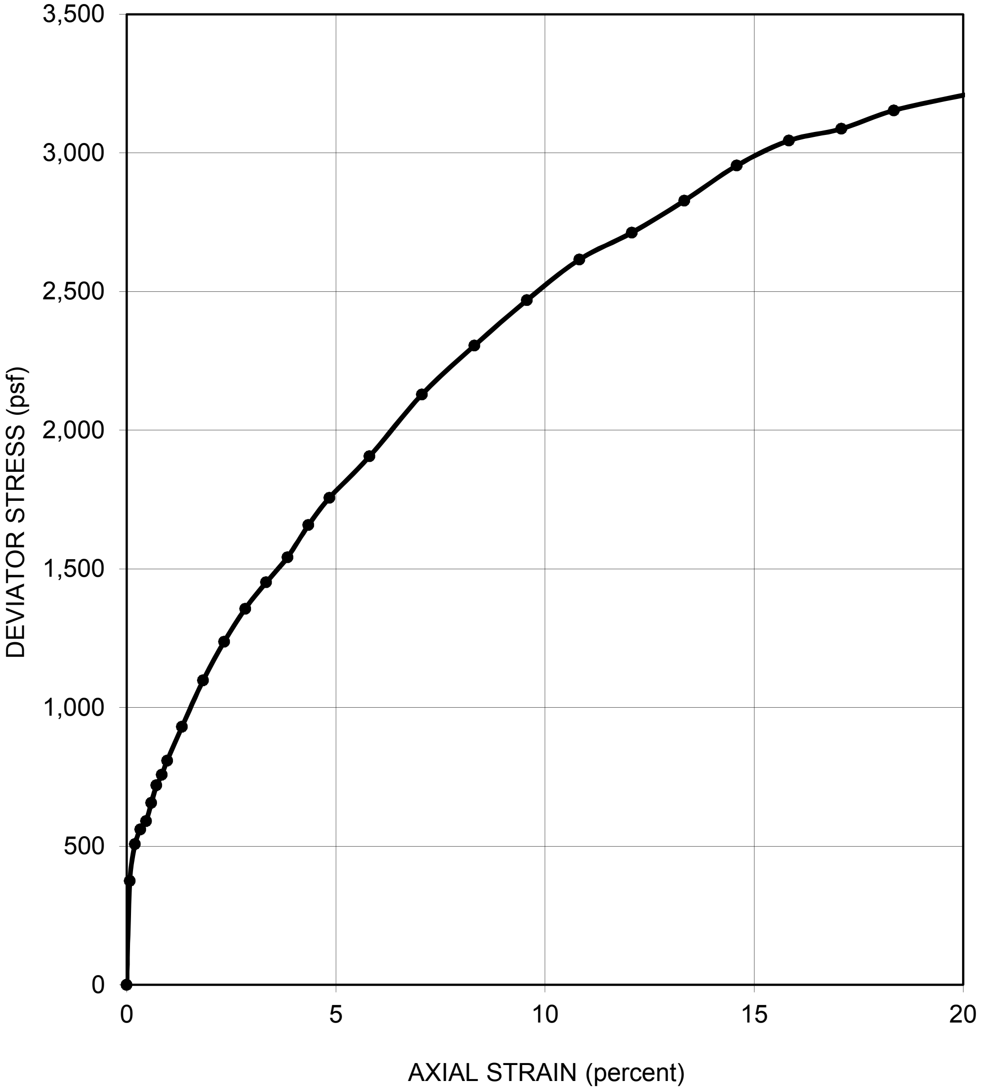
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	720	psf
DIAMETER (in.)	2.42	HEIGHT (in.)	5.96	STRAIN AT FAILURE	19.8	%
MOISTURE CONTENT		21.3	%	CONFINING PRESSURE	4,300	psf
DRY DENSITY		106	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown with orange mottling			SOURCE	B-1 at 46 feet	

MUSEUM PLACE
San Jose, California

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 03/01/16 | Project No. 770627701 | Figure C-6



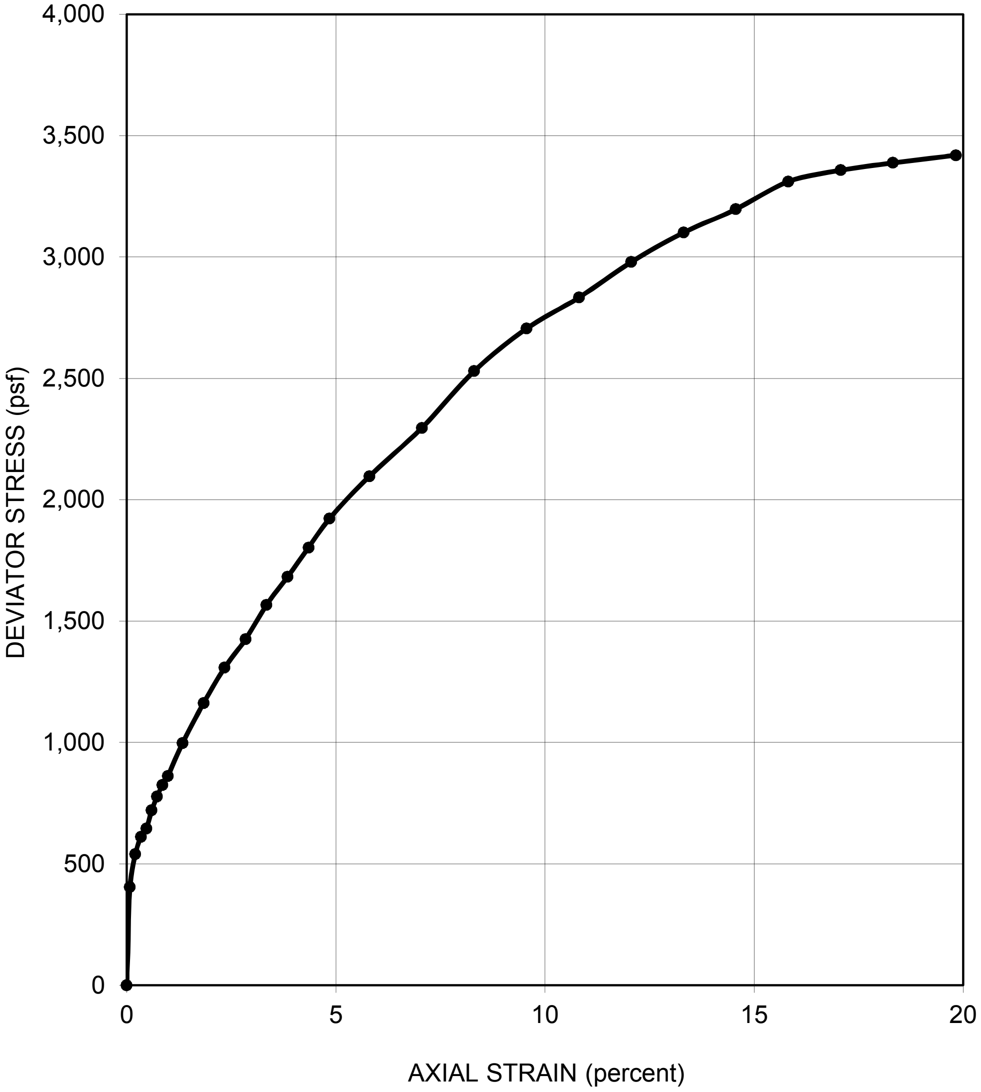
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH		1,600	psf
DIAMETER (in.)	2.40	HEIGHT (in.)	5.73	STRAIN AT FAILURE		20.0	%
MOISTURE CONTENT		22.8	%	CONFINING PRESSURE		6,500	psf
DRY DENSITY		105	pcf	STRAIN RATE		0.75	% / min
DESCRIPTION	CLAY (CL), olive-gray with occasional light gray mottling				SOURCE	B-1 at 81 feet	

MUSEUM PLACE
San Jose, California

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 03/01/16 | Project No. 770627701 | Figure C-7



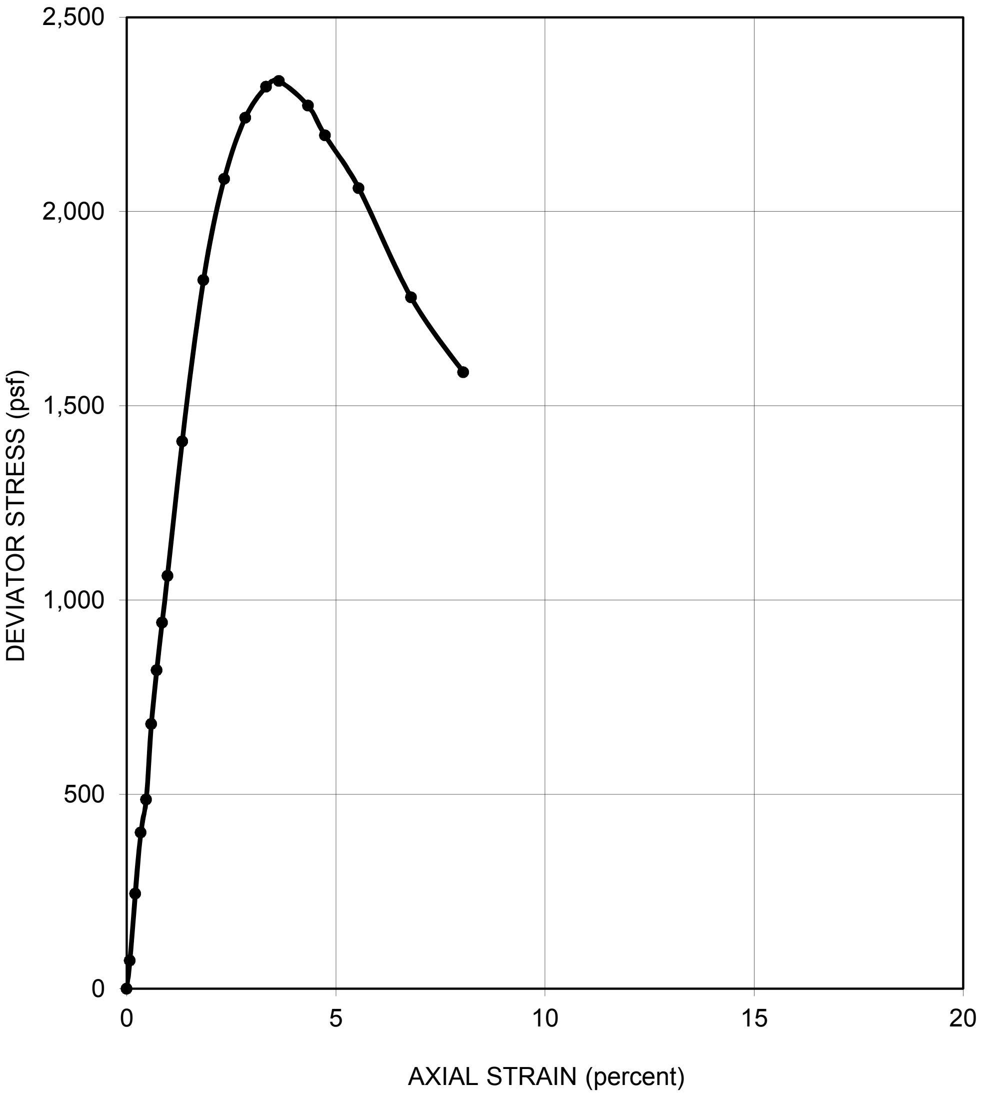
SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	1,710	psf
DIAMETER (in.)	2.40	HEIGHT (in.) 6		STRAIN AT FAILURE	19.8	%
MOISTURE CONTENT		20.2	%	CONFINING PRESSURE	6,800	psf
DRY DENSITY		108	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), olive-gray to gray				SOURCE	B-2 at 85.5 feet

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 03/01/16 | Project No. 770627701 | Figure C-8



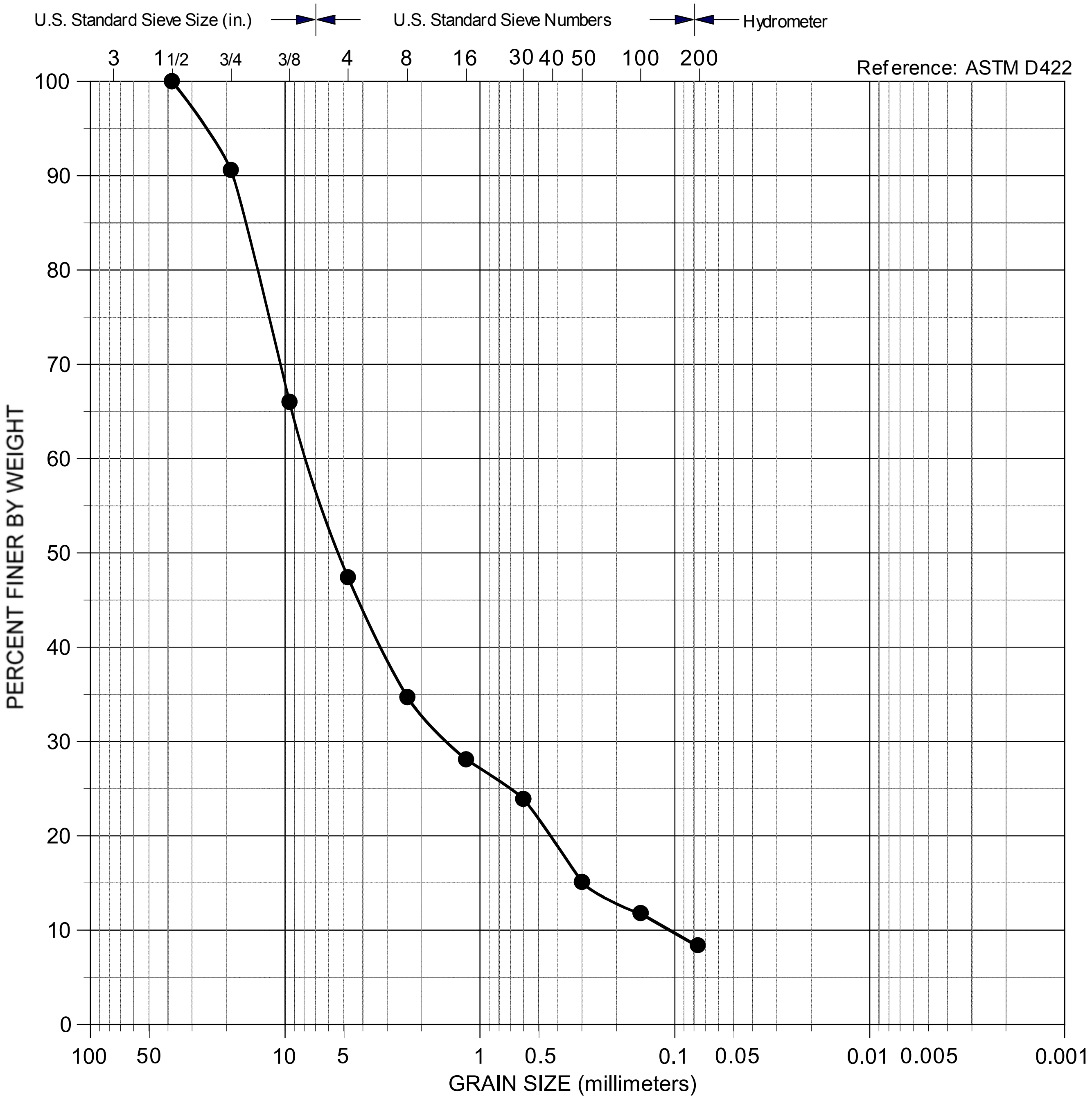
SAMPLER TYPE	Srague & Henwood		SHEAR STRENGTH	1,170	psf
DIAMETER (in.)	2.86	HEIGHT (in.) 6.08	STRAIN AT FAILURE	3.6	%
MOISTURE CONTENT	35.5	%	CONFINING PRESSURE	1,600	psf
DRY DENSITY	82	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), yellow-brown and olive with occasional orange		SOURCE	B-3 at 13 feet	

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 03/01/16 | Project No. 770627701 | Figure C-9



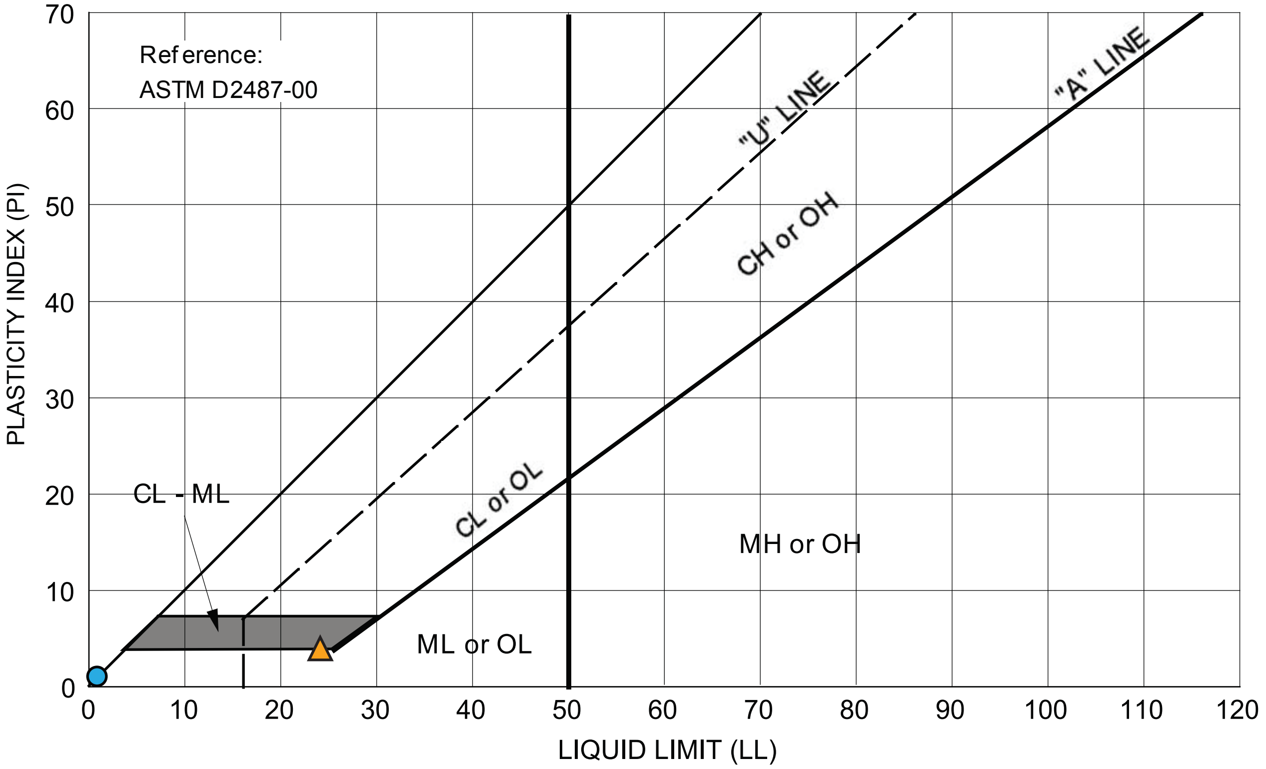
Symbol	Sample Source	Classification
●	B-2 at 40 feet	GRAVEL with SILT and SAND (GP-GM), yellow-brown

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

PARTICLE SIZE ANALYSIS

Date 3/8/16	Project No. 770627701	Figure C-10



NP = Non Plastic

Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 31.5 feet	SILTY SAND (SM), olive-gray	22.7	--	NP	35.8
▲	B-3 at 31.5 feet	CLAYEY SILTY SAND (SC-SM), gray	19.9	24	4	---

MUSEUM PLACE
San Jose, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 03/08/16 | Project No. 770627701 | Figure C-11

APPENDIX D

SOIL CORROSIVITY 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

10 March, 2016

Job No. 1603023
Cust. No. 12242

Mr. Gabriel Alcantar
Langan Treadwell Rollo
4030 Moorpark Avenue, Suite 210
San Jose, CA 95117

Subject: Project No.: 770627701.700.313
Project Name: 180 Park Ave., San Jose
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Alcantar:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on March 02, 2016. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurements, the sample 001 is classified as "moderately corrosive" and sample 002 is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations are both none detected to 15 mg/kg

The sulfate ion concentrations range from 70 to 440 mg/kg and are determined to be sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The pH of the soils range from 8.08 to 8.27, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 430 to 450-mV which is indicative of aerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

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: Langan Treadwell Rollo
Client's Project No.: 770627701.700.313
Client's Project Name: 180 Park Ave., San Jose
Date Sampled: 01/25 & 27/16
Date Received: 2-Mar-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 10-Mar-2016

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)*
1603023-001	B-2 + B-3 @ 6'	450	8.27	-	3,500	-	N.D.	70
1603023-002	B-1 @ 18'	430	8.08	-	890	-	N.D.	440

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	7-Mar-2016	7-Mar-2016	-	9-Mar-2016	-	7-Mar-2016	7-Mar-2016

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen

Cheryl McMillen
Laboratory Director

<u>**Quality Control Summary**</u> - **All laboratory quality control parameters were found to be within established limits**

1603023

Chain of Custody

Page 1 of 1

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775
CERCO analytical

Job No.	CU#		Client Project I.D.
770627701-700-313	12242		

Schedule
Analyte

Date Sampled 1/27/15

Date Due 3/7

Full Name Gabriel Alcantar

Fax

Phone

Company Langan Engineering

Cell 415 955-5298 ☒

Sample Source 180 Park Ave San Jose, CA

ASTM w/Brief Evaluation

ANALYSIS

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation
①	B-2+B-3 @ 6'	1/25, 1/26/16	–	S	stt	1/2gal	–	1	x	x	x	x	x		X
②	B-1 at 18'	1/27/16	–	S	stt	1/2gal	–	1	X	X	X	X	X		X

GSO

MATRIX	ABBREVIATIONS	SAMPLE RECEIPT	
DW - Drinking Water GW - Ground Water SW - Surface Water WW - Waste Water Water SL - Sludge S - Soil Product	HB - Hosebib PV - Petcock Valve PT - Pressure Tank PH - Pump House RR - Restroom GL - Glass PL - Plastic ST - Sterile	Total No. of Containers Rec'd Good Cond/Cold Conforms to Record Temp. a t Lab -°C Sampler	

	Date	Time
Relinquished By:	3/1/16	9am
Received By:	3/2/16	1146
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		

Comments:

THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES

DISTRIBUTION

4 copies: Insight King Wah, LLC
c/o Mathew Love
333 W. Santa Clara Street, Suite 805
San Jose, California 95113
Insight Realty Company

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John Gouchon

John Gouchon, G.E. #2282
Principal