GEOTECHNICAL INVESTIGATION
PASSAGE AT SAN MATEO
San Mateo, California

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GEOTECHNICAL INVESTIGATION
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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation by Langan Engineering and Environmental Services, Inc. (Langan) for the proposed Passage at San Mateo development in San Mateo, California. The approximate location of the site is shown on Figure 1.

The site is bound by Concar Drive to the north, Highway 92 to the south, South Delaware Street to the west, and South Grant Street to the east, as shown on Figure 2. The site is currently occupied by several retail and commercial buildings, including buildings occupied by Ross, Rite Aid, Peninsula Ballet Theatre, the Pantry, Trader Joes and The Shane Company. The buildings are surrounded by asphalt-paved parking lots. We understand the 0.23 acre parcel at the northwest corner of the block that is currently occupied by an existing 7-Eleven is not part of the proposed development. Based on a topographic survey (BKF, 2018), the site is relatively flat with the ground surface elevations ranging from approximately Elevation 101 to 104 feet[1].

We understand current development plans (MVE, 2018) for the site consist of the demolition of the existing buildings and the construction of four new structures, as shown on Figure 3. The proposed development will occupy the majority of the city block. The four proposed structures, designated as Buildings 1 through 4 (as shown on Figure 3), will be five-stories above one basement level. The proposed basement and Level 1 will be concrete construction and Levels 2 through 5 will be wood frame construction. Based on information from BKF, the project civil engineer, the first floor (ground floor) elevation will be at approximately Elevation 104 feet for Buildings 1 and 2 and Elevation 103 to 104 feet for Buildings 3 and 4. The finished floor elevation for the basement levels will be at Elevation 93 to 94 feet.

In addition, the proposed development will include a new access road (Depot Way) and an at-grade landscape area (Central Park) at the center of the property.

Langan presented the results of a preliminary geotechnical investigation in a letter report dated 9 February 2016 for a previously planned development at the site that was not constructed. We understand Coastal California Properties obtained the right to rely on the information

[1] All elevations reference San Mateo City Datum plus 100 feet, except where noted.

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contained in that letter report. This report uses the data obtained during the previous studies and supersedes the 9 February 2016 letter report.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposal dated 15 November 2016. Our scope of services consisted of reviewing available information from our previous exploration and performing engineering analyses to develop conclusions and recommendations regarding:

- anticipated subsurface conditions including groundwater levels;
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, fault rupture;
- appropriate foundation type(s) including shallow foundations and alternatives for deep foundations, as necessary;
- estimates of foundation settlement;
- design parameters for the recommended foundation type(s), including allowable bearing capacity, passive pressure, and coefficient of base friction for shallow foundations and vertical and lateral resistance of deep foundations, as appropriate;
- subgrade preparation for at grade floors and exterior slabs and flatwork, including sidewalks;
- site preparation, grading, and excavation, including criteria for fill quality and compaction;
- below grade wall pressures;
- 2016 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};
- flexible pavements;
- soil corrosivity;
- construction considerations.

3.0 PREVIOUS FIELD EXPLORATION AND LABORATORY TESTING

We began our investigation by reviewing the results of the previous geotechnical investigation we performed at the site. During our previous geotechnical investigation at the site, we drilled seven borings and performed eleven Cone Penetration Tests (CPTs) at the site. The approximate locations of the borings and CPTs are presented on Figures 2 and 3.

Prior to performing the field exploration, we obtained a geotechnical drilling permit from San Mateo County Environmental Health Services, notified Underground Service Alert (USA) and checked the boring locations for underground utilities using a private utility locator. Details of each aspect of the field exploration and laboratory testing are discussed in the remainder of this section.

3.1 Borings

Seven borings, designated B-1 through B-7, were drilled on 10 and 11 December 2015 using truck-mounted drill rigs operated by Exploration Geoservices, Inc. and Pitcher Drilling Company. Borings B-1 through B-3 were drilled with rotary wash drilling equipment to approximately 61.5 feet below ground surface (bgs) and borings B-4 through B-7 were drilled with a hollow stem auger to approximately 60 feet bgs. Our engineers logged the borings and obtained samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-7. The soil encountered in the borings was classified in accordance with the Classification Chart presented on Figure A-8. Soil samples were obtained using three different types of samplers: two driven split-barrel samplers and two piston 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) thin wall sampler with a 3.0-inch outside diameter and a 2.875-inch inside diameter

- Dames & Moore (D&M) thin walled sampler with a 2.5-inch outside diameter, lined with 2.43-inch-inside-diameter brass tubes

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 penetration resistance of sandy soil. The ST and D&M samplers were 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 (Borings B-1 through B-3) and a downhole wireline hammer (Borings B-4 through B-7)

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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 0.7 and 1.2 for Borings B-1 through B-3 and 0.6 and 1.0 for Borings B-4 through B-7, respectively, to account for sampler type and hammer energy, and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts.

The ST and D&M samplers are pushed hydraulically into the soil; the piston pressure required to advance the sampler is shown on the log, measured in pounds per square inch (psi).

Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of San Mateo County Environmental Health Services.

The soil cuttings from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and transported off-site for proper disposal.

3.2 Cone Penetration Test

Eleven CPTs (designated as CPT-1 through CPT-11) were performed on 10 to 11 December 2015 by Middle Earth Geo Testing Inc. at the approximate locations shown on Figures 2 and 3. The CPTs were advanced to depths of approximately 56 to 92 feet bgs.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe, with a projected area of 15 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-11. Soil types were estimated using the classification chart shown on Figure B-12.

Pore-pressure dissipation tests (PPDTs) were performed during the advancement of CPT-1, CPT-2, CPT-5 through CPT-9, and CPT-11 at various depths. PPDTs are conducted at various depths to measure hydrostatic water pressures and to determine the approximate depth of the groundwater level. The variation of pore pressure with time is measured behind the tip of the cone and recorded. For this investigation, the duration of the tests range from approximately 100 to 350 seconds. The results of the eight PPDTs are presented on Figures B-13 through B-20.

Upon completion of the field investigation, the CPT holes were backfilled with cement-bentonite grout in accordance with the requirements of San Mateo County Environmental Health Services.

3.3 Laboratory Testing

The samples recovered from the field investigation were examined to verify their soil classification, and representative samples were selected for laboratory testing. Samples were tested to measure moisture content, fines content, gradation, shear strength, plasticity (Atterberg Limits), R-value and compressibility, where appropriate. Results of the laboratory tests are included on the boring logs and in Appendix C.

3.4 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

Site and subsurface conditions are presented in this section.

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4.1 Site History

The site is located within a broad area of tidal slough and marshland reclaimed along the peninsula section of San Francisco Bay. Historically, reclamation in these low-lying areas involved constructing dykes and draining enclosed tracts, then capping the surface with a layer of imported fill. Based on data from a nearby site (Treadwell & Rollo, 2001), the existing fill was most likely placed in the 1960s.

The site is approximately 14½-acres and is currently occupied by commercial buildings with surface parking. Based on a topographic survey (BKF, 2018), the existing site is relatively flat with ground surface elevations ranging from approximately Elevation 101 to 104 feet.

4.2 Subsurface Conditions

Idealized subsurface profiles are presented on Figures 4 and 5; the locations of the profiles are shown on Figures 2 and 3. Where explored, pavement sections of approximately 2 to 6 inches of asphalt concrete (AC) underlain by up to 12 inches of aggregate base (AB) were encountered. Beneath the pavement section, the fill blanketing the site is a mixture of loose to dense sand, silty sand, and clayey sand with varying amounts of gravel and medium stiff to very stiff clay with varying amounts of sand, gravel, organics, and glass and wood debris. Where tested, laboratory tests indicate the clay fill is moderately expansive[2] with a plasticity index (PI) of 22. The fill varies in thickness from about 4 to 7 feet, with corrosivity analyses indicating the fill material is corrosive.

The fill overlies a layer of weak, compressible marine clay known locally as Bay Mud. The thickness of Bay Mud underlying the project site ranges from 4 to 13½ feet and increases in thickness to the east. Where tested, the undrained shear strength of the Bay Mud is 340 pounds per square foot (psf). Laboratory test results indicate the Bay Mud has a compression ratio of 0.25, is under to normally consolidated[3] and severely corrosive. Contours of approximate bottom of Bay Mud elevations are presented on Figure 6.

The Bay Mud is underlain by medium stiff to very stiff clay, clay with sand, and sandy clay and interbedded layers of medium dense to very dense sand and gravel with varying amounts of silt and clay to the maximum extent explored. Where tested, the undrained shear strengths of the

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[2] Moderately expansive soil undergoes moderate volume changes with changes in moisture content.

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

clay range from 1,100 to 1,720 psf. The majority of the clay is overconsolidated, except near B-6 where the clay was slightly underconsolidated. Also where tested, the sand contains 12 to 39 percent fines (particle passing the No. 200 sieve).

Historically, groundwater was encountered in the site vicinity at the bottom of fill elevation at depths of approximately 4 to 8 feet bgs, corresponding to Elevation 95.5 to 97.5 feet. At the time of our field investigation in December 2015, the Hines development at 400 and 450 Concar Drive (located approximately 150 feet west) was under construction. The Hines site was being dewatered and was excavated to a depth of approximately 25 feet bgs (Rollo & Ridley, 2009). The groundwater levels encountered in the borings and CPTs during the investigations are summarized in Table 1.

TABLE 1

Summary of Groundwater Depths Encountered During Field Exploration

Exploration Point	Date Measured	Groundwater Depth[1] (feet)	Groundwater Elevation (feet)
B-3	12/10/2015	20	83.4
B-4	12/10/2015	13.5	89.0
B-5	12/11/2015	13.5	89.7
B-6	12/10/2015	12.5	90.0
B-7	12/11/2015	13.5	89.3
CPT-1 (PPDT[2])	12/10/2015	20.9	81.6
CPT-2 (PPDT)	12/10/2015	18.1	83.9
CPT-5 (PPDT)	12/10/2015	27.5	75.9
CPT-6 (PPDT)	12/10/2015	26.0	76.5
CPT-7 (PPDT)	12/10/2015	24.0	78.5
CPT-8 (PPDT)	12/11/2015	22.6	79.7
CPT-9 (PPDT)	12/11/2015	21.0	81.4
CPT-11 (PPDT)	12/11/2015	21.4	81.2

Notes:
1. Boring groundwater depths may not represent stabilized levels and groundwater levels, may be influenced by dewatering of the nearby Hines development and may fluctuate due to seasonal rainfall.
2. PPDT = pore pressure dissipation test.

During our investigation in 2015, the groundwater appears to have been drawn down approximately 5 to 22 feet below historic groundwater levels likely due to drought conditions and the dewatering of the Hines site in 2015. Current groundwater readings are not available.

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We expect groundwater levels at the site to fluctuate considerably based on seasonal variations in rainfall.

5.0 SEISMIC AND GEOLOGIC HAZARDS

5.1 Regional Seismicity

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 7. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[4] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 2.

TABLE 2

Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
N. San Andreas - Peninsula	6	West	7.23
N. San Andreas (1906 event)	6	West	8.05
Monte Vista-Shannon	14	Southeast	6.50
San Gregorio Connected	18	West	7.50
Total Hayward	24	Northeast	7.00
Total Hayward-Rodgers Creek	24	Northeast	7.33
N. San Andreas - North Coast	35	Northwest	7.51
Total Calaveras	36	East	7.03
Mount Diablo Thrust	42	Northeast	6.70
Green Valley Connected	47	Northeast	6.80
N. San Andreas - Santa Cruz	50	Southeast	7.12

Figure 7 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

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

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on the Modified Mercalli (MM) scale (Figure 8) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, Mw, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a Mw 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 Mw 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 Mw of 6.9, approximately 69 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 be felt in the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 74 kilometers north of the site, with a Mw of 6.0.

The 2014 WGCEP at the U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by 2043. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 3.

TABLE 3

WGCEP (2014) Estimates of 30-Year Probability (2014-2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
N. San Andreas	22
Calaveras	26
San Gregorio	6

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5.2 Liquefaction and Associated Hazards

The site is in a seismically active area and will be subject to very strong shaking during a major earthquake on a nearby fault. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[5], lateral spreading[6], and cyclic densification[7]. Each of these conditions has been evaluated based on our literature review, field investigation and analyses, and is discussed in this section.

5.2.1 Liquefaction

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 on map titled, *State of California Seismic Hazard Zones, San Mateo Quadrangle, Santa Clara County prepared by the California Geologic Survey* (dated 17 August 2017), as shown on Figure 9. 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 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.

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5 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporally 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.

6 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.

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

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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. We assumed a peak ground acceleration (PGA_M) of 0.659g.

Layers of loose to medium dense saturated sand and silty sand, varying in thickness of up to approximately five feet, were encountered below the groundwater level from depths of approximately 5 to 69 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.

We estimate that up to one inch of liquefaction-induced settlement may occur throughout the site. Because the potentially liquefiable layers are discontinuous, we estimate that up to one inch of differential settlement may occur during an earthquake. If an excavation of 13 feet is made for the basement, as planned, the liquefaction-induced settlement is estimated to be up to one inch within the basement footprint.

5.2.2 Seismic Densification

Cyclic densification refers to seismically-induced differential compaction of non-saturated granular material (sand and gravel above the groundwater table) caused by earthquake vibrations. The borings and CPTs indicate that the materials above the water table are sufficiently dense or clayey, and therefore the potential for seismic densification is low.

5.2.3 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes.

The site is relatively flat and the potentially liquefiable soils are not continuous, hence, we preliminarily conclude widespread shear zones should not develop for significant lateral displacements to occur during a major earthquake. Therefore, lateral spreading is not likely to affect the site.

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5.3 Fault Rupture

Historically, ground surface ruptures closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. Therefore, we conclude the risk of fault offset through 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 that the risk of surficial ground deformation from faulting at the site is low.

5.4 Tsunami

Recent published maps (California Emergency Management Agency, 2009) indicate the project site is not within the tsunami inundation zone; therefore, we conclude the potential risk by inundation from tsunami to be low within the project site. However, the project civil engineer should evaluate the impact of sea level rise on the potential risk of inundation from a tsunami.

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, the proposed project is feasible provided the site conditions and geotechnical issues discussed below are properly addressed during the design and construction of the proposed buildings. The primary geotechnical issues include:

- adequate foundation support and settlement behavior of the structures
- the presence of near-surface fill and Bay Mud
- the presence of shallow groundwater
- the potential for liquefaction-induced settlement

These issues and their impact on the geotechnical aspects of the project are discussed in the following subsections.

6.1 Foundations and Settlement

Currently, a site grading plan is not available. Placement of new fill across the site and supporting buildings will increase the load on the Bay Mud, causing the Bay Mud to consolidate, which will result in settlement of the ground surface and the new structures.

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We judge the settlement under the weight of the new fill is complete. However, for every foot of new fill placed, the settlement of the Bay Mud will continue. Table 4 presents estimated settlement if new fill is placed to raise existing grades.

TABLE 4

Settlement Estimate from New Fill

Height of New Fill (feet)	Estimated Consolidation Settlement (inch)
1	1 to 2
2	1½ to 3½

The primary considerations related to the selection of the foundation systems of the proposed structures are the bearing capacity of the on-site soil and estimated total and differential settlements. The proposed at-grade building sites are susceptible to the following potential sources of settlement:

- consolidation of the Bay Mud under the weight of new building loads and/or new fill
- liquefaction-induced settlement.

The proposed development will consists of five-story podium structures with a one level basement. Structural loads are currently not available for the proposed structures. However, based on our experience with similar buildings, we anticipate buildings loads of approximately 1,250 psf for the proposed podium structures. Based on current development plans, the finished floor elevation of the basement will be 10 feet below the ground floor, corresponding to approximately Elevation 93 to 94 feet.

Assuming a two-foot thick basement slab and a 12-inch working pad, we estimate the basement excavation will extend approximately 13 feet below the ground surface (bgs), corresponding to approximately Elevation 90 feet. Based on available subsurface data and the bottom of Bay Mud contour map presented on Figure 6, the anticipated soil at the bottom of the proposed excavation will consists be a medium stiff to hard clay beneath Buildings 1 and 2 and Bay Mud beneath Buildings 3 and 4. The recommended foundation type will depend on the depth of excavation for the basement with respect to the bottom of Bay Mud elevation.

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For Buildings 1 and 2, we anticipate the bottom of excavation at Elevation 90 feet will extend through the Bay Mud; therefore, we conclude the structures may be supported on a mat foundation.

For Buildings 3 and 4, we anticipate the bottom of excavation at Elevation 90 feet will be above the bottom of Bay Mud elevation. Assuming a building load of 1,250 psf, we anticipate settlements on the order of four to five inches may occur. Therefore, we conclude the structures will need to be supported on deep foundations (such as Auger Cast Displacement Piles (ACDPs) or driven piles) or a mat foundation supported on ground improvement elements. An option for Buildings 3 and 4 is to overexcavate the bottom of excavation by approximately five feet (corresponding to Elevation 85 feet) to remove all the Bay Mud beneath the proposed buildings. If all the Bay Mud is removed and the overexcavation is backfilled with engineered fill or lean concrete, we conclude Buildings 3 and 4 may be supported on a mat foundation.

Potential foundation types for the proposed structures, including shallow and deep foundations, are discussed in the following subsections.

6.1.1 Mat Foundation

Where the basements extend below the Bay Mud, the soil exposed at the bottom of the excavation should generally consist of stiff to very stiff clay. The clay should be capable of supporting moderate foundation loads without large settlement; the removal of soil to accommodate the basement should compensate for a portion of the new building load. An excavation of 13 feet would provide a stress reduction of about 1,100 to 1,300 psf. Based on laboratory test data and the CPT data, the clay below the proposed excavation is normally to overconsolidated, with an overconsoidated ratio of 1 to 3.5. We estimate the building load will be approximately 1,250 psf; therefore, static settlement should be mainly recompression.

We estimated the settlement for the proposed mat foundation will be about one inch. We estimate post-construction differential settlement between columns may be on the order of ½ inch; this estimate does not include the rigidity of the foundation system, which would tend to reduce the differential. In addition, we estimate that during a major earthquake, there could be liquefaction-induced settlements of up to one inch.

6.1.2 Mat Foundations Supported on Ground Improvement

For structures where the bottom of excavation is above the bottom of Bay Mud elevation, a mat in combination with ground improvement may be considered. We considered several

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ground improvement methods to reduce the static settlement affecting the proposed structures, including Rammed Aggregate Piers (RAPs) and drilled displacement columns (DDCs) to improve the weak surficial soil. However, after discussions with a local contractor that design and installs RAPs, it was concluded that RAPs may be too difficult to install through the soft Bay Mud.

The purpose of the DDCs is to improve the weak soil and reduce the associated settlements by strengthening the soil matrix with a grid of shafts filled with controlled low-strength material (CLSM). A mat foundation can then be used on top of the DDCs. DDC systems are installed under design-build contracts by specialty contractors.

DDCs are constructed by using a displacement auger to create a shaft that is filled with CLSM injected under pressure as the displacement auger is withdrawn. Installation of DDCs produces minimal soil cuttings because the soil is displaced during column installation; because the soil is displaced, some densification occurs in the soil between the columns. Typically, DDCs are 16 to 24 inches in diameter. Because DDCs inject the CLSM under pressure, there is the potential for soil heave near the column. To eliminate the potential to damage nearby improvements, DDCs may need to be set back a horizontal distance from adjacent improvements. DDCs can also be designed to resist uplift loads by installing steel reinforcement in the columns before the CLSM has set.

Because the DDC systems are installed by specialty design-build contractors, we do not provide specific design recommendations or settlement estimates for these systems. However, for cost estimating purposes, we contacted a local design-build ground improvement contractor, to assist us in providing preliminary design estimates. Based on preliminary discussions with a local specialty contractor, a mat foundation supported on 16- to 18-inch DDCs installed to depths ranging from 25 to 30 feet bgs is estimated to have a preliminary allowable bearing capacity of 4,000 to 5,000 psf for dead plus live loads.

Based on our experience with sites with similar soil conditions, we anticipate static settlement of properly constructed footings or mat supported on DDC-improved soil will be limited to ½ to 1 inch under the weight of the building loads. The settlements should be confirmed by the design-build contractor. The settlements presented are preliminary. The design-build contractor should provide specific design recommendations and final settlement estimates for their system.

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If the ground improvement elements are relied upon for uplift resistance, the ground improvement specialty contractor should provide the uplift capacity.

In addition, we estimate that during a major earthquake, there could be liquefaction-induced settlements of up to one inch.

6.1.3 Deep Foundations

For structures where the bottom of excavation is above the bottom of Bay Mud elevation, deep foundations may also be considered. Based on the subsurface conditions at the project site, auger cast displacement piles (ACDPs) or driven concrete or steel piles would be appropriate. Driven piles may be considered provided noise and vibrations are acceptable at the site.

Because the fill is corrosive and the Bay Mud is severely corrosive, piles will require protection from corrosion.

6.1.3.1 Auger-Cast Displacement Piles

ACDPs are a low-vibration, low-noise, deep foundation option. These pile types are designed and installed by specialty contractors. ACDPs are installed by drilling to the required depth with a hollow-stem auger. The auger has a reverse tread, which results in displacement and densification of the surrounding soil and results in little to no spoils. When the auger reaches the required depth, cement grout or concrete is injected through the bottom of the hollow-stem auger. Grout or concrete is injected continuously as the auger, still rotating in a forward direction, is slowly withdrawn, replacing the displaced soil. While the grout is still fluid, a steel reinforcing cage is inserted into the shaft.

Piles should gain support primarily in side resistance (friction) below the Bay Mud. Uplift capacities will be limited to the embedment below the Bay Mud. Properly constructed ACDPs gaining support below the compressible layers should have a total settlement less than one inch, with less than ½ inch of differential settlements between columns, under static conditions. Most of these static settlements are expected to occur during construction.

ACDP piles are designed and installed by specialty contractors. If these pile types are used, they will need to be tested to confirm the design values.

6.1.3.2 Driven Piles

If there are no limitations to noise and vibration, a driven pile could be used for support of the structures. Driven pile types could consist of precast, prestressed, (PCPS) concrete piles, steel

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H-piles or steel pipe piles. Based on our experience with similar subsurface conditions, we conclude that PCPS concrete piles are the most appropriate driven pile type for the project. To prevent damage to concrete piles from debris in the fill, predrilling through the fill should be performed, which would produce spoils.

During our field investigation, dense to very dense sand layers of varying thickness, density and fines content were encountered at various depths beneath the Bay Mud; these layers do not appear to be continuous across the site and therefore, we conclude these should not be relied on for end bearing at this time. An indicator pile program should be performed to provide additional information regarding, ease of installation, final pile lengths and capacities. Details of an indicator pile program are presented in Section 7.4.2.3.

Most of the settlement of piles gaining support in skin friction in the soil beneath the Bay Mud is anticipated to occur during construction. We estimate differential settlement will be less than ½ inch between adjacent columns supported on new piles.

6.2 Groundwater Consideration

At the time of our field investigation in December 2015, the Hines development across South Delaware Street was being dewatered. According to the geotechnical investigation report for the Hines Development (Rollo & Ridley, 2009), the minimum recommended drawdown depth of the groundwater during construction was approximately 19 to 21 feet bgs.

Current groundwater readings are not available. We anticipate that when Hines completed construction of their buildings in 2016/2017, the dewatering wells were shut off and the groundwater levels returned to the historic groundwater levels of approximately 4 to 8 feet bgs, corresponding to Elevation 97.5 to 95.5 feet. Therefore, on the basis of our knowledge of the historic groundwater conditions in the area, we conclude a high groundwater elevation of Elevation 98 feet should be used in design. If the weight of the building and mat foundation is not sufficient to resist uplift then tiedown anchors may be required to resist the anticipated uplift pressures.

6.3 Floor Slabs

Because liquefaction induced settlement on the order of one-inch are estimated to occur during a major earthquake and slabs will be near or below the design groundwater table, the building slabs should be designed to span between pile caps and grade beams where DDC or ACIP

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piles are used. The structural slab or mat should be waterproofed and checked for hydrostatic uplift.

6.4 Dewatering

To construct the basement of the buildings, the groundwater will need to be temporarily lowered to a depth of at least three feet below the bottom of the planned excavation. The method of dewatering will depend to an extent on the method of shoring. The dewatered level should be maintained at that depth until sufficient building weight is available to resist the hydrostatic uplift pressure of the groundwater at its design elevation.

Based on our experience with similar developments, 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. 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. In addition to the wells, the working pad as recommended in Sections 7.1 and 7.7, can be used as a temporary drainage blanket to assist with the dewatering of 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.

Groundwater seepage through the fill may be high, though flow through the Bay Mud and native soil will be slow. Where excavations extend to or below the Bay Mud, the contractor should be prepared to manage the water in the excavation. Localized sumps and pumps can be used.

6.5 Shoring Considerations

During excavation of the basements, the adjacent property and streets should be supported by temporary shoring. There are several key considerations in selecting a suitable shoring system. Those we consider to be primary concerns are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures

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- proper construction of the shoring system to reduce the potential for ground movement
- cost.

Construction of the basements will require an excavation of about 13 feet below the adjacent sidewalk grades. If areas where there is insufficient space to slope the sides of the excavation, shoring will be required. The shoring design should allow for over excavation of at least 12 inches across the footprint of each of the buildings to create a working pad for the mat foundation. During excavation for the proposed basement level, shoring will be required to laterally restrain the sides of the excavation and limit the movement of adjacent improvements, such as public streets and sidewalks.

Based on our experience on projects with similar excavation depths, soldier pile and lagging systems may be the most economical shoring system for the excavation for this project. Soldier piles should be placed in predrilled holes, which will be backfilled with concrete or installed with a soil-cement mixing drill rig. Wood lagging should be placed between the soldier beams as the excavation proceeds. Drilling of the shafts for the soldier piles may require casing and/or the use of drilling mud to prevent caving of any sand layers that are present.

For excavations on the order of 13 feet deep, the shoring typically can be designed as a cantilever system or with a shoring system with lateral restraint with either grouted tiebacks or internal bracing. Tiebacks will require encroachment permits from adjacent property owners. Tiebacks may need to be designed to gain support in the fill above the Bay Mud. Tiebacks on the street sides of the excavation should avoid underground utilities in the street. Minor deflections of the ground surface and adjacent structures should be expected with a soldier pile and lagging system. The amount of movement and distress to adjacent improvements will depend on the rigidity of the shoring and the workmanship of the contractor. If cohesionless layers are encountered, some caving may occur while lagging boards are installed. To reduce movements and caving, it may be necessary to limit the unsupported height of the excavation to the height of the lagging boards.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle and move. The magnitude of shoring movements and resulting settlements of the ground surface behind shoring walls are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Clough and O'Rourke (1990) summarized the measured settlements adjacent to excavations in sand and concluded that the settlements varied from

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0.1 to 0.3 percent of the excavation depth. The data also show the settlements at some sites where the excavations were shored with a soldier-pile-and-lagging system were higher than these values. Therefore, for an excavation depth of up to 13 feet, we estimate settlement immediately behind the shoring wall could be on the order of 0.5 to 1 inches. These settlements assume the quality of construction will meet or exceed that considered standard in the construction industry. The settlement should decrease with distance from the wall, and should be small at a distance twice the excavation depth.

6.6 Excavation and Monitoring

The soil to be excavated from the site consists of materials that can be excavated with conventional earthmoving equipment such as loaders and backhoes, except where foundations and slabs of existing buildings are encountered. Removal of these may require the use of jackhammers or hoe-rams. Excavations resulting from the removal of foundations, slabs and underground utilities that extend below the bottom of the proposed foundation/floor level should be cleaned of any loose soil/debris and backfilled with lean concrete or properly compacted fill.

If earthwork is performed in wet weather conditions, it may be difficult to compact the soil; it may need to be aerated during dry weather. Because of proximity of the ground water level, the soil subgrade will likely be at or near saturation and light grading equipment may be needed to avoid damaging the subgrade.

A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of the adjacent ground surface during excavation.

6.7 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 two soil samples from the site to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Appendix D and summarized in Table 5.

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TABLE 5

Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfate (ppm)	Resistivity (ohms-cm)	Redox (mV)	Chloride (ppm)
B-2	3	7.79	240	770	330	300
B-3	1 to 5	7.81	63	660	330	500

N.D. = None Detected

Based upon resistivity measurements, the fill is corrosive and the Bay Mud is severely corrosive to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. The results of the chemical analysis indicate that the soil could be detrimental to reinforced concrete and cement mortar coated steel. To protect reinforcing steel from corrosion, adequate coverage should be provided as required by the building code.

A brief evaluation of the corrosivity of the soil samples is presented in Appendix D. For more detailed recommendations regarding the corrosion protection of buried metals and concrete, a licensed corrosion consultant should be retained.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation

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 obstructions are encountered. These may require hoe rams or jackhammers to remove. Any portions of existing buried foundations that could interfere with the proposed foundations or basement walls should be removed.

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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 or walls. If utilities are abandoned in-place, they should be completely filled with flowable cement grout over their entire length within the property limits. Existing utility lines, where encountered, should be addressed on a case-by-case basis.

7.1.1 Mat Foundation and Basement Floor Subgrade Preparation

Because the excavation for the basements 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 the subgrade elevation and that the final excavations 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 if it is susceptible to disturbance, we recommend overexcavating the site and backfilling with drain rock on which the mat is constructed. This layer of crushed rock can also be used as part of a dewatering system (further discussed in Section 7.6).

For the working pad, we anticipate an overexcavation of about 12 inches will suffice if used in conjunction with a woven reinforcing fabric (geotextile), such as Mirafi 500x. After placing the reinforcing fabric on the exposed subgrade, the overexcavation should be backfilled with clean one-inch minus crushed rock or similar material. 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 or structure slab constructed.

Because the proposed basement foundation will be below the groundwater level, waterproofing the base of the foundation, slab and basement walls is recommended. The waterproofing should be placed directly on the crushed rock or on a mud slab (thin layer of lean concrete) and be covered by a second 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 or structural slab. 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.

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The soil subgrade should be free of standing water, debris, and disturbed materials prior to placing the reinforcing fabric and crushed rock. If loose 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.

If any Bay Mud or loose to medium dense sand is exposed at the mat subgrade where ground improvement are not used, it should be overexcavated and replaced with either lean concrete or engineered fill.

7.1.2 At-Grade Improvements

We recommend new sidewalks and concrete flatwork (in non-vehicular traffic area) be underlain by at least four inches of Class 2 aggregate base material (or the minimum thickness per City of San Mateo Standards) that has been compacted to at least 95 percent relative compaction.

The majority of the fill encountered was sand and gravel with varying amount and types of fines and should meet the requirements for select fill. However, in a few locations throughout the site (at Boring B-1 and B-6), a dark brown to black clay with moderate expansion potential was encountered as fill. If the moderately expansive dark clay is encountered during grading and subgrade preparation of at-grade improvements, we recommend that up to 12 inches of the material should be removed and replaced with select fill.

Select 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[8] and be approved by Langan. In addition, the select fill should contain at least 20 percent fines (particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate beneath slabs. Select 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 should be rolled to a firm, non-yielding surface. If the compacted subgrade is disturbed during utility trench or foundation excavations, the subgrade should be re-rolled to provide a smooth, firm surface for concrete slab support.

8 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.

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Prior to placement of select fill, the onsite soil exposed by stripping should be scarified to a depth of at least 8 inches, moisture-conditioned to at least optimum moisture content, and compacted to 90 percent relative compaction[9]. The soil subgrade should be kept moist until it is covered by select fill.

Where utility trenches backfilled with sand or gravel cross planter areas and pass below asphalt or concrete pavements, an impermeable plug consisting of native clay or lean concrete at least five feet in length, should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the pavements. This trapped water can cause softening of subgrade soil beneath pavements.

Where used, sand containing less than 10 percent fines (particles passing the No. 200 sieve) should also be compacted to at least 95 percent relative compaction. Samples of on-site and proposed import fill materials should be submitted to Langan for approval at least three business days prior to use at the site.

7.2 Mat Foundation

We conclude Buildings 1 and 2 can be supported on a mat foundation supported on native stiff to hard clay. To design the mat using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 15 kips per cubic foot (kcf). The modulus value is 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 modulus value is appropriate. The modulus is applicable for localized dead plus live loads up to 2,500 psf.

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. A uniform pressure of 600 psf may be used to compute passive resistance against the vertical faces of the mat. Frictional resistance should be computed using a base friction coefficient of 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.

[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.

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7.3 Mat Foundation with Ground Improvement

As discussed in Section 6.1.2, for structures where the bottom of excavation is above the bottom of Bay Mud elevation, a mat foundation in combination with ground improvement may be considered. Typically, DDCs are designed by specialty design-build contractor; therefore, we cannot provide specific design recommendations or settlement estimates for these systems. Our geotechnical report should be provided to the design-build contractor to provide final foundation design plans; we should be retained to provide technical input and review of the design prior to construction.

On the basis of our discussions with a local design-build ground improvement contractor we understand the mat may be designed using a modulus of subgrade reaction of 96 kcf; however, this should be verified by the design-build contractor, who will estimate the corresponding settlement once building loads are available.

Lateral forces can be resisted by a combination of friction along the base of the mat, and passive resistance against the vertical faces of the mat. To provide a uniform distribution of the foundation loads, a load transfer platform (LTP), consisting of 12 inches of compacted open-graded angular crushed rock should be placed above the ground improvement elements. The LTP should be designed by the design/build contractor. Alternatively, the soil between the elements can be neglected and the foundation designed to span between elements. The ground improvement contractor selected for the project should confirm that these values can be obtained and which approach is taken.

To calculate the passive resistance against the vertical faces of the mat, a uniform pressure of 600 psf may be used to compute passive resistance against the vertical faces of the mat. The value for passive pressure includes a factor of safety of 1.5. Frictional resistance against the base of the mat should be calculated based on parameters provided by the design-build subcontractor.

The design capacity of the DDCs should be verified by at least one load test in compression and one test in tension, if uplift elements are used. The test column locations should be selected by the geotechnical engineer and approved by the structural engineer. The compression load tests should be performed in accordance with current edition of ASTM D1143, Standard Test Method for Piles Under Static Axial Compressive Load, and the tension tests should be performed in accordance with ASTM D3689. Equipment used for the test (load frame, jacks, and reaction piles) should be capable of applying at least 2 times the

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allowable dead plus live design load and at least 1.5 times the total load. The Davisson Method or other accepted criteria per the 2016 California Building Code should be used to interpret the ultimate capacities of the DDCs.

7.4 Deep Foundations

For structures where the bottom of excavation is above the bottom of Bay Mud elevation, the proposed structure may be supported on a deep foundation system consisting of ACDPs or driven concrete piles. The piles will primarily gain capacity from skin friction in soils below the Bay Mud.

7.4.1 Auger-Cast Displacement Piles (ACDP)

ACDP are installed by design-build or specialty contractors. The vertical and lateral capacities presented in the following subsections for ACDP are preliminary and may be used in pricing and estimating. Final design capacities should be determined by the selected specialty/design-build contractor and verified by a test program. ACDPs can range in diameter; however, 16- and 18-diameter ACDPs are typical.

7.4.1.1 Axial Capacity

Table 6 presents preliminary design axial capacities for use in pricing and estimating. The preliminary allowable compressive capacities and lengths are based on our discussions with contractors with experience installing these pile types in the Bay Area. Typically, the maximum lengths of ACDPs are about 60 to 70 feet. Final design axial pile capacities for ACDPs should be determined by the design/build contractors after they have been selected.

TABLE 6

Preliminary Axial Pile Capacities for ACDPs

Pile Diameter	Approximate Average Length (feet)	Ultimate Compressive Axial Capacity (kips)	Allowable Dead plus Live Load Compressive Axial Capacity[1] (kips)	Allowable Uplift Capacity (kips)
16-inch-diameter	60 to 65	200 to 300	100 to 150	100 to 150
18-inch-diameter	60 to 65	300 to 400	150 to 200	150 to 200

Note:
1. The allowable dead plus live load axial capacities (compressive and uplift) include a factor of safety (FS) of at least
2. The allowable dead plus live load capacities may be increased by one-third for total loads, including wind or seismic forces.

ACDP design capacities should be verified by a test program. We recommend at least one compression and one tension pile load test be performed per 2016 CBC Section 1810.3.3.1.2. Pile should be spaced at least three pile diameters center-to-center to prevent vertical capacity reductions due to pile interaction effects; the outer auger-tip diameter should be used when determining the pile spacing for ACDP piles. The piles should also be designed to account for the presence of corrosive soil; a corrosion consultant should be retained to provide specific recommendations regarding the long term corrosion protection of pile elements.

7.4.1.2 Lateral Load Resistance

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

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

We evaluated the preliminary lateral capacity of a 16-inch and 18-inch diameter ACDP piles for ½-inch deflection at the pile head below the basement level. For a free-head condition, the pile top is free to move laterally and rotate. For a fixed-head condition, the pile top is restrained from rotating but free to move laterally. Preliminary deflection and moment profiles for a single 16- and 18-inch diameter ACDP pile are presented on Figures 10 through 13. Final design lateral pile capacities for ACDPs should be determined by the design/build contractors.

The lateral capacities are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown in Table 7. However, the maximum moment for a single pile with an unfactored load should be used to check the design of individual piles in a group. The reduction factors are based on a minimum center-to-center spacing of three pile widths. Where piles are spaced at least six pile diameters in all directions, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known.

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TABLE 7

Lateral Group Reduction Factors

Number of Piles within Pile Cap	Lateral Group Reduction Factor
2	0.9
3 to 5	0.8
≥6	0.7

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. To calculate the passive resistance against the vertical faces of pile caps and grade beams, we recommend a uniform pressure of 600 psf. This value has a factor of safety of about 1.5. The upper foot should be ignored unless it is confined by a slab.

7.4.1.3 ACDP Construction Considerations

We recommend that before production ACDP pile lengths are selected, indicator piles be installed to: 1) evaluate predrilling requirements, if any, and 2) estimate production pile lengths. We recommend a minimum of 10 indicator piles be installed. We expect the indicator piles can be used as production piles if installed in the proper location and are not damaged during installation or testing. If indicator piles are to be abandoned following the indicator program, then the indicator piles should be located at least seven pile diameters (center-to-center) from production pile locations. Indicator piles should be installed with the same equipment and using the same procedure, including predrilling depth and predrill auger diameter, that will be used for production piles.

7.4.1.4 Pile Load Test Program

We recommend load tests of the ACDP piles be performed to confirm the axial compression and tensile pile capacities. We recommend a minimum of one compression and one uplift load tests be performed for each proposed production pile installation methodology (i.e. rig type, predrilling depth and diameter, pile length, etc.) The test pile locations should be selected by the geotechnical engineer and approved by the structural engineer. The compression load tests should be performed in accordance with the current edition of ASTM D1143, Standard Test Method for Piles Under Static Axial Compressive Load, and the tension tests should be performed in accordance with ASTM D3689. Equipment used for the test (load frame, jacks, and reaction piles) should be capable of applying at least 2 times the allowable dead plus live

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design load and at least 1.5 times the total load. The Davisson Method or other accepted criteria per the 2016 California Building Code should be used to interpret the ultimate capacities of the piles.

7.4.1.5 Pile Installation Work Plan

A work plan describing the proposed ACDP installation equipment and methodology, including, but not limited to, predrilling depth, diameter of auger used for predrilling, pile diameter and pile length, as well as the proposed indicator pile location, pile load test set-up and procedure should be submitted to Langan for review and approval at least five working days prior to the indicator pile and pile load test programs. The work plan should include a site plan showing the locations of indicator test and reaction piles relative to permanent foundation elements and a drawing showing the layout of the load test set up. Following the completion of pile load tests, the Geotechnical Engineer will require at least three working days to review and evaluate the load test results and propose recommendations for production pile installation.

Additional pile load tests will be required if, during production pile installation, the equipment or installation procedure deviates from the approved work plan and indicator pile load test program.

7.4.2 Driven Piles

Based on our experience with similar subsurface conditions, we conclude that precast, prestressed, (PCPS) concrete piles are the most appropriate driven pile type for the project.

7.4.2.1 Axial Capacity

Piles should gain support primarily through skin friction below the fill and Bay Mud layers. Allowable dead and live load capacities of piles versus tip elevation are shown on Figure 14. The capacity may be increased by one-third for total loads, including wind or seismic forces.

Piles will develop resistance to uplift loads through skin friction along their perimeter surfaces. The allowable capacities presented on Figure 14 may be used for temporary uplift loads. For permanent uplift loads, use 80 percent of the indicated capacities on Figure 14.

Piles should be spaced no closer than three pile-widths on centers to avoid reductions to the axial capacities due to group effects.

7.4.2.2 Lateral Load Resistance

Lateral load resistance can be mobilized by the individual piles in combination with other foundation elements embedded below the ground surface. Lateral resistance of piles will depend on the stiffness of the pile, the strength of the surrounding soil, the allowable defection of the pile top, and the bending moment capacity of the pile.

We evaluated the lateral capacity of 14-inch concrete piles for ½-inch deflection at the pile head. Deflection and moment profiles for a single pile are presented on Figures 15 and 16.

The lateral capacities on Figures 15 and 16 are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown in Table 7. However, the maximum moment for a single pile with an unfactored load should be used to check the design of individual piles in a group. The reduction factors are based on a minimum center-to-center spacing of three pile widths. Where piles are spaced at least six pile diameters in all directions, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known.

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. To calculate the passive resistance against the vertical faces of pile caps and grade beams, we recommend a uniform pressure of 600 psf. This value has a factor of safety of about 1.5. The upper foot should be ignored unless it is confined by a slab.

7.4.2.3 Indicator Pile Program

Before production piles are cast, we recommend 10 indicator piles be driven to provide blow count data to correlate with information obtained from the test borings and estimate production pile lengths. Indicator piles should be located at production pile locations selected by the geotechnical engineer and approved by the structural engineer. They should be driven with the same equipment that will be used to drive the production piles. We recommend casting the indicator piles at least 10 feet longer than the lengths determined from Figure 14. Cutoff lengths of up to 25 feet should be anticipated during the indicator program.

During the installation of all indicator piles, we recommend using a Pile Driving Analyzer (PDA) to evaluate pile stresses during driving and soil skin friction and end bearing. When used in conjunction with the Case Pile Wave Analysis Program (CAPWAP), the PDA data can be used to:

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- verify the hammer selected is appropriate to drive the piles to the desired tip elevation without damaging the pile

- estimate the ultimate capacity of the piles (assuming the piles can be retapped at least four days after driving).

A minimum of 10 piles should be retapped at least four days after the initial drive. A CAPWAP analysis should be performed on a representative blow near the end of the initial drive and during the beginning of restrike.

The PDA should be operated by experienced and qualified personnel. If the results indicate driving stresses (tension or compression) could damage the piles, the PDA operator should immediately notify the contractor and geotechnical engineer.

Determination of driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length. Special consideration should be given to selecting a hammer that can deliver enough energy to the tip of the piles to drive them efficiently without damaging them. We recommend using a hammer with a maximum rated energy between 60,000 to 90,000 foot-pounds per blow. Each pile should be driven to the design tip elevation without interruptions, unless it meets practical refusal. For planning purposes, we recommend using a refusal blow-count of 50 blows per foot. The refusal blow count may be modified after the indicator pile installation.

Pile driving will cause vibrations on adjacent sites. These vibrations can cause settlement of the fill materials surrounding the site or could affect nearby improvements. We also recommend monitoring the vibration of critical structures that are close to the pile driving activities. In addition, a thorough crack survey of adjacent buildings should be performed prior to the start of pile installation and after completion to check if pile-driving activities have any effects on adjacent structures.

Buried rubble may be encountered in the fill. If piles are driven before the basement excavations are made, it may be necessary to predrill pile locations to a depth of 10 feet below existing grade to facilitate pile installation through the fill. Where rubble is encountered, predrilling will allow the pile to be driven with no or less damage, and will help the contractor to maintain close alignment of the tops of the piles. Predrilling should be performed as part of the indicator pile program. The auger used for predrilling should have a diameter no greater than the pile width.

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7.5 Basement Walls

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 ground motion level to compute the seismic pressure increment. Basement walls should be designed for the more critical loading condition of static or seismic conditions using the equivalent fluid weights and pressures presented in Table 8.

TABLE 8
Basement Wall Design Earth Pressures
(Drained Conditions)

Condition	Static Conditions		Seismic Conditions[1]
	Unrestrained Walls (Active)	Restrained Walls (At rest)	Total Pressure – Active Plus Seismic Pressure Increment
Above Groundwater	40 pcf	60 pcf	65 pcf
Below Groundwater	80 pcf	90 pcf	90 pcf

Notes:
1. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.

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 an equivalent fluid weight of 250 pcf in the fill and a uniform pressure of 600 psf in the Bay Mud. These values include a factor of safety of about 1.5. The structural engineer should check the structural capacity of the walls and the amount of movement necessary to develop the passive pressure. We can provide passive mobilization curves, if needed to estimate the amount of wall movement for a given passive pressure.

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The lateral earth pressures given assume the walls are properly backdrained above the design groundwater table to prevent the buildup of hydrostatic pressure. If the walls are not drained, they should be designed using the below groundwater earth pressures presented in Table 8 to account for 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 a perforated PVC collector pipe at the design groundwater elevation (Elevation 98 feet). The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material and should be sloped to drain into an appropriate outlet. 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.

If backfill is required behind basement walls, the walls should be braced or hand-compaction equipment used to prevent unwanted surcharges on the walls.

7.6 Shoring Design

To construct below-grade walls, the site may be open cut and/or temporarily shored. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with CAL-OSHA standards (29 CFR Part 1926). It is the responsibility of the contractor to determine the safe excavation slopes; however, we recommend temporary cuts be no steeper than 1.5:1 (horizontal:vertical). Where space does not permit a sloped excavation and where excavations extend below the fill into the Bay Mud, shoring will be required.

For a cantilevered soldier-pile-and-lagging shoring system, we recommend the system be designed to resist active pressures using an equivalent fluid weight of 40 pcf for above the design groundwater table and a uniform pressure of 600 psf for below the design groundwater table. The passive pressures presented on Figure 17 may be used for Buildings 1 and 2 and the passive pressure presented on Figure 18 may be used for Buildings 3 and 4. The shoring should be designed to limit ground deformations to less than one inch.

Tie-back or braced soldier piles and lagging shoring systems for Buildings 1 and 2 should be designed to resist the pressures presented on Figure 17. Tied-back or braced soldier piles and lagging shoring systems for Buildings 3 and 4 should be designed to resist the pressures presented on Figure 18.

The soldier piles should extend below the excavation bottom a minimum of three feet and be sufficient to achieve lateral stability and resist the downward loading of the tiebacks. Recommendations for computing penetration depth of soldier piles are presented in Section 7.6.3.

If traffic occurs within 10 feet of the shoring depth, a uniform surcharge load of 100 psf should be added to the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed 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 designed by a licensed civil engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring engineer should check for basal stability. They should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of the adjacent ground surface during excavation.

7.6.1 Tieback Design Criteria and Installation Procedure

Temporary tiebacks may be used to restrain the shoring. The vertical load from the temporary tiebacks should be accounted for in the design. Design criteria for tiebacks are presented on Figures 17 and 18.

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 should have a

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minimum unbonded length of 15 feet. All tiebacks should have a minimum bonded length of 15 feet and spaced at least four feet on center. 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, and workmanship. The existing sandy soils may cave, therefore, solid flight augers 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 skin friction values presented on Figures 17 and 18. These values include a factor of safety of about 1.5. Higher skin friction values may be used if confirmed with pre-production performance 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 retaining systems. Determination of the tieback length should be based on the contractor's familiarity with his installation method. The computed bond length should be confirmed by a performance- and proof-testing program under the observation of an engineer experienced in this type of work. Replacement tiebacks should be installed for tiebacks that fail the load test.

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. Recommendations for tieback testing are presented in Section 7.5.2. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

7.6.2 Tieback Testing

We should observe tieback testing. 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. The remaining tiebacks should be confirmed by proof tests also to at least 1.25 times the design load.

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The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is 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.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the 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 10 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 less than 0.08 inch movement between six and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that failed to meet the first criterion will be assigned a reduced capacity.

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 contractor should replace the tiebacks at no additional cost to the owner.

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7.6.3 Penetration Depth of Soldier Piles

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient axial capacity to support the vertical load component of the tiebacks and the vertical load acting on the piles, if any. To compute the axial capacity of the piles, we recommend using an allowable friction of 1,000 psf on the perimeter of the piles below the Bay Mud and excavation level.

7.7 Dewatering

As previously discussed, the water table within the site should be drawn down to three feet below the bottom of the excavation during construction. If dewatering wells are installed within the excavation, the wells should be properly sealed through the floor slabs upon abandonment to reduce the potential for water leakage.

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.

As discussed in Sections 6.4, the crushed rock working pad can be used as part of the dewatering system 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.

7.8 Tiedown Anchors

If the weight of a building is not sufficient to resist the hydrostatic uplift loads or the mat cannot resist the uplift pressure between columns, tiedown anchors should be installed. Tiedowns typically consist of relatively small-diameter, drilled, grout-filled shafts with steel bars or tendons embedded in the grout. The tiedowns develop their uplift resistance from friction between the perimeter of the shaft and the surrounding soil.

Tiedowns should be spaced at least four shaft diameters apart or a minimum center-to-center spacing of four feet, whichever is greater. Because specialty contractors who install tiedowns

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use different installation procedures, the uplift capacity of the tiedowns will vary with the procedure. For planning purposes, however, we recommend using an allowable friction of 1,000 psf for post-grouted tiedowns installed in the native stiff clays; this value includes a factor of safety of 2.0 for permanent uplift loads (i.e. hydrostatic uplift). Higher values can be obtained depending upon the installation techniques employed by the contractor and the results of pullout tests.

Special attention should be given to waterproofing the connections between the tiedowns and the mat. Because the 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 auger-cast system or be prepared to case the holes to prevent caving. High strength bars or strands may be used as tensile reinforcement in the anchors. For stressing, the steel bars and strands should have at least 10 and 15 feet of free length, respectively. After testing, tiedowns should be locked-off at 10 percent of the design load or higher, if required by the structural engineer to limit deformation of the tiedown under the hydrostatic loading.

The bond length should be at least 15 feet. The design capacity of the tiedowns for permanent 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 2.0 times the design load. The remainder should be proof tested to 1.5 times the design load. The test procedure and acceptance criteria described in Section 7.6 for tieback testing should also be used for tiedowns. Replacement tiedowns should be provided, as directed by the structural engineer, for tiedowns that fail the test. All tiedowns should be locked off. The lock-off load and allowable amount of deformation after the tiedown is locked off should be determined by the structural engineer.

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7.9 Seismic Design

For seismic design in accordance with the provisions of 2016 California Building Code/ASCE 7-10, we recommend the following:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.856g and 0.864g, respectively
- Site Class D
- Site Coefficients F_A and F_V of 1.0 and 1.5
- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.856g and 1.297g, respectively
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.238g and 0.864g, respectively
- Peak ground acceleration, PGA_M of 0.732g

7.10 Asphalt Pavements

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of on-site soil. On the basis of the laboratory test results on this soil, we selected an R-value of 5 for design.

For our calculations, we assumed a Traffic Index (TI) of 4 for automobile parking areas with occasional trucks, and 5 and 6 for driveways and truck-use areas; these TIs should be confirmed by the project civil engineer. Table 9 presents our recommendations for asphalt pavement sections.

TABLE 9
Pavement Section Design

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4	2.5	8
5	3.5	9
6	4	12

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

To reduce the potential for irrigation water entering the pavement section, vertical curbs adjacent to landscaped areas should extend through any aggregate base and at least six inches into the underlying soil. In heavily watered areas, such as lawns, it may also be necessary to install a subdrain behind the curb to intercept excess irrigation water.

7.11 Utilities

Seismically-induced settlements of up to 1 inch with differential settlement of ½ inch over a short distance could be expected outside the basement footprint. Where utilities enter and exit the building and differential settlement are not tolerable flexible connections which allow for the anticipated differential movement should be used.

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

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

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.

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7.12 Construction Monitoring

Survey points should be installed on the adjacent streets and improvements that are within 100 feet of the proposed excavation. These points should be used to monitor the vertical and horizontal movements of the shoring and these improvements. These points should be selected with the help of the geotechnical engineer, so they can provide the most value to the project. The survey 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 shoring work at the site
- after installing cutoff wall elements
- weekly during excavation work
- after the excavation reaches the planned excavation level
- every week until the street-level floor slab is constructed.

In addition, the conditions of existing buildings within 100 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction. A thorough crack survey of the adjacent buildings, especially those surrounding the proposed excavation should be performed prior to the start of construction and immediately after its completion.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, we should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, we should observe the installation of the shallow or deep foundations and preparation of the building pad subgrade. We should also observe the subgrade preparation and any fill placement and perform field density tests to check that adequate moisture conditioning and fill compaction has been achieved beneath proposed sidewalks and pavement areas. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

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9.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of engineering studies and our interpretations of the existing geotechnical conditions. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan should be notified so that supplemental recommendations can be developed. Our scope of services relates solely to the geotechnical aspects of the project and does not address environmental concerns.

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FIGURES

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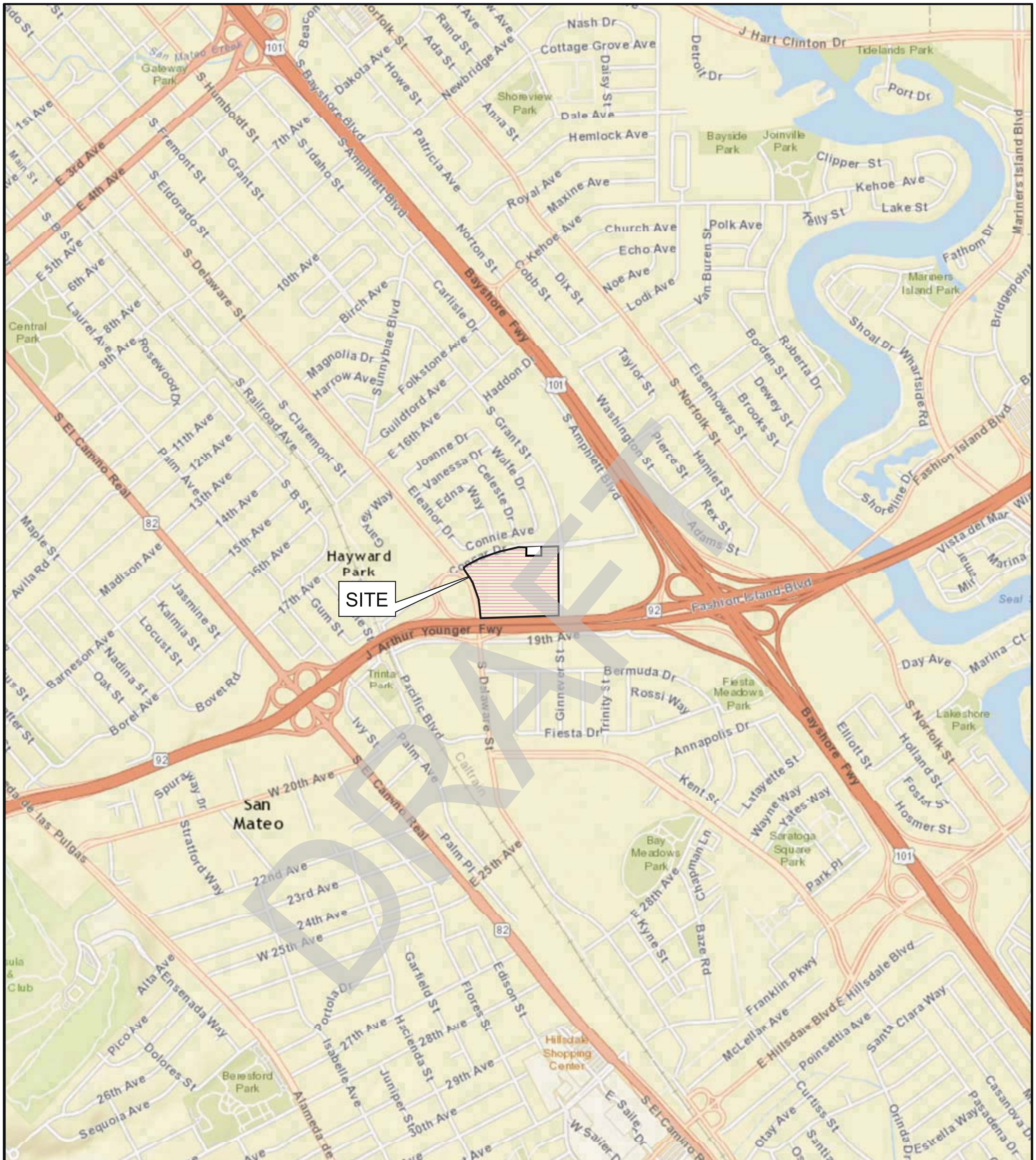

NOTES:

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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PASSAGE AT SAN MATEO
San Mateo, California

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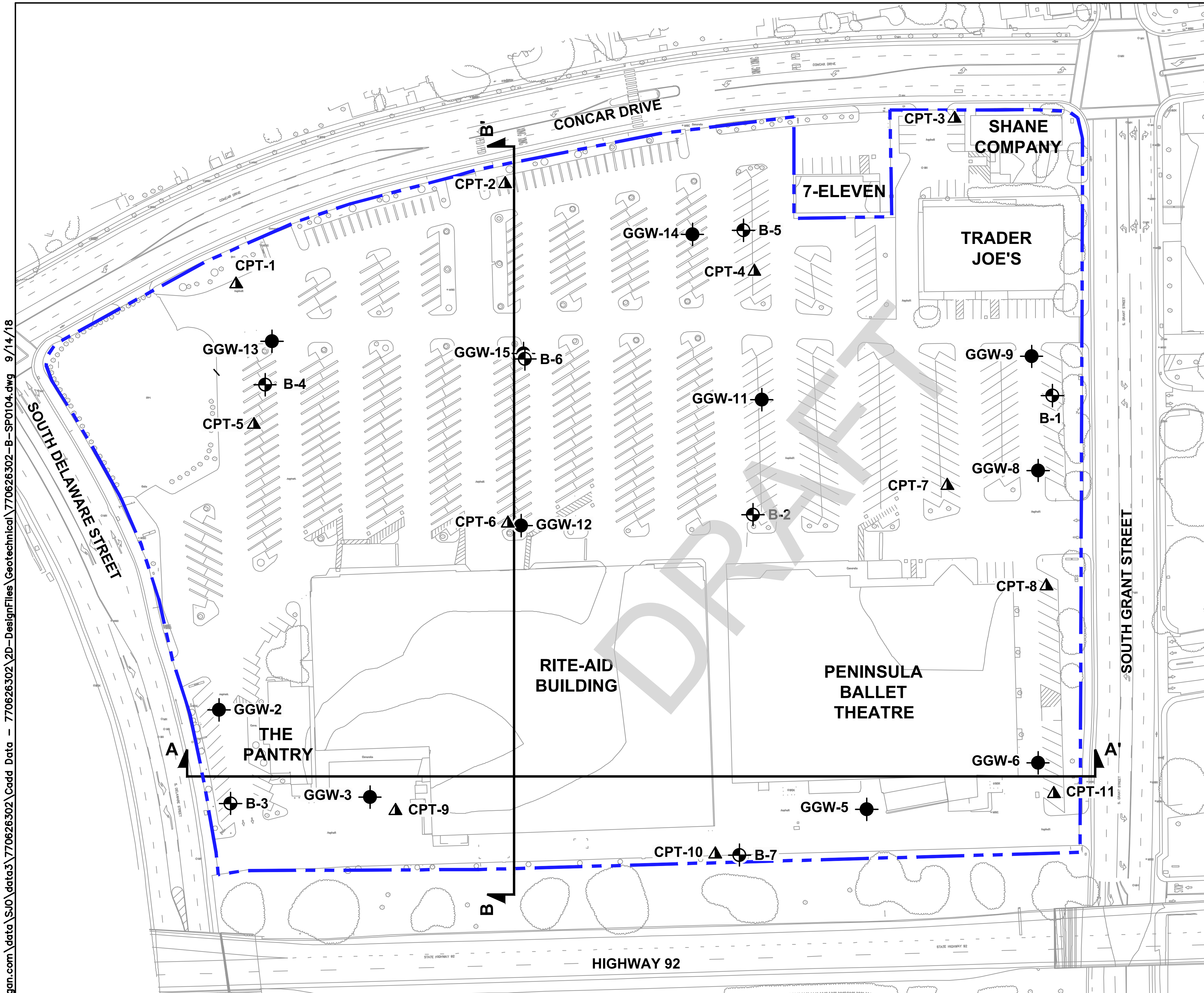
SITE LOCATION MAP

Date 08/15/18	Project No. 770626302	Figure 1



EXPLANATION

B-1 — Approximate location of boring by Langan, December 2015

CPT-1 — Approximate location of cone penetration test by Langan, December 2015

GGW-2 — Approximate location of environmental boring by Erler & Kalinowski, Inc., January 2016

Site boundary

A — A' Idealized subsurface profile location

0 — 100 Feet

Approximate scale

Reference: Base map from an electronic file titled "CONCAR_ALTA-CHAN USE ONLY.dwg," by BKF, dated 12/29/15 and "Soil Sample Locations," by Erler & Kalinowski, Inc., dated January 2016.

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PASSAGE AT SAN MATEO
San Mateo, California

EXISTING SITE PLAN

Date 09/14/18	Project No. 770626302	Figure 2

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Reference: Base map from a drawing titled "Site Plan," Sheet A0-1.0, by MVE Partners, dated 08/31/18.

EXPLANATION

- B-1 — Approximate location of boring by Langan Treadwell Rollo, December 2015
- CPT-1 — Approximate location of cone penetration test by Langan Treadwell Rollo, December 2015
- GGW-2 — Approximate location of environmental boring by Erler & Kalinowski, Inc., January 2016
- Site boundary
- A A' — Idealized subsurface profile location

PASSAGE AT SAN MATEO
San Mateo, California

SITE PLAN WITH PROPOSED IMPROVEMENTS

Date 08/15/18	Project No. 770626302	Figure 3

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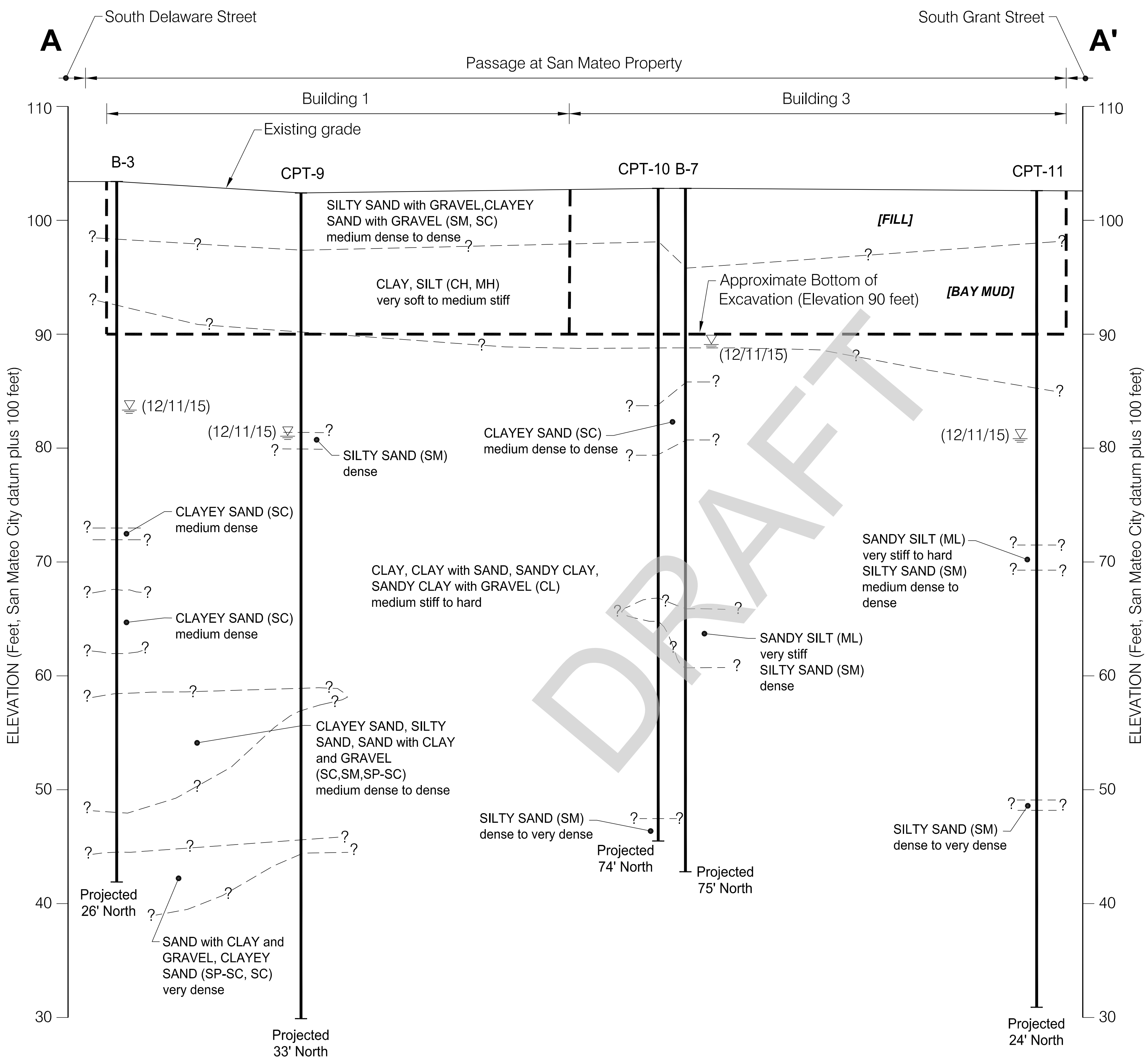

Notes:
1. Existing grade taken from an electronic file titled "CONCAR_ ALTA-CHAN USE ONLY.dwg," by BKF dated 12/29/15.
2. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
3. Bottom of Bay Mud Elevation shown is from contour plan on Figure 6.
4. Bottom of excavation assumes basement finished floor at Elevation 93 feet, two-foot thick mat slab and 12-inch thick crushed rock working pad.

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PASSAGE AT SAN MATEO
San Mateo, California

IDEALIZED SUBSURFACE PROFILE A-A'

Date 01/29/18	Project No. 770626302	Figure 4

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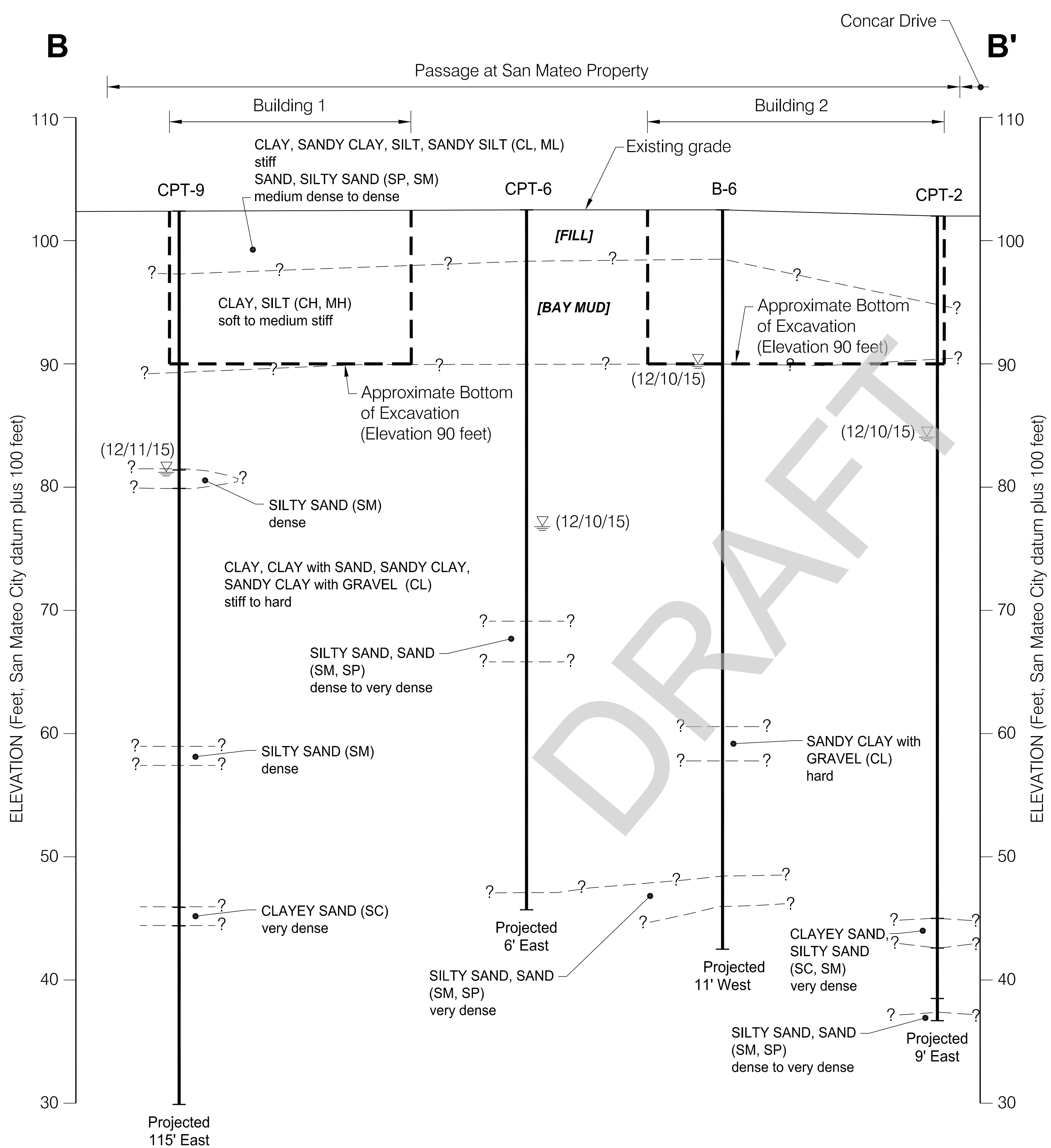
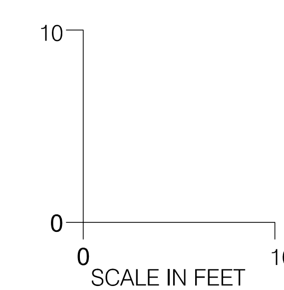


Notes:
1. Existing grade taken from an electronic file titled "CONCAR_ ALTA-CHAN USE ONLY.dwg," by BKF dated 12/29/15.
2. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
3. Bottom of Bay Mud Elevation shown is from contour plan on Figure 6.
4. Bottom of excavation assumes basement finished floor at Elevation 93 feet, two-foot thick mat slab and 12-inch thick crushed rock working pad.



PASSAGE AT SAN MATEO
San Mateo, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 01/29/18	Project No. 770626302	Figure 5

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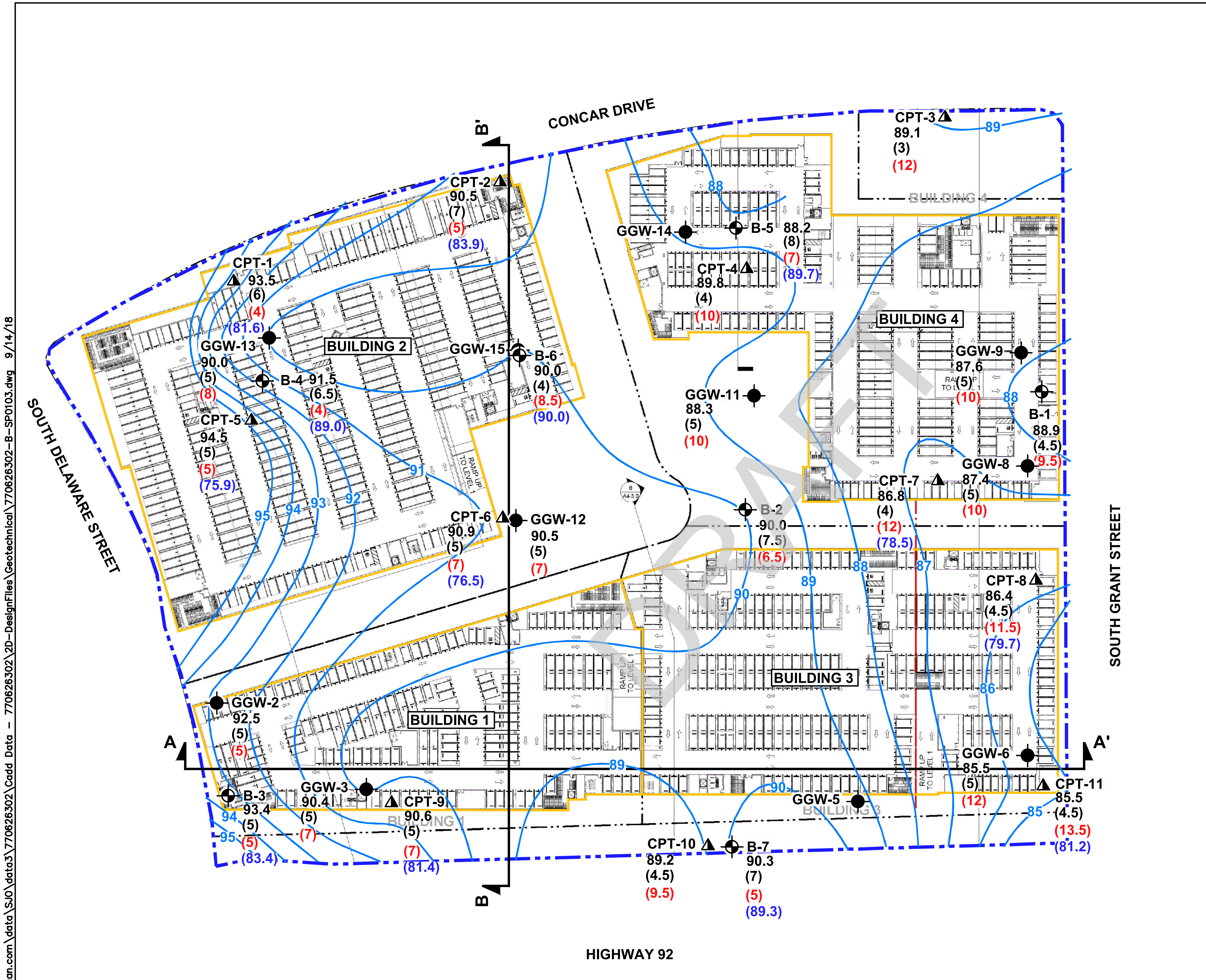

EXPLANATION

B-1	Approximate location of boring by Langan, December 2015
CPT-1	Approximate location of cone penetration test by Langan, December 2015
GGW-2	Approximate location of environmental boring by Erler & Kalinowski, Inc., January 2016
	Site boundary
88	Bottom of Bay Mud elevation contour (San Mateo City Datum plus 100 feet)
94.5	Bottom of Bay Mud elevation (San Mateo City Datum plus 100 feet)
(8)	Fill thickness (feet)
(4)	Bay Mud thickness (feet)
(83.4)	Groundwater elevation (San Mateo City Datum plus 100 feet)
A A'	Idealized subsurface profile location
	Basement outline

0 100 Feet

Approximate scale

PASSAGE AT SAN MATEO
San Mateo, California

BOTTOM OF BAY MUD ELEVATION CONTOURS OVERLAID ON BASEMENT PLAN

Date 08/15/18	Project No. 770626302	Figure 6

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Reference: Base map from a drawing titled "Site Plan_Basement," Sheet A0-1.1, by MVE Partners, dated 08/31/18.

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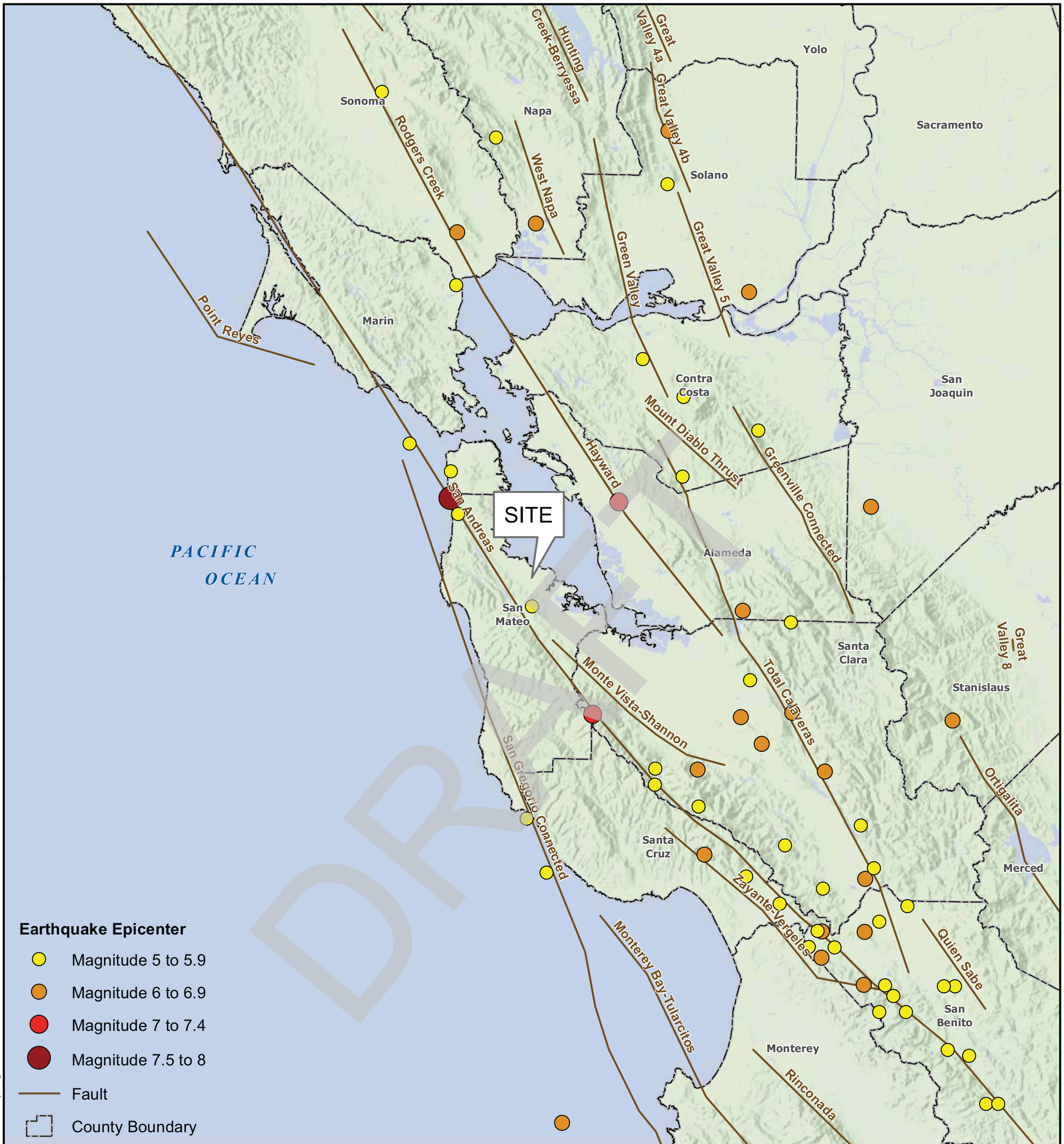

Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (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.

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PASSAGE AT SAN MATEO San Mateo, California	MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA		
LANGAN	Date 01/29/18	Project No. 770626302	Figure 7

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

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PASSAGE AT SAN MATEO
San Mateo, California

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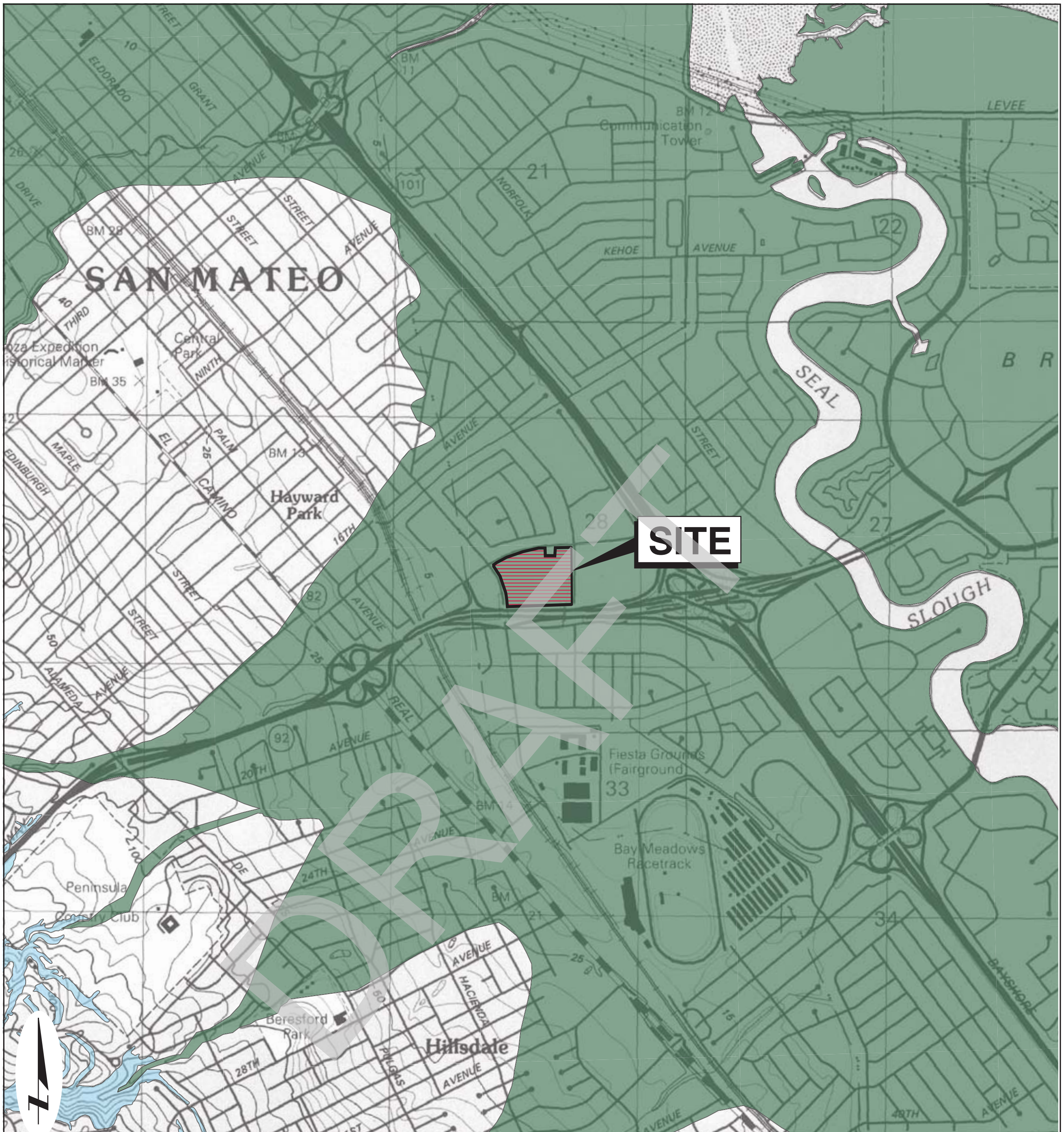
MODIFIED MERCALLI INTENSITY SCALE

Date 01/29/18 | Project No. 770626302 | Figure 8



EXPLANATION

Liquefaction; Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

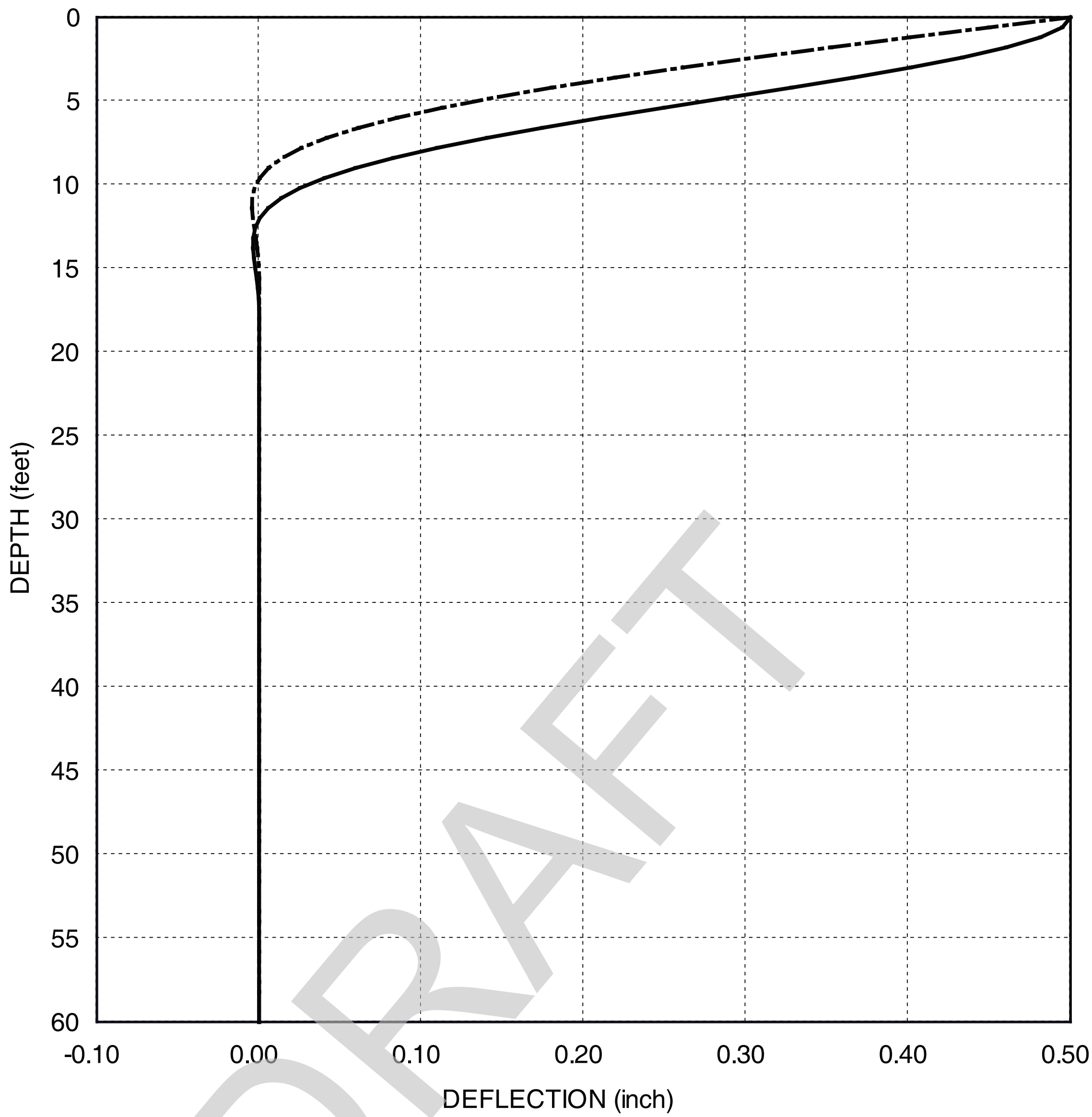
Earthquake-Induced Landslides; Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 2,000 4,000 Feet

Approximate scale

Reference:
State of California Preliminary "Seismic Hazard Zones," San Mateo Quadrangle, released on August 17, 2017.

PASSAGE AT SAN MATEO San Mateo, California	**REGIONAL SEISMIC HAZARD ZONES MAP**
LANGAN	Date 01/30/18 | Project No. 770626302 | Figure 9



Curve	Restraint	Lateral Load (kips)
——— (solid)	Fixed	14
—·—·— (dash-dot)	Free	5

Notes for Figure:

1. The profiles shown are for a single 16-inch Auger-Cast Displacement (ACDP) pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 200 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D .
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Table 7, however, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no additionally applied moment at the pile head.
4. Passive resistance of pile caps has not been included.
5. Top of pile assumed at Elevation 90 feet.

PASSAGE AT SAN MATEO
San Mateo, California

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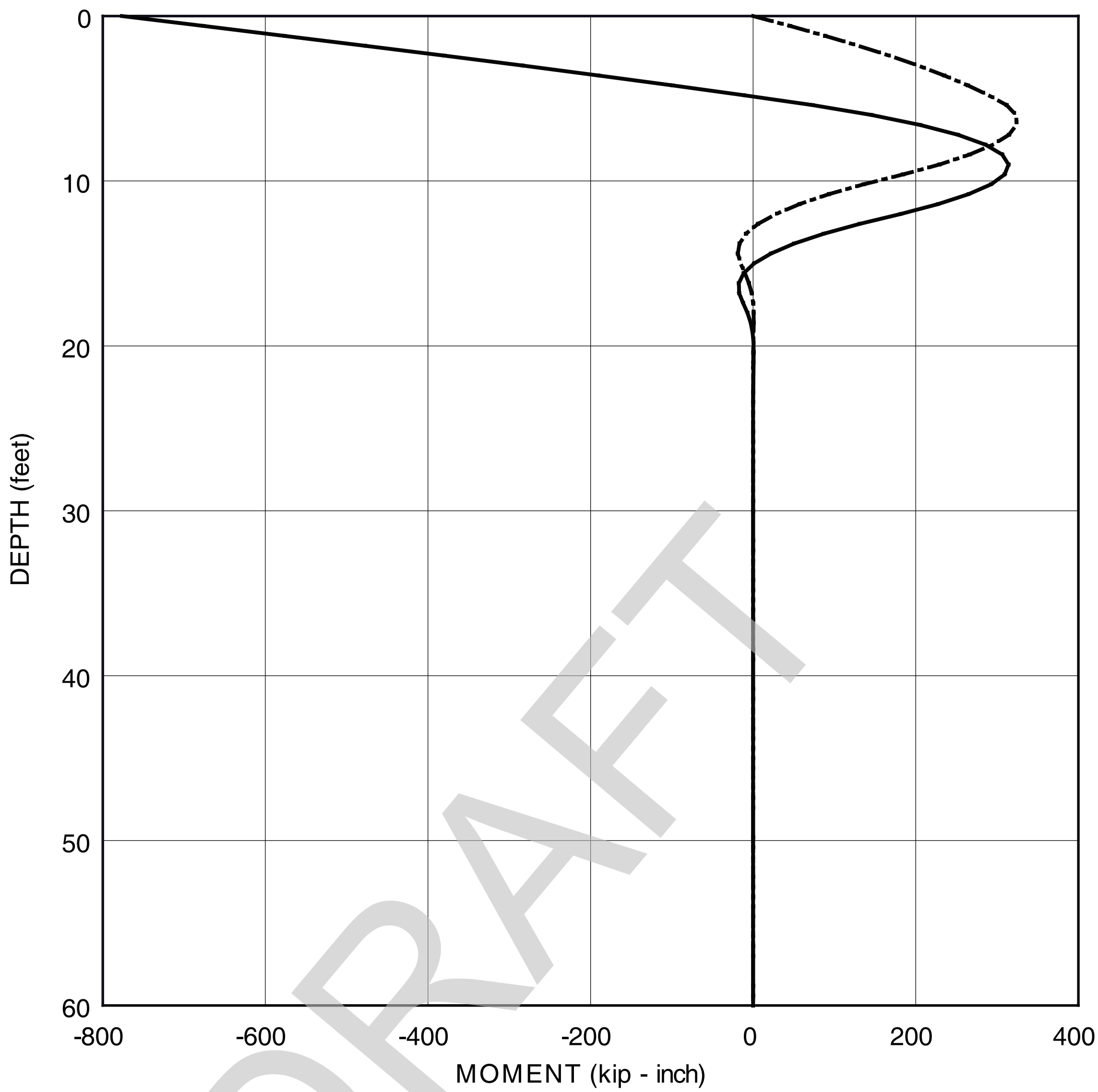
PRELIMINARY DEFLECTION PROFILE
16-INCH-DIAMETER ACDP PILE
WITH ONE BASEMENT

Date 09/05/18 | Project No. 770626302 | Figure 10



Curve	Restraint	Lateral Load (kips)
———	Fixed	14
—·—·—·—	Free	5

Notes for Figure:

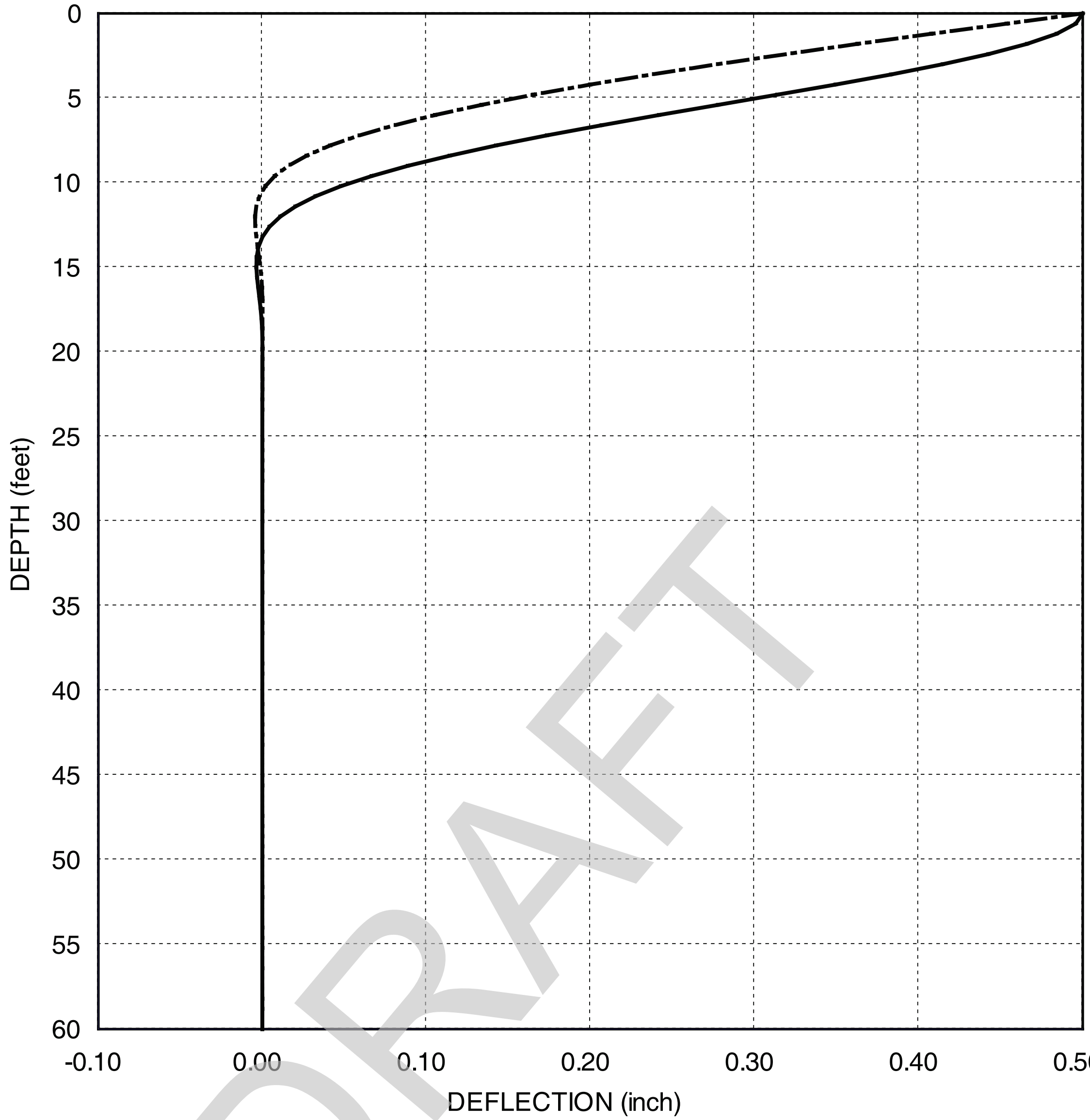
1. The profiles shown are for a single 16-inch Auger-Cast Displacement (ACDP) pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 200 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Table 7, however, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no additionally applied moment at the pile head.
4. Passive resistance of pile caps has not been included.
5. Top of pile assumed at Elevation 90 feet.

PASSAGE AT SAN MATEO
San Mateo, California

LANGAN

PRELIMNIARY MOMENT PROFILE
16-INCH-DIAMETER ACDP PILE
WITH ONE BASEMENT

Date 09/05/18 | Project No. 770626302 | Figure 11

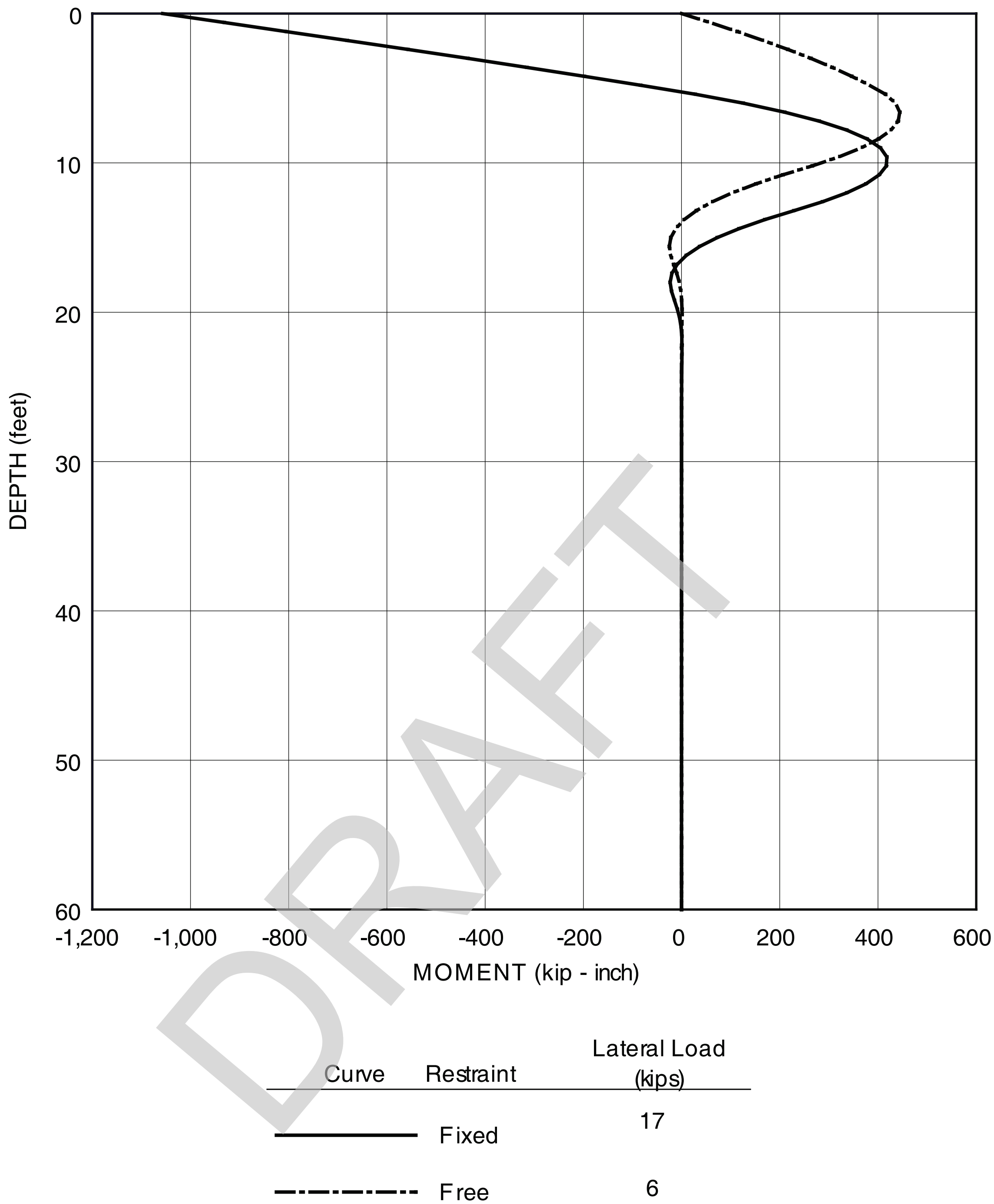


Curve	Restraint	Lateral Load (kips)
————	Fixed	17
—·—·—·—	Free	6

Notes for Figure:

1. The profiles shown are for a single 18-inch Auger-Cast Displacement (ACDP) pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 300 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Table 7, however, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no additionally applied moment at the pile head.
4. Passive resistance of pile caps has not been included.
5. Top of pile assumed at Elevation 90 feet.

PASSAGE AT SAN MATEO San Mateo, California	**PRELIMINARY DEFLECTION PROFILE** **18-INCH-DIAMETER ACDP PILE** **WITH ONE BASEMENT**
LANGAN	Date 09/05/18 | Project No. 770626302 | Figure 12



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Notes for Figure:

1. The profiles shown are for a single 18-inch Auger-Cast Displacement (ACDP) pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 300 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Table 7, however, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no additionally applied moment at the pile head.
4. Passive resistance of pile caps has not been included.
5. Top of pile assumed at Elevation 90 feet.

PASSAGE AT SAN MATEO
San Mateo, California

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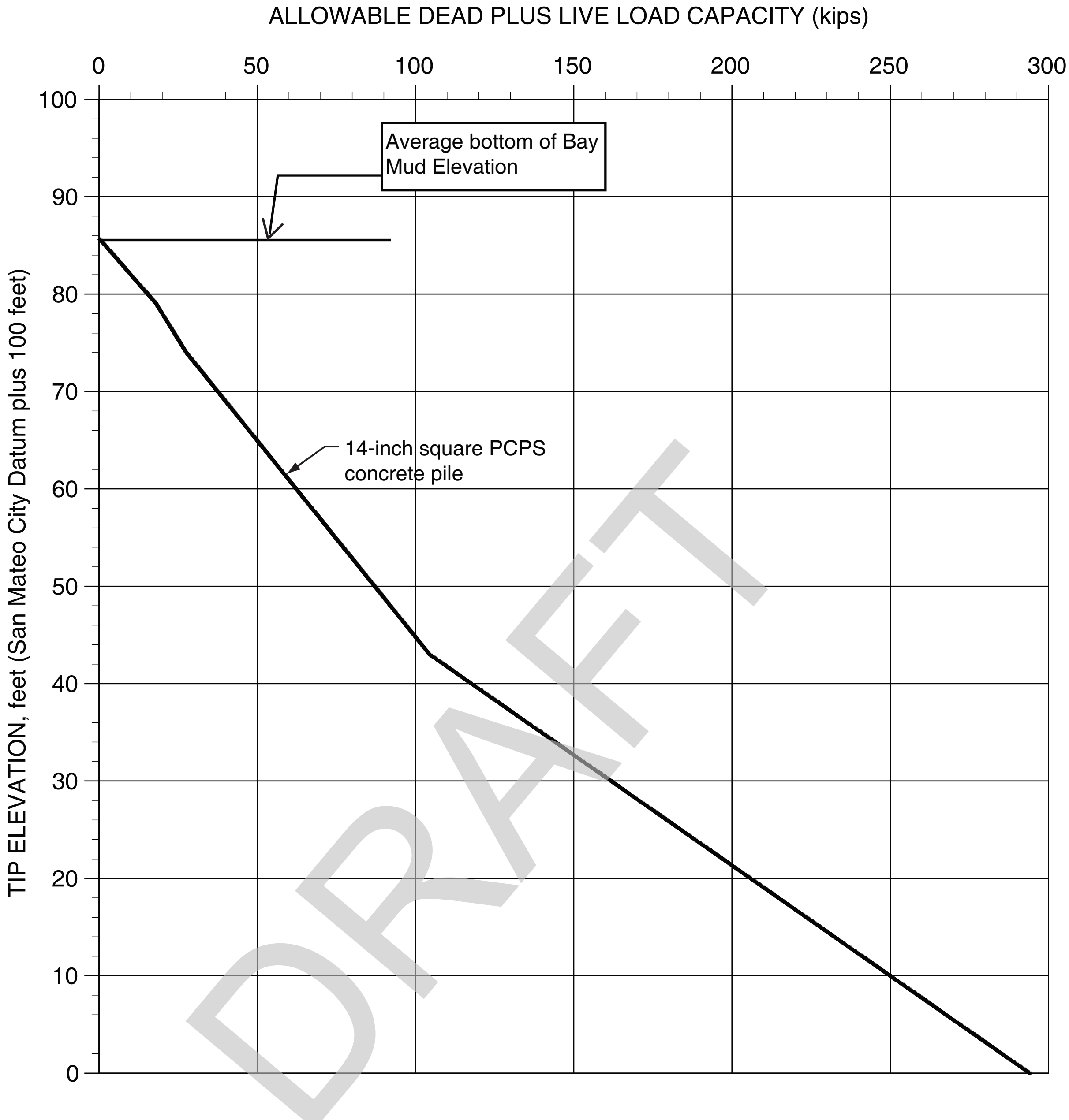
PRELIMINARY MOMENT PROFILE
18-INCH-DIAMETER ACDP PILE
WITH ONE BASEMENT

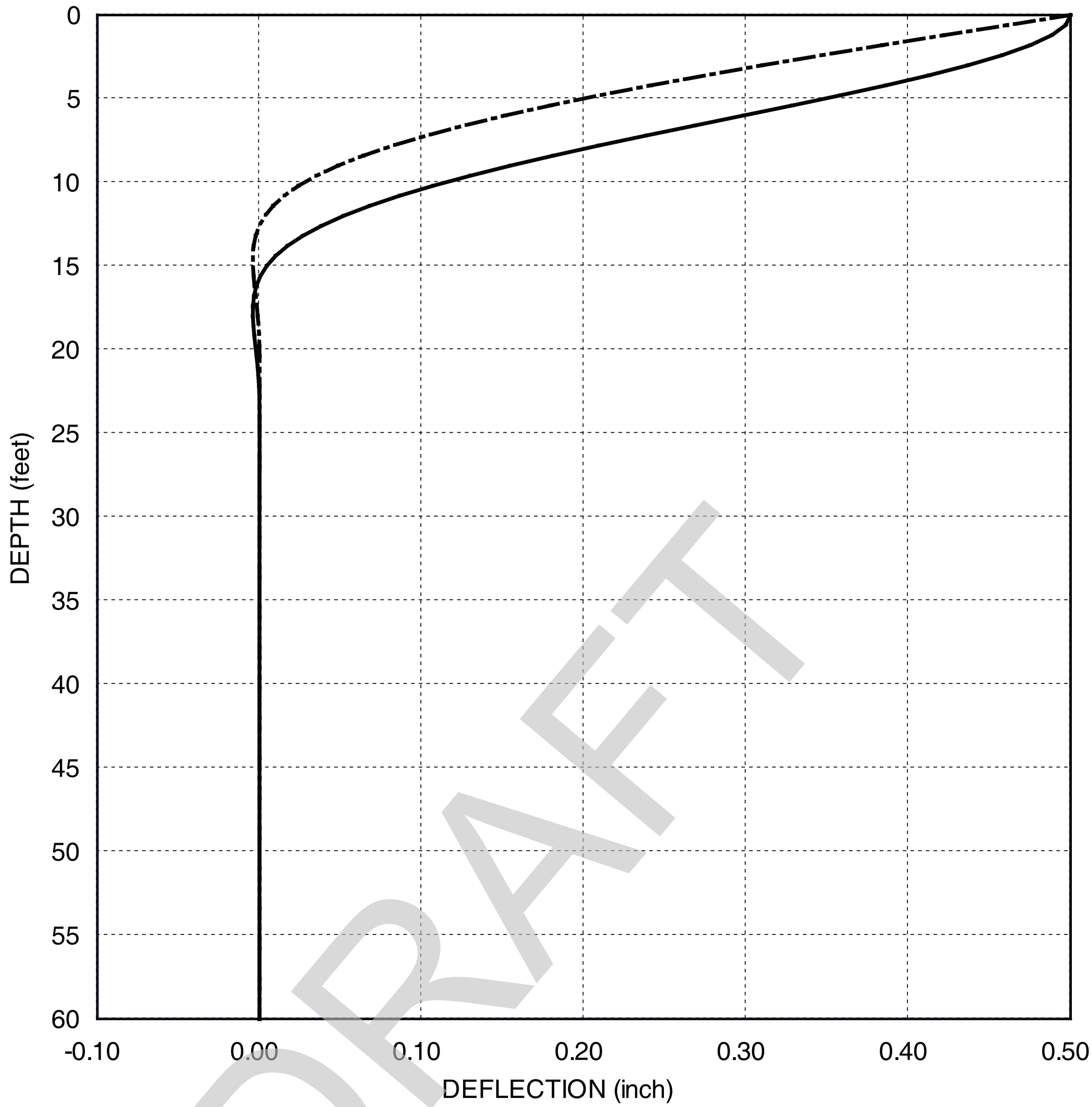
Date 09/05/18	Project No. 770626302	Figure 13



Notes:

1. The indicated capacities are for a single 14-inch square precast, prestressed (PCPS) concrete pile and are for dead plus live loads (FS= 2) and may be increased by one-third for total loads. For uplift, use indicated capacity for temporary load. For permanent uplift loads, use 80 percent of the indicated capacities.
2. Capacities are based on the allowable strength of the supporting soil; the structural capacity of the pile may govern.
3. Piles should be spaced no closer than three diameters center to center.

PASSAGE AT SAN MATEO San Mateo, California	**AXIAL PILE CAPACITY** **14-INCH SQUARE PCPS** **CONCRETE PILE**
LANGAN	Date 09/06/18 | Project No. 770626302 | Figure 14



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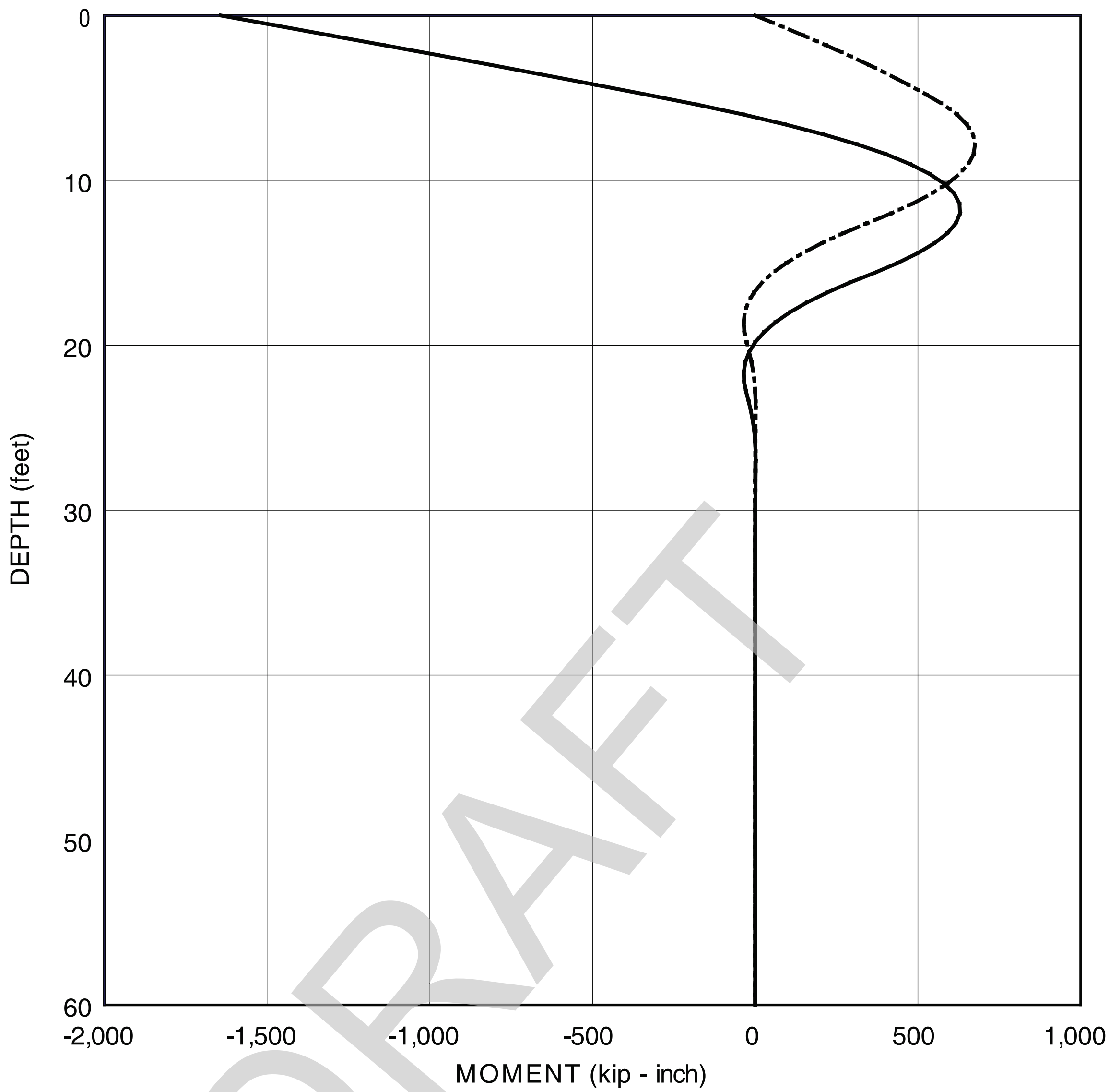
Notes for Figure:

1. The profiles shown are for a single 14-inch square precast, prestressed, (PCPS) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 300 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Table 7, however, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no additionally applied moment at the pile head.
4. Passive resistance of pile caps has not been included.
5. Top of pile assumed at Elevation 90 feet.

PASSAGE AT SAN MATEO San Mateo, California	**DEFLECTION PROFILE** **14-INCH-SQUARE PCPS CONCRETE PILE** **WITH ONE BASEMENT**
LANGAN	Date 09/05/18 | Project No. 770626302 | Figure 15



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Curve	Restraint	Lateral Load (kips)
——————	Fixed	23
—·—·—·—	Free	9

Notes for Figure:

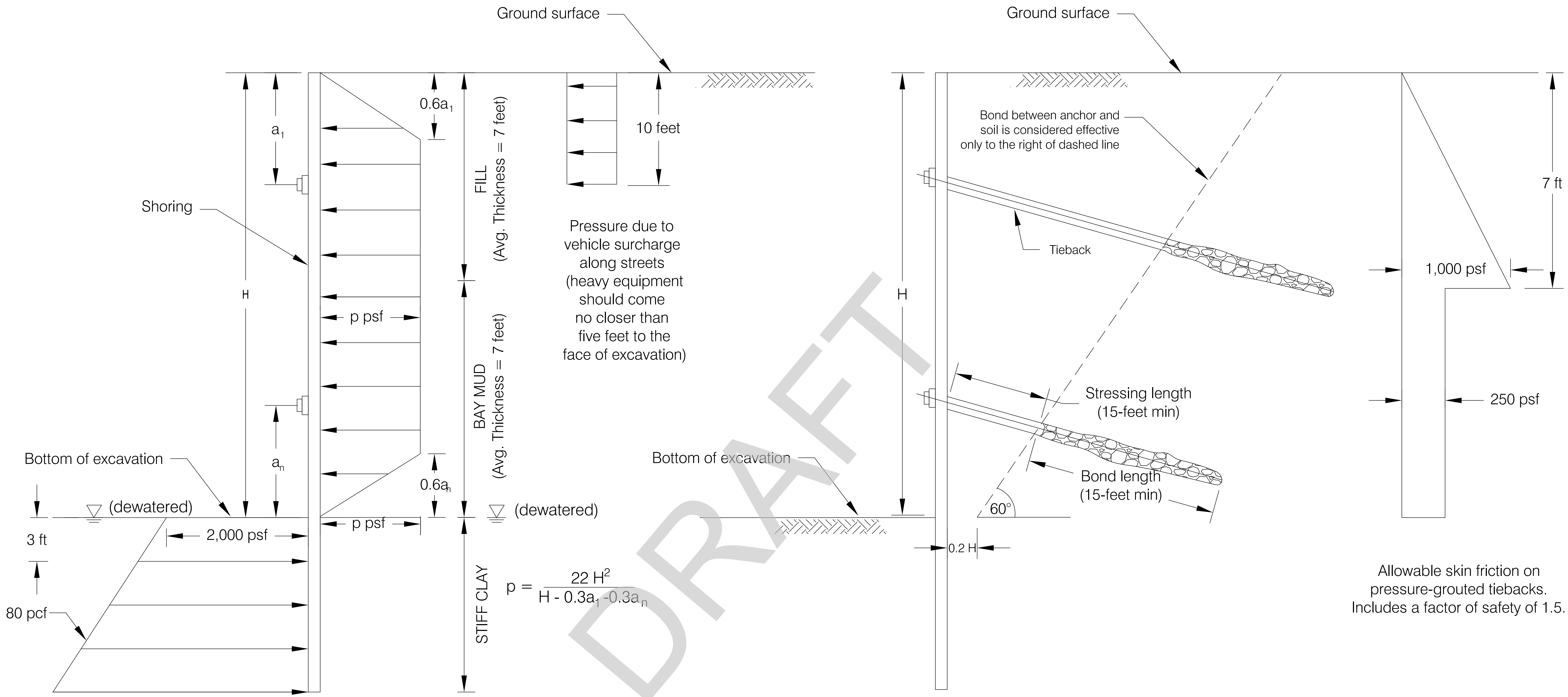
1. The profiles shown are for a single 14-inch square precast, prestressed, (PCPS) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 300 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Table 7, however, moment profile used to check individual piles in a group should be for the unfactored load.
3. Assumes there is no additionally applied moment at the pile head.
4. Passive resistance of pile caps has not been included.
5. Top of pile assumed at Elevation 90 feet.

PASSAGE AT SAN MATEO
San Mateo, California

LANGAN

MOMENT PROFILE
14-INCH-SQUARE PCPS CONCRETE PILE
WITH ONE BASEMENT

Date 09/05/18	Project No. 770626302	Figure 16



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Notes:
1. Passive pressures includes a factor of safety of approximately 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Surcharge pressure due to construction, if any, should be added to the above shoring pressures
4. Assumes groundwater will be lowered to 3 feet below bottom of excavation.
5. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
6. Thickness of fill is assumed to be approximately 7 feet; thickness of Bay Mud is assumed to be approximately 7 feet.
7. All elevations reference San Mateo City Datum plus 100 feet.

NOT TO SCALE

PASSAGE AT SAN MATEO
San Mateo, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING WITH TIEBACKS TEMPORARY SHORING SYSTEM (BUILDING 1 AND 2)

Date 09/10/18	Project No. 770626302	Figure 17

LANGAN

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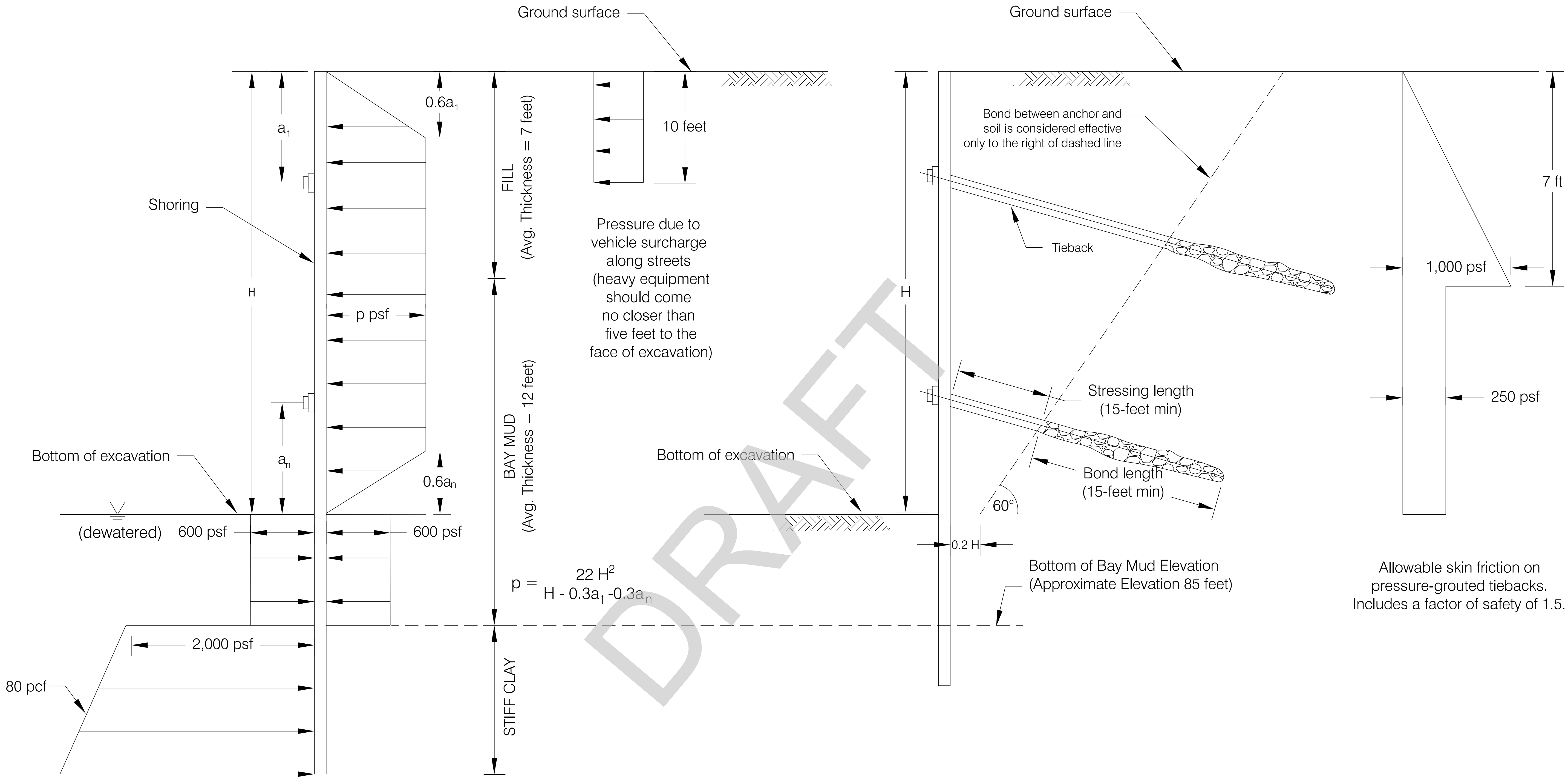

Notes:
1. Passive pressures includes a factor of safety of approximately 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Surcharge pressure due to construction, if any, should be added to the above shoring pressures
4. Assumes groundwater will be lowered to 3 feet below bottom of excavation.
5. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
6. Thickness of fill is assumed to be approximately 7 feet; thickness of Bay Mud is assumed to be approximately 12 feet.
7. All elevations reference San Mateo City Datum plus 100 feet.

NOT TO SCALE

PASSAGE AT SAN MATEO
San Mateo, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING WITH TIEBACKS TEMPORARY SHORING SYSTEM (BUILDING 3 AND 4)

Date 09/10/18	Project No. 770626302	Figure 18

LANGAN

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APPENDIX A

LOG OF TEST BORINGS

DRAFT

PROJECT:	**CONCAR PROPERTY** San Mateo, California	**Log of Boring B-1**

Boring location:	See Site Plan, Figure 2	Logged by:	K. Watkins
Date started:	12/10/15	Date finished:	12/10/15
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Automatic Safety
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shebly Tube (ST), Dames & Moore (D&M)		

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 102.4 feet[2])		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches asphalt concrete (AC)	FILL						
1–2	GRAB			CL	SANDY CLAY with GRAVEL (CL) dark brown to black, stiff, moist, fine sand, fine gravel	FILL						
3	S&H	2 5 12	12	CL		FILL						
4	GRAB											
5–6	S&H	0 0 0	0	MH	SILT (MH) olive-gray, very soft, moist, with clay, with fibrous organics LL = 74, PI = 37, see Figure C-10	BAY MUD						
8–9	ST		50 psi	MH	soft, less fibrous organics Consolidation Test, see Figure C-1	BAY MUD					95.1	48
9					TxUU Test, see Figure C-5	BAY MUD	TxUU	900	340		84.3	51
10–11	S&H	0 0 2	1	MH	very soft	BAY MUD						
14–16	S&H	3 4 5	6	CL	SANDY CLAY (CL) gray, medium stiff, wet, fine sand, trace fine gravel		PP		800			
19–21	S&H	16 15 12	19	SC	CLAYEY SAND with GRAVEL (SC) red-yellow, medium dense, wet, fine-grained, fine angular gravel					28.6	12.8	
21–24				SC	SANDY CLAY (CL) yellow-brown with gray-brown mottling, very stiff, wet, fine sand							
24–26	S&H	4 7 9	11	CL	CLAY with SAND (CL) light brown, stiff, wet, fine sand		PP		2,300			
29–31	D&M		200 psi	SC	CLAYEY SAND (SC) yellow-brown, dense, fine- to coarse-grained, trace fine gravel							
32												

DRAFT

LANGAN TREADWELL ROLLO

Project No.:	Figure:
770626301	A-1a

Log of Boring B-1

PROJECT: **CONCAR PROPERTY**
San Mateo, California

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33					SC	CLAYEY SAND (SC) (continued)						
34						CLAY with SAND (CL) gray, very stiff, wet, fine sand, trace organic inclusions						
36	S&H		10 17 25	29	CL		PP		3,000			
39						SANDY CLAY (CL) gray, stiff, wet, fine sand						
41	S&H		4 6 13	13	CL							
43–44						grades with increase in sand content, with fine angular gravel						
45–46	SPT		12 4 4	10		CLAY (CL) light brown, stiff, wet, trace organic inclusions						
47	SPT		2 4 6	12								
51	S&H		5 6 9	11	CL	olive-gray with yellow-brown mottling	PP		1,700			
56	S&H		3 4 6	7			PP		1,600			
59						SANDY CLAY (CL) gray-brown, stiff, wet, fine sand, trace fine gravel						
60–61	S&H		7 6 12	13	CL							
62–64												

Boring terminated at a depth of 61.5 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 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-1b

DRAFT

PROJECT:	CONCAR PROPERTY San Mateo, California	Log of Boring B-2

Boring location: See Site Plan, Figure 2

Logged by: K. Watkins

Date started: 12/11/15 | Date finished: 12/11/15

Drilling method: Rotary Wash

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

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)

MATERIAL DESCRIPTION

Ground Surface Elevation: 103.5 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches asphalt concrete (AC)	FILL						
1–2	GRAB			SP-SM	SAND with SILT and GRAVEL (SP-SM) brown, loose, moist, fine- to coarse-grained, fine-to coarse angular gravel up to 1-1/2 inches in diameter Particle Analysis, see Figure C-11	FILL				7.1	6.1	
3–4	S&H	7 8 16	17	SC	CLAYEY SAND with GRAVEL (SC) brown, medium dense, moist, fine- to coarse-grained angular gravel	FILL						
5–7	SPT	9 8 8	19	SP	SAND with SILT (SP) red-brown, medium dense, moist, fine- to coarse-grained	FILL						
8	S&H	1 1 1	1	CH	CLAY (CH) gray, very soft, wet, with silt	BAY MUD						
10–11	S&H	1 0 0	0	CH		BAY MUD						
14–16	S&H	3 4 5	6	CL	CLAY (CL) olive-gray, stiff, wet, trace fine sand		PP		1,500			
19–21	SPT	8 7 14	25	SC	CLAYEY SAND with GRAVEL (SC) red-brown, medium dense, wet, fine- to coarse-grained, trace fine angular gravel					18.8	14.6	
23–26	S&H	5 6 7	9	CL	SANDY CLAY (CL) gray-brown and yellow-brown, stiff, wet, fine to medium sand							
30–31	S&H	5 7 9	11	SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet, fine-grained							
31–32				CL	CLAY with SAND (CL) yellow-brown and gray-brown, stiff, wet, with silt, fine sand		PP		2,000			

DRAFT

LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-2a

PROJECT:	**CONCAR PROPERTY** San Mateo, California	**Log of Boring B-2** PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33–35				CL	CLAY with SAND (CL) (continued)						
36	D&M		175 psi								
36–39	SPT	8 10 8	22	SM	SILTY SAND with GRAVEL (SM) yellow-brown, medium dense, wet, fine-grained, with coarse gravel up to 1-inch in diameter, trace silt				13.8	20.5	
39.5–41	SPT	6 5 6	13	SM	SILTY SAND (SM) yellow-brown, medium dense, wet, fine- to medium-grained, trace fine gravel						
41–44				CL	SANDY CLAY (CL) gray-brown, stiff, wet, fine sand						
44–49.5	S&H	10 17 23	28	SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet, fine-grained, with silt						
47	SPT	14 10 12	26								
49.5–54	D&M		200 psi	CL	CLAY (CL) blue-gray, stiff, wet, trace organic inclusions	PP		2,300			
51					TxUU Test, see Figure C-6	TxUU	3,500	1,410		33.8	86
54–59	S&H	6 7 12	13	CL	CLAY with SAND (CL) gray, stiff, wet, fine sand	PP		1,600			
59–61.5	S&H	7 8 22	21	CL	SANDY CLAY (CL) olive-gray, very stiff, wet, fine sand				52.9	20.9	
62–64											

DRAFT

Boring terminated at a depth of 61.5 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 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-2b

TEST GEOTECH LOG 770626301.GPJ TR.GDT 2/2/16

PROJECT:	CONCAR PROPERTY San Mateo, California	Log of Boring B-3 PAGE 1 OF 2

Boring location:	See Site Plan, Figure 2
Date started:	12/10/15
Date finished:	12/11/15
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic Safety
Samplers:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by:	K. Watkins

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 103.4 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches asphalt concrete (AC) (FILL)						
1–2	GRAB			SM	SILTY SAND with GRAVEL (SM) gray-brown, moist, fine- to coarse-grained, with fine- to coarse angular gravel up to 2 inches in diameter (FILL)						
3	S&H	11 13 17	21	SC	CLAYEY SAND with GRAVEL (SC) dark brown, medium dense, moist, fine sand, fine angular gravel, wood debris (FILL)						
4	GRAB			SC					46.6	11.5	
5–6	S&H	1 0 1	1	MH	SILT (MH) gray-brown, very soft to soft, wet (BAY MUD)						
8	S&H	1 1 1	1	MH	LL = 54, PI = 8, see Figure C-10 olive-gray, with fibrous organics (BAY MUD)	PP		500			
10–11	S&H	3 5 9	10	CL	CLAY (CL) dark gray to gray-brown, stiff, wet, trace fine sand	PP		1,200			
14				CL	SANDY CLAY (CL) yellow-brown, very stiff, wet, fine sand, trace fine angular gravel						
15–16	SPT	4 7 10	20	CL							
19–20				CL	CLAY (CL) yellow-brown with light brown and gray mottling, stiff, wet ▽ (12/11/15, 8:00 a.m.)						
20–21	S&H	5 8 12	14	CL		PP		3,200			
25–26	S&H	5 8 11	13	CL	yellow-brown, trace fine sand	PP		1,600			
30–31	S&H	8 11 12	16	SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet, fine- to coarse-grained						
32	SPT		17	CL							

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LANGAN TREADWELL ROLLO

Project No.: 770626301

Figure: A-3a

PROJECT: **CONCAR PROPERTY**
San Mateo, California

Log of Boring B-3

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33	SPT	4 6 8	17	CL	SANDY CLAY (CL) yellow-brown, very stiff, wet, fine sand						
35–36	D&M		200 psi	CL	CLAY with SAND (CL) olive-gray, stiff, wet, fine sand; Consolidation Test, see Figure C-2; TxUU Test, see Figure C-7	TxUU	2,600	1,670		25.6 24.1	96 100
37–40				SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet, fine-grained						
41	S&H	15 16 22	27	SC	gray-brown to yellow-brown, trace coarse gravel				33.4	17.9	
42	SPT	4 5 6	13	CL	SANDY CLAY with GRAVEL (CL) gray-brown, stiff, wet, fine sand, fine gravel						
43–44				CL	CLAY (CL) gray-brown, stiff, wet, trace organic inclusions						
45–48	S&H	5 8 15	16	SC	CLAYEY SAND (SC) blue-gray, medium dense, wet, fine-grained				22.7	23.3	
49–50				SM	SILTY SAND (SM) gray, dense, wet, fine-grained						
51	S&H	15 23 32	39	SM	with trace coarse-grained sand, trace fine gravel				20.6	20.1	
52	SPT	8 23 19	50	SM	very dense						
53–55				SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray, medium dense to dense, wet, fine- to coarse-grained, fine- to, coarse gravel up to 1 inch in diameter						
56–59	SPT	12 11 13	29	CL	SANDY CLAY (CL) light blue-gray, very stiff, wet, fine sand, trace fine gravel						
59–61.5	SPT	15 19 18	44	SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, very dense, wet, fine- to coarse-grained, fine- to coarse angular to subangular gravel up to 1-inch in diameter				11.6	15.1	
62–64											

Boring terminated at a depth of 61.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater measured at 20 feet below ground surface on 12/11/15 at 8:00 a.m.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-3b

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PROJECT: **CONCAR PROPERTY**
San Mateo, California

Log of Boring B-4

Boring location: See Site Plan, Figure 2

Date started: 12/10/15 | Date finished: 12/10/15

Drilling method: Hollow Stem Auger (Mobile B-61 rig)

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Safety Downhole Wireline

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

Logged by: A. Nabulsi

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 102.5 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						5 inches asphalt concrete (AC)						
						7 inches aggregate base (AB)						
1–2	GRAB				CL	SANDY CLAY with GRAVEL (CL) red-brown, stiff, moist, fine sand, fine subangular to angular gravel; R-Value Test, see Figure C-12 (FILL)						
3	S&H		4 7 8	9	CL							
5	S&H		4 8 4	7	CL	medium stiff (FILL)						
6					CH	CLAY (CH) black, medium stiff, moist, with silt, organics, trace glass, strong odor (FILL)						
8	S&H		2 2 4	4	CH	CLAY (CH) gray, soft, moist, with silt; grades to black (BAY MUD)						
11	S&H		5 9 16	15	CH	(BAY MUD)						
12					CL	SANDY CLAY (CL) olive-brown, stiff, moist, fine sand						
13						▽ (12/10/15, 2:25 p.m.)						
14	S&H		14 30 38	41	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, dense, wet, fine- to coarse-grained, fine subangular to angular gravel						
15					SC					38.5	12.5	
19	S&H		8 12 16	17	CL	SANDY CLAY with GRAVEL (CL) yellow-brown, very stiff, wet, fine sand, fine subangular gravel						
24	S&H		9 9 14	14	CL	SANDY CLAY (CL) red-brown, stiff, wet, fine sand, with silt						
29	S&H		9 11 17	17	CL	CLAY (CL) olive, very stiff, wet						
32												

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LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-4a

PROJECT: **CONCAR PROPERTY**
San Mateo, California

Log of Boring B-4

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33						SANDY CLAY (CL) olive, very stiff, wet, fine sand						
34	S&H		8 11 7	17								
35–38												
39	SPT		9 14 15	29								
40–43												
44	S&H		12 24 34	35	CL	hard						
45–48												
49	S&H		13 18 23	25		gray, very stiff						
50–53												
54	S&H		13 50/ 5"	30/ 5"		hard						
55–57												
58					SC	CLAYEY SAND with GRAVEL (SC) gray-brown, dense, wet, fine sand, with fine angular gravel						
59	S&H		15 33 25	35								
60					CL	CLAY with SAND (CL) yellow-brown, hard, wet, fine sand, with silt						
61–64												

Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 13.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-4b

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PROJECT: **CONCAR PROPERTY**
San Mateo, California

Log of Boring B-5

Boring location:	See Site Plan, Figure 2
Date started:	12/11/15
Date finished:	12/11/15
Drilling method:	Hollow Stem Auger (Mobile B-61 rig)
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Safety Downhole Wireline
Samplers:	Sprague & Henwood (S&H), Shebly Tube (ST)
Logged by:	A. Nabulsi

Ground Surface Elevation: 103.2 feet[2]

SAMPLES

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]
3	S&H	4, 6, 7	8
6	S&H	13, 13, 13	16
8	S&H	2, 1, 2	2
11	S&H	2, 3, 3	4
15	ST		200 psi
19	S&H	11, 15, 23	23
24	S&H	7, 9, 13	13
26	ST		200 psi
29	S&H	7, 8, 10	11

MATERIAL DESCRIPTION

Depth (feet)	LITHOLOGY	MATERIAL DESCRIPTION	
0–0.5		6 inches asphalt concrete (AC)	
0.5–1.3		10 inches aggregate base (AB)	
1.3–4.5	CL	SANDY CLAY with GRAVEL (CL) red-brown and gray, medium stiff to stiff, moist, fine sand, fine subangular gravel, trace organics	FILL
4.5–7.5	SP	SAND with GRAVEL (SP) red-brown, medium dense, moist, fine-grained, fine to coarse subrounded to angular gravel	FILL
7.5–15	CH	CLAY (CH) gray, soft, moist, strong odor ▽ (12/11/15, 10:32 a.m.)	BAY MUD
15–21.5	SC	CLAYEY SAND (SC) gray-brown to red-brown, medium dense, wet, fine-grained	
21.5–32	CL	SANDY CLAY (CL) light brown, stiff, wet, fine sand, with silt TxUU Test, see Figure C-8 Consolidation Test, see Figure C-3	

LABORATORY TEST DATA

Depth (feet)	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
27	TxUU	2,100	1,100		23.0 26.9	102 98

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LANGAN TREADWELL ROLLO

Project No.: 770626301
Figure: A-5a

PROJECT:	**CONCAR PROPERTY** San Mateo, California	**Log of Boring B-5** PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33–35	S&H		12 15 22	22	CL	SANDY CLAY (CL) (continued) olive, very stiff						
35–41	S&H		7 14 28	25	CL	CLAY (CL) gray, very stiff, wet, trace fine sand						
41–52	S&H		14 18 21	23	CL	SANDY CLAY (CL) gray, very stiff, wet, fine sand						
	S&H		8 23 26	29		light brown						
52–59.5	S&H		13 27 28	33	CL	CLAY (CL) gray, hard, wet						
	S&H		7 14 25	23		very stiff						
59.5–60					CL	SANDY CLAY (CL) gray, very stiff, wet, fine sand						

Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 13.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

LANGAN TREADWELL ROLLO

Project No.: 770626301 | Figure: A-5b

TEST GEOTECH LOG 770626301.GPJ TR.GDT 2/2/16

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Log of Boring B-6

PROJECT: **CONCAR PROPERTY**
San Mateo, California

Boring location:	See Site Plan, Figure 2
Date started:	12/10/15
Date finished:	12/10/15
Drilling method:	Hollow Stem Auger (Mobile B-61 rig)
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Safety Downhole Wireline
Samplers:	Sprague & Henwood (S&H), Shebly Tube (ST)
Logged by:	A. Nabulsi

Ground Surface Elevation: 102.5 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					5 inches asphalt concrete (AC)							
1					6 inches aggregate base (AB)	FILL						
1–2	BULK			CL	CLAY with SAND (CL) black, moist LL = 40, PI = 22, see Figure C-10	FILL						
3	S&H	11 10 8	11	CL	SANDY CLAY (CL) red-brown, stiff, moist, fine sand, trace organics and fine subangular gravel grades to black	FILL						
6	S&H	2 2 2	2	MH	SILT (MH) gray, soft, moist to wet	BAY MUD						
8	S&H	2 3 3	4	MH	medium stiff LL = 73 , PI = 36, see Figure C-10	BAY MUD						
11	S&H	2 2 3	3	MH	dark gray	BAY MUD						
12.5	ST		250 psi	MH	Consolidation Test (12.5'), see Figure C-4 ▽ (12/10/15, 10:06 a.m.)	BAY MUD					32.8	92
13				CL	SANDY CLAY (CL) olive, stiff, wet, fine sand TxUU Test, see Figure C-9		TxUU	1,300	1,720		12.7	120
16	S&H	10 22 23	27	CL								
18				CL	SANDY CLAY (CL) yellow-brown, very stiff, wet, fine sand							
19	S&H	8 16 16	19	CL								
24	S&H	6 9 12	13	CL	stiff							
29	S&H	9 13 21	20	CL								
30				CL	very stiff					56.0	20.4	
32												

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LANGAN TREADWELL ROLLO

Project No.: 770626301
Figure: A-6a

TEST GEOTECH LOG 770626301.GPJ TR.GDT 2/2/16

PROJECT:	CONCAR PROPERTY San Mateo, California	Log of Boring B-6 PAGE 2 OF 2

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
33					CL	CLAY with SAND (CL) gray-brown, stiff, wet, fine sand, trace silt						
34	S&H	■	7 8 10	11								
35												
36												
37					CL	SANDY CLAY (CL) gray-brown, very stiff, wet, fine sand						
38												
39	S&H	■	12 17 20	22								
40												
41												
42												
43												
44	S&H	■	8 44 50/ 4"	56/ 10"		grades with less sand						
45					CL	SANDY CLAY with GRAVEL (CL) yellow-brown, hard, wet, fine sand, fine subrounded to angular gravel						
46												
47												
48												
49	S&H	■	10 10 14	14								
50					CL	CLAY (CL) yellow-brown, stiff, wet, trace fine sand						
51												
52												
53												
54	S&H	■	7 50/ 5"	30/ 5"								
55					SP	SAND (SP) gray, very dense, wet, fine-grained, trace clay						
56												
57					CL	CLAY with SAND (CL) gray, stiff, wet, fine sand						
58												
59	S&H	■	7 9 14	14								
60												
61												
62												
63												
64												

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Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 12.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

LANGAN TREADWELL ROLLO

Project No.:	Figure:
770626301	A-6b

TEST GEOTECH LOG 770626301.GPJ TR.GDT 2/2/16

PROJECT:	CONCAR PROPERTY San Mateo, California	Log of Boring B-7

Boring location: See Site Plan, Figure 2

Logged by: A. Nabulsi

Date started: 12/11/15

Date finished: 12/11/15

Drilling method: Hollow Stem Auger (Mobile B-61 rig)

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Safety Downhole Wireline

Samplers: Sprague & Henwood (S&H), Shebly Tube (ST), Dames & Moore (D&M)

MATERIAL DESCRIPTION

Ground Surface Elevation: 102.8 feet[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5					6 inches asphalt concrete (AC)						
0.5–1.5					12 inches aggregate base (AB)						
1.5–6.5				SC	CLAYEY SAND with GRAVEL (SC) dark brown, dense, moist, fine- to coarse-grained, fine- to coarse subangular to angular gravel (FILL)						
3	S&H	33, 41, 29	42								
6	S&H	5, 7, 9	10								
6.5–7.7				CL	CLAY (CL) gray, medium stiff, moist (BAY MUD)						
7.7–9.5				CL	SANDY CLAY with GRAVEL (CL) black, medium stiff, moist, fine sand, fine gravel, trace organics (BAY MUD)						
8	S&H	5, 5, 6	7								
9.5–12.5				CH	CLAY (CH) dark gray to black, medium stiff, moist, strong odor (BAY MUD)						
11	S&H	3, 5, 6	7								
12.5–17				CL	SANDY CLAY (CL) gray, very stiff, wet, fine sand (12/11/15, 2:25 p.m.)						
14	S&H	5, 10, 16	16								
17–22				SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet, fine- to medium-grained, trace fine gravel						
19	S&H	6, 13, 19	19								
20									35.6	15.8	
22–32				CL	SANDY CLAY (CL) light brown, stiff, wet, fine sand, with silt						
24	S&H	6, 9, 9	11								
29	S&H	9, 10, 12	13								

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LANGAN TREADWELL ROLLO

Project No.: 770626301

Figure: A-7a

TEST GEOTECH LOG 770626301.GPJ TR.GDT 2/2/16

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SANDY CLAY (CL) (continued)						
33												
34	S&H		7 9 12	13	CL	brown						
35												
36												
37						SILTY SAND (SM) yellow-brown, dense, wet, fine- to medium-grained, trace fine subangular gravel						
38												
39	S&H		17 31 35	40	SM					18.5	14.4	
40												
41												
42						SANDY CLAY (CL) yellow-brown, hard, wet, fine sand						
43												
44	S&H		13 23 29	31	CL							
45												
46												
47						CLAY (CL) gray, stiff, wet, with silt						
48												
49	S&H		8 10 15	15								
50												
51	ST			300 psi								
52												
53					CL							
54	S&H		14 15 18	20		very stiff						
55												
56												
57												
58												
59	S&H		8 12 18	18								
60					CL	SANDY CLAY (CL) gray, very stiff, wet, fine sand						
61												
62												
63												
64												

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Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 13.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations base on San Mateo City datum plus 100 feet.

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

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

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APPENDIX B

CONE PENETRATION TESTS

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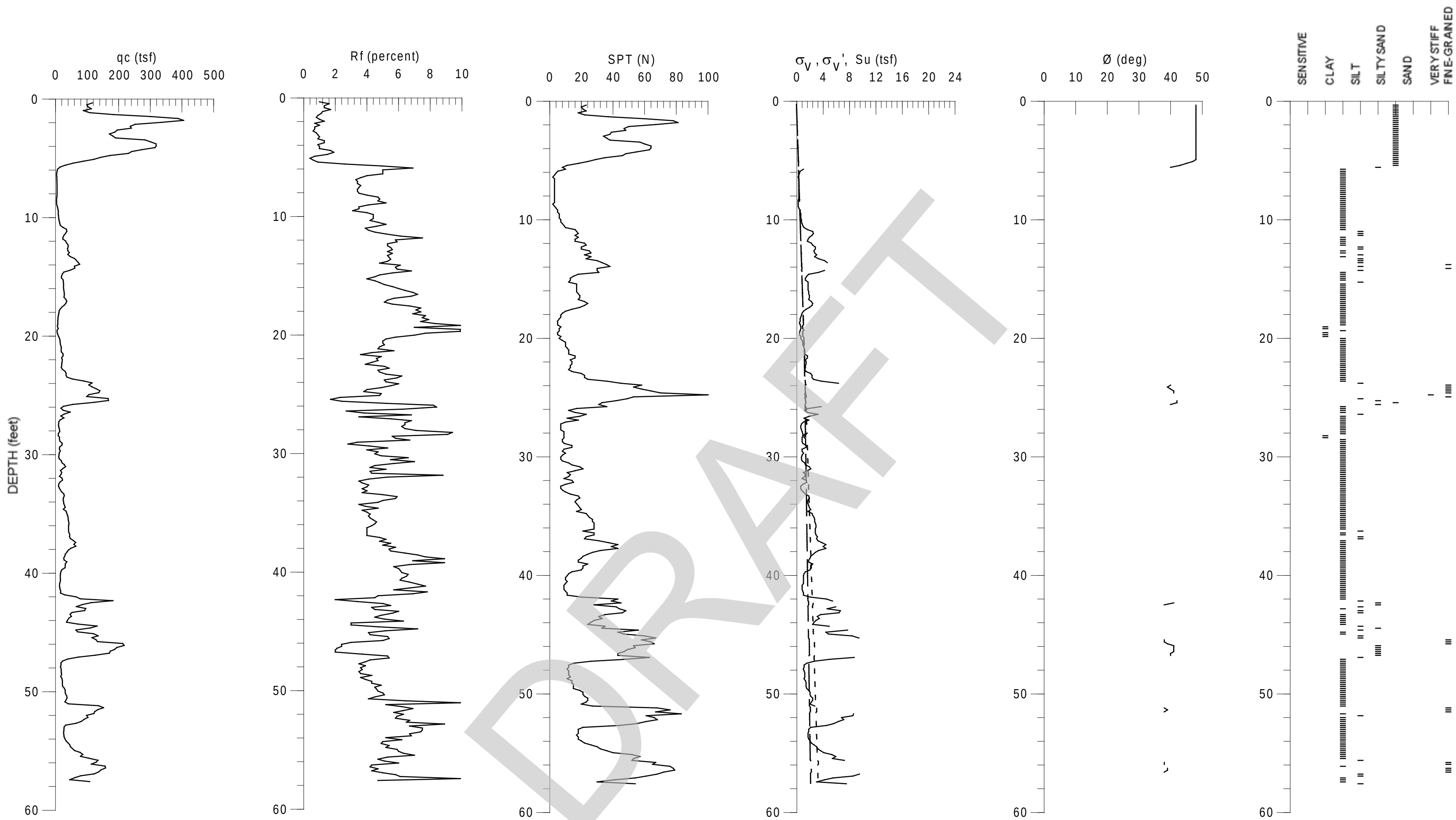


Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, s_u

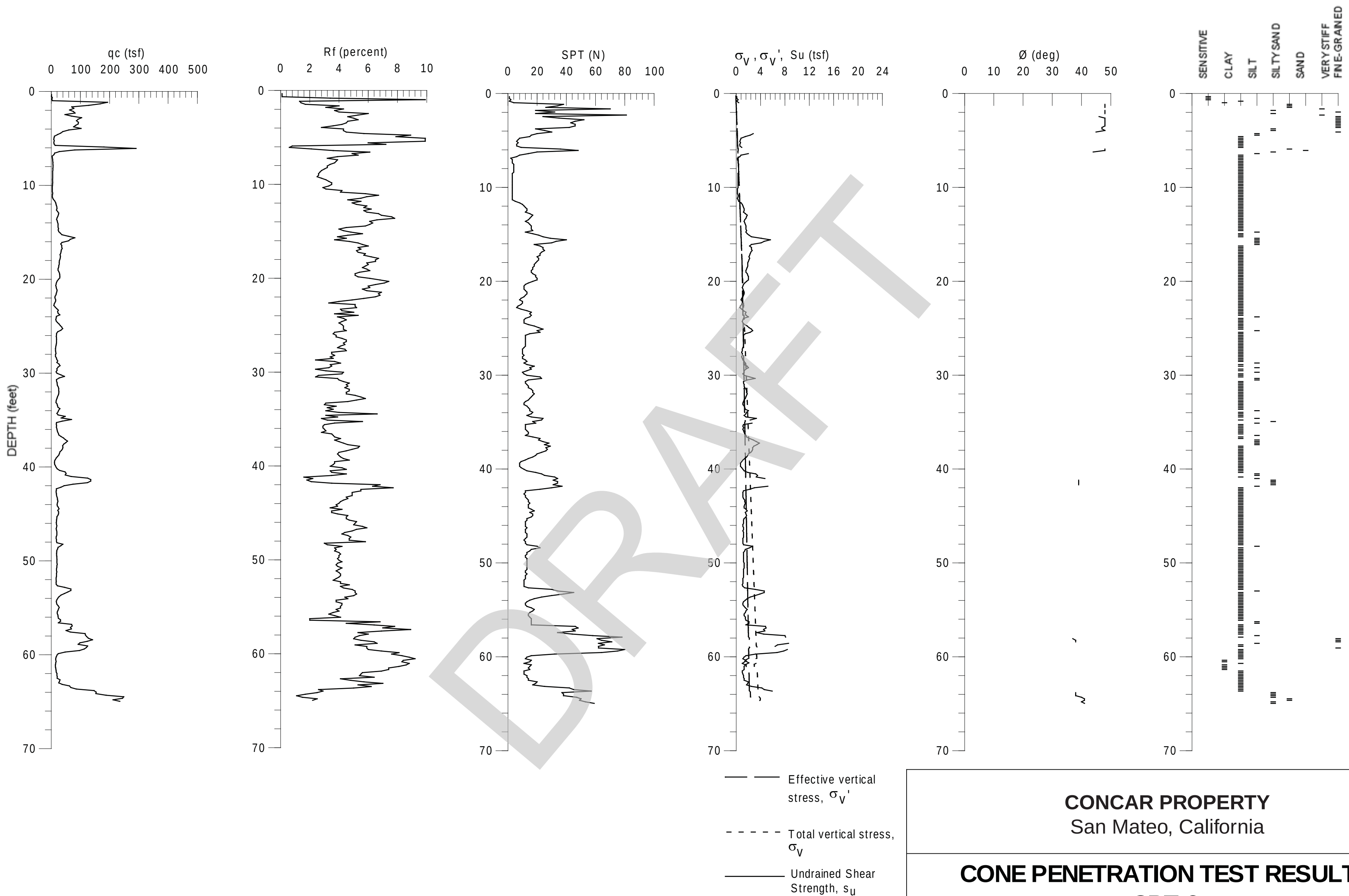
Terminated at a depth of 57.6 feet.
Groundwater calculated at a depth of 20.8 feet, see Figure B-13 (PPDT).
Date performed: 12/10/15.
Ground surface elevation: 102.5 feet, San Mateo City datum plus 100 feet.

CONCAR PROPERTY
San Mateo, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 01/05/16	Project No. 770626301	Figure B-1

LANGAN TREADWELL ROLLO



Terminated at a depth of 65 feet.
Groundwater calculated at a depth of 18.2 feet, see Figure B-14 (PPDT).
Date performed: 12/10/15.
Ground surface elevation: 102.0 feet, San Mateo City datum plus 100 feet.

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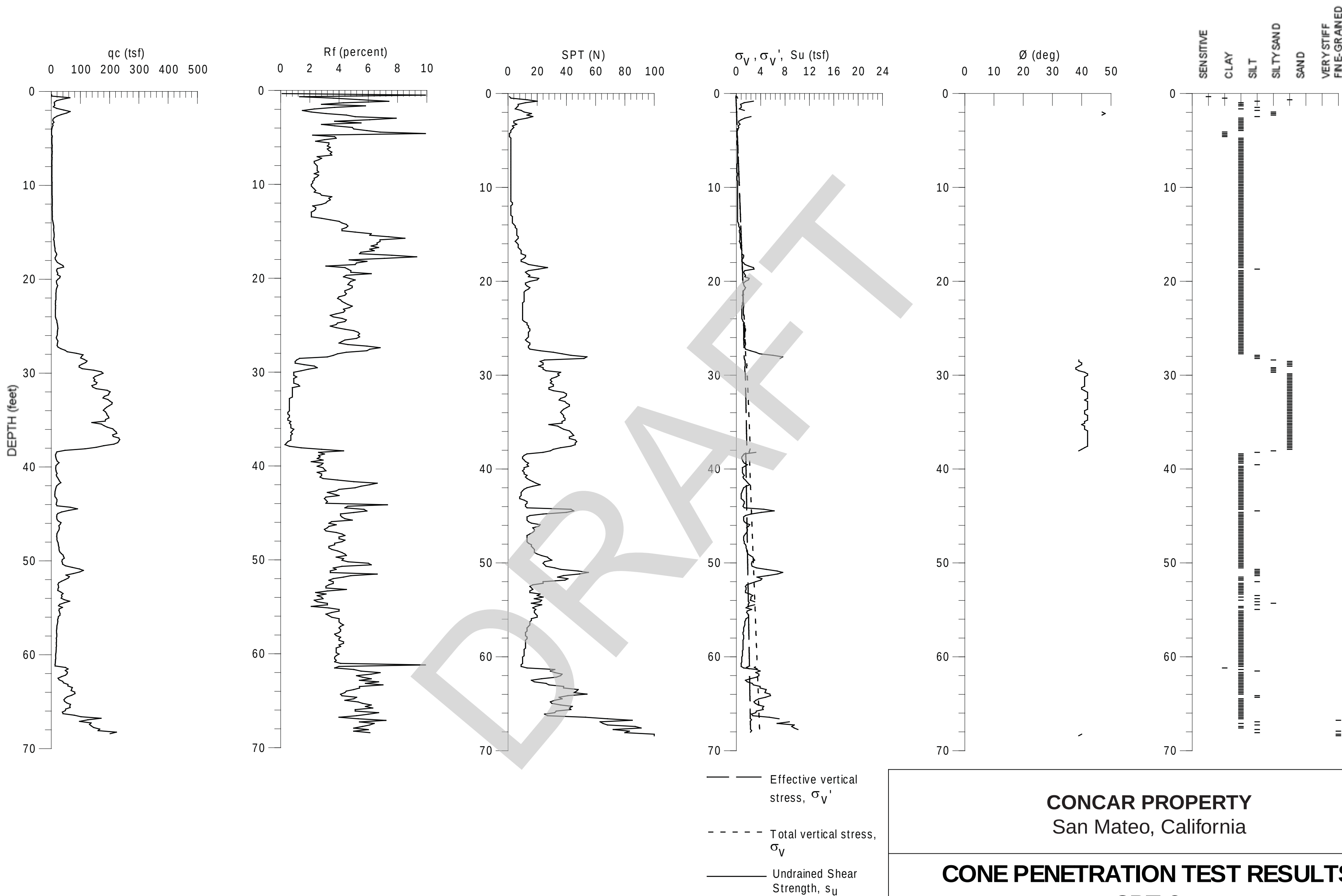
CONE PENETRATION TEST RESULTS
CPT-2

Date 01/05/16	Project No. 770626301	Figure B-2

LANGAN TREADWELL ROLLO



Terminated at a depth of 68.4 feet.
Groundwater not measured.
Date performed: 12/11/15.
Ground surface elevation: 103.2 feet, San Mateo City datum plus 100 feet.

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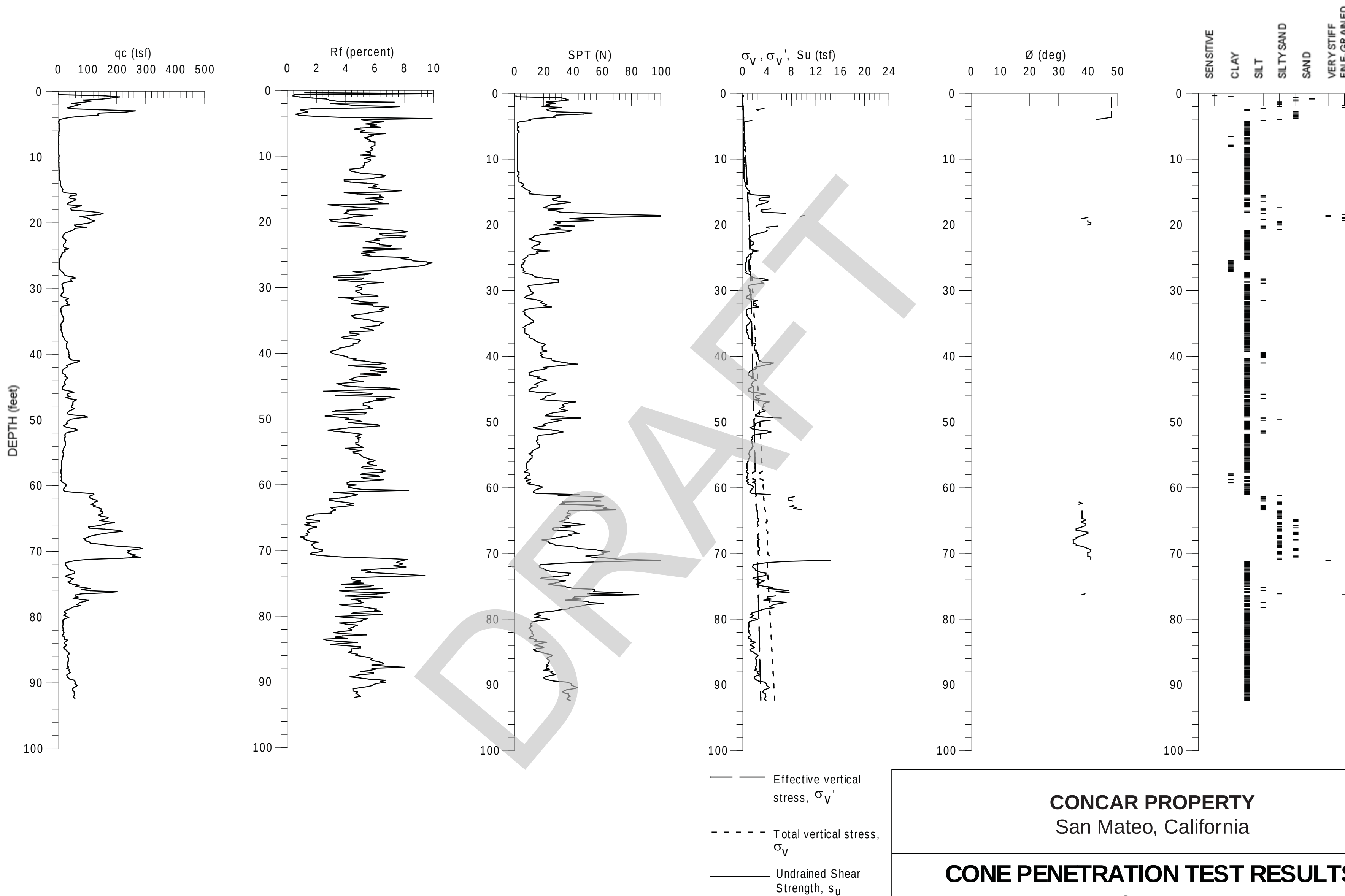
CONE PENETRATION TEST RESULTS
CPT-3

Date 01/05/16	Project No. 770626301	Figure B-3

LANGAN TREADWELL ROLLO



Terminated at a depth of 92.4 feet.
Groundwater not measured.
Date performed: 12/10/15.
Ground surface elevation: 103.2 feet, San Mateo City datum plus 100 feet.

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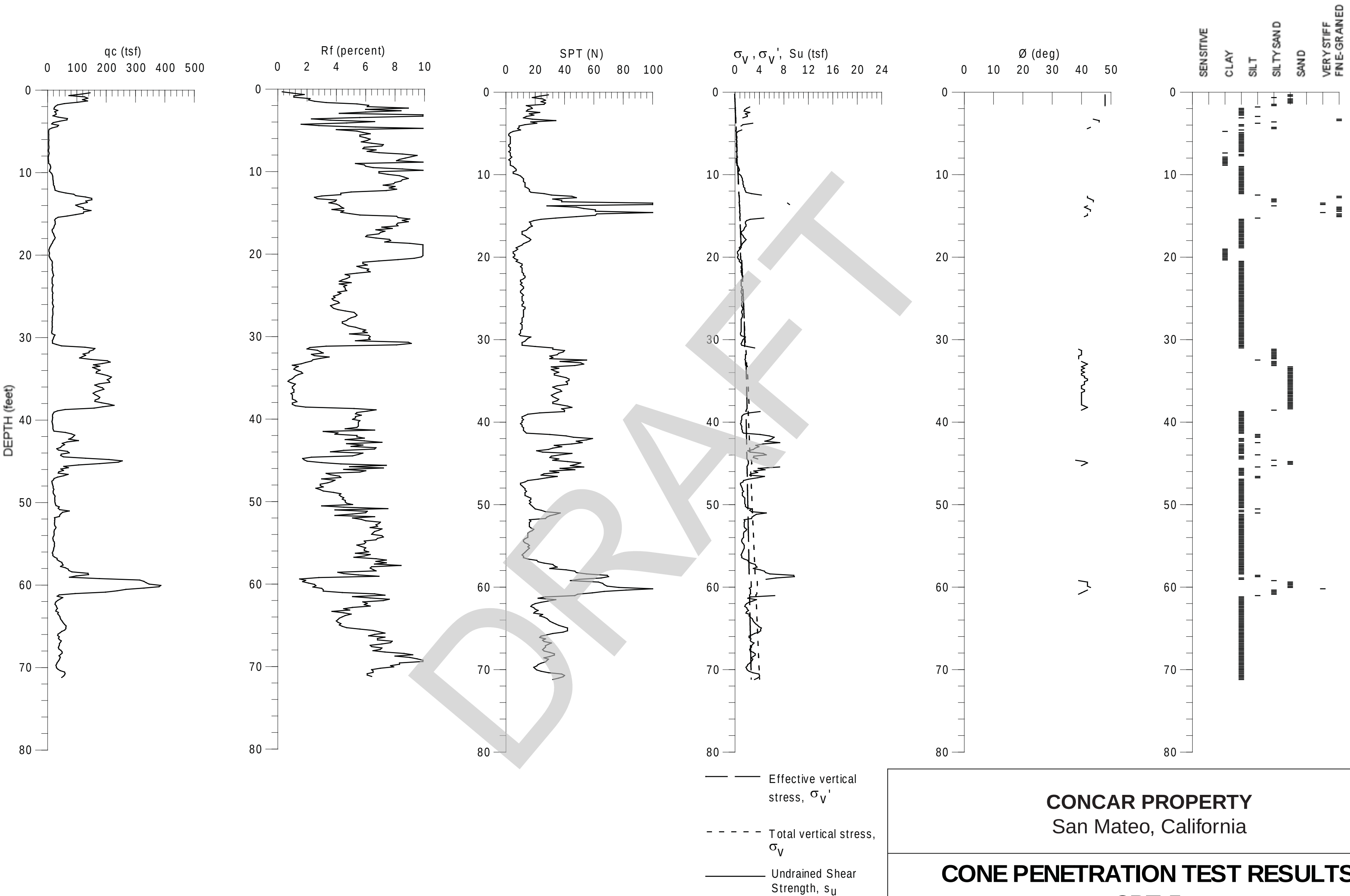
CONE PENETRATION TEST RESULTS
CPT-4

Date 01/05/16	Project No. 770626301	Figure B-4

LANGAN TREADWELL ROLLO



Terminated at a depth of 71.2 feet.
Groundwater calculated at a depth of 27.5 feet, see Figure B-15 (PPDT).
Date performed: 12/10/15.
Ground surface elevation: 103.4 feet, San Mateo City datum plus 100 feet.

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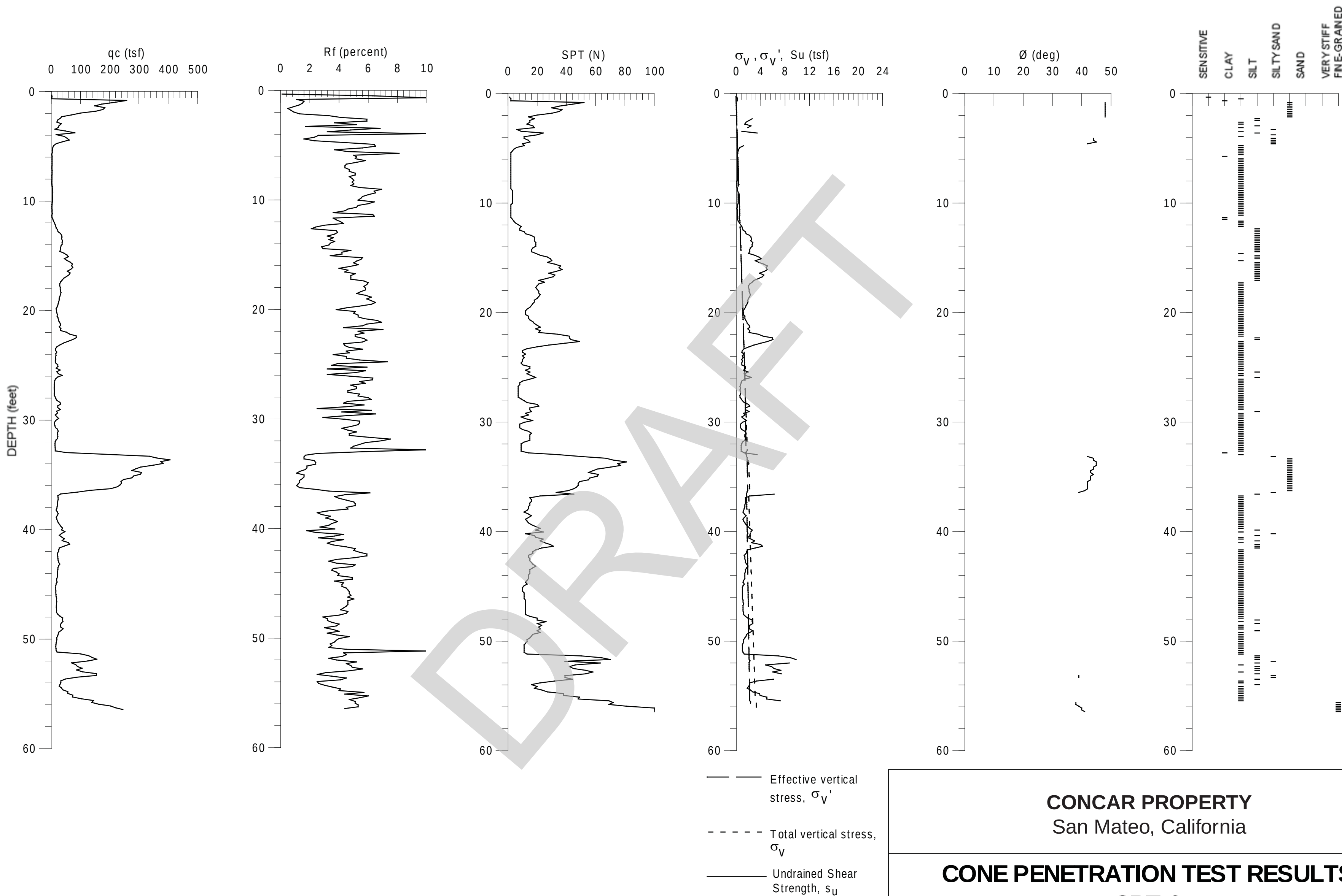
CONE PENETRATION TEST RESULTS
CPT-5

Date 01/05/16	Project No. 770626301	Figure B-5

LANGAN TREADWELL ROLLO



Terminated at a depth of 56.4 feet.
Groundwater calculated at at depth of 25.9 feet, see Figure B-16 (PPDT).
Date performed: 12/10/15.
Ground surface elevation: 102.5 feet, San Mateo City datum plus 100 feet.

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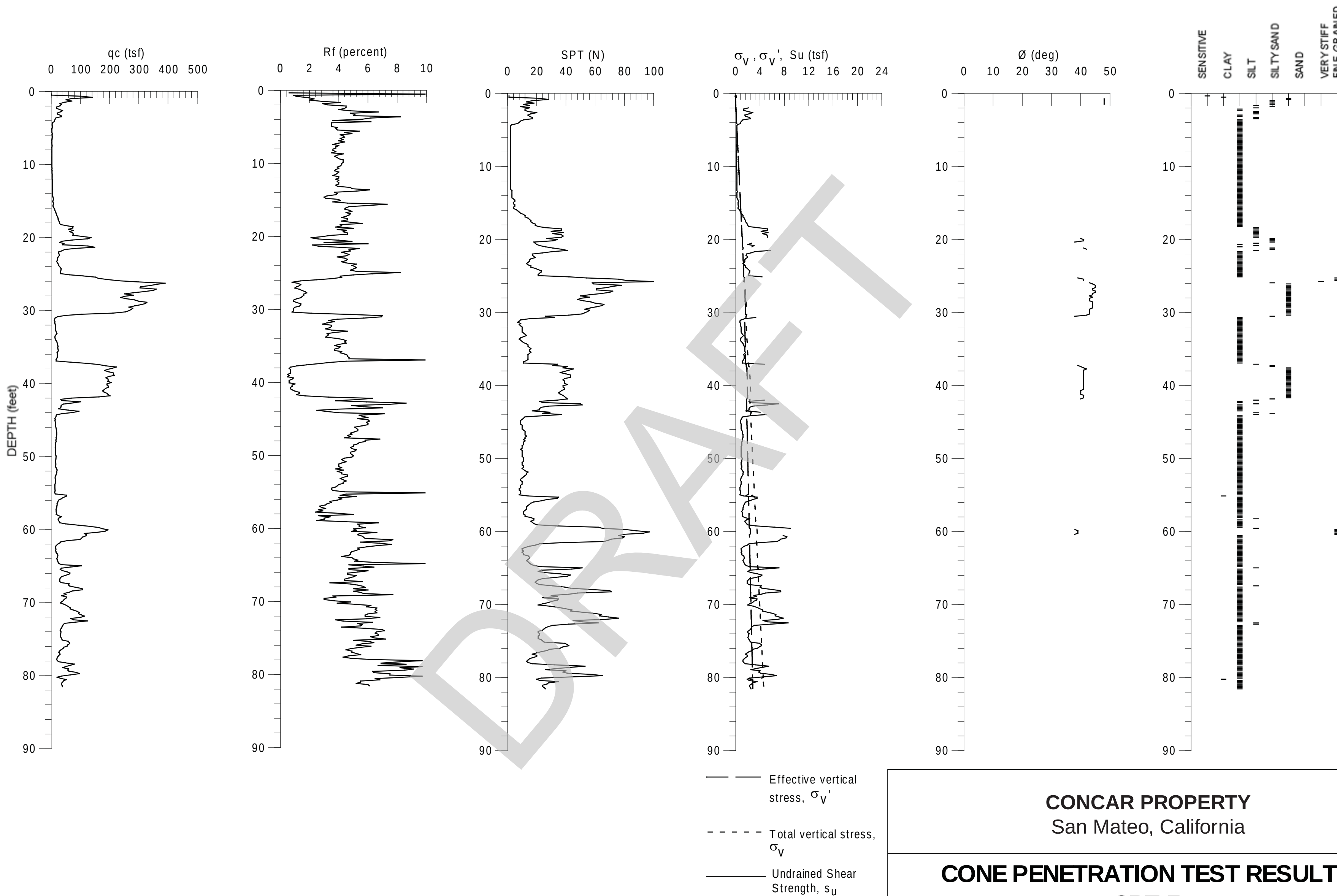
CONE PENETRATION TEST RESULTS
CPT-6

Date 01/05/16	Project No. 770626301	Figure B-6

LANGAN TREADWELL ROLLO



Terminated at a depth of 81.5 feet.
Groundwater calculated at a depth of 24.0 feet, see Figure B-17 (PPDT).
Date performed 12/10/15.
Ground surface elevlation: 102.5 feet, San Mateo City datum plus 100 feet.

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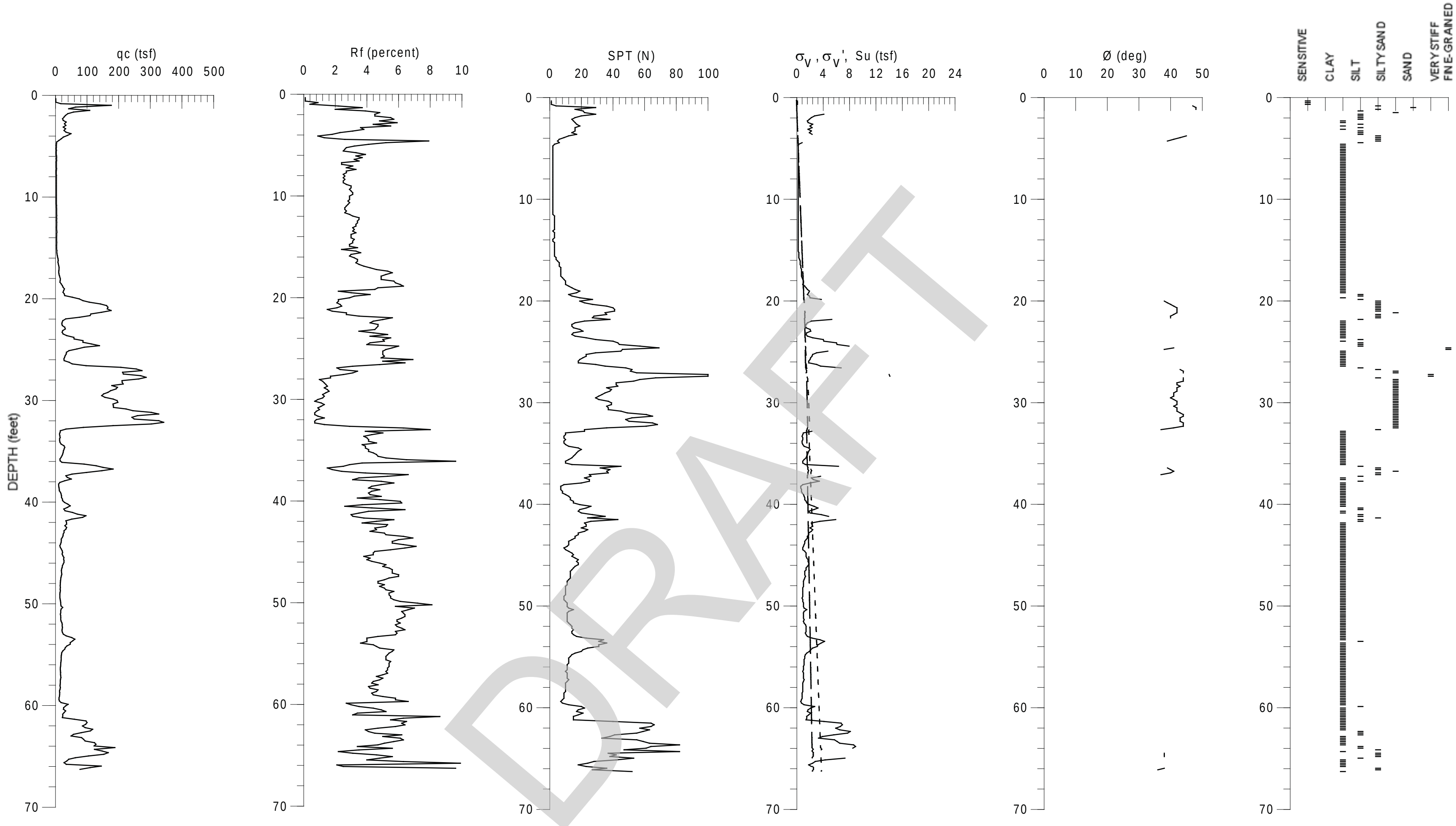
CONE PENETRATION TEST RESULTS
CPT-7

Date 01/05/16	Project No. 770626301	Figure B-7

LANGAN TREADWELL ROLLO



Terminated at a depth of 66.3 feet.
Groundwater calculated at a depth of 22.6 feet, see Figure B-18 (PPDT).
Date performed 12/11/15.
Ground surface elevlation: 102.3 feet, San Mateo City datum plus 100 feet.

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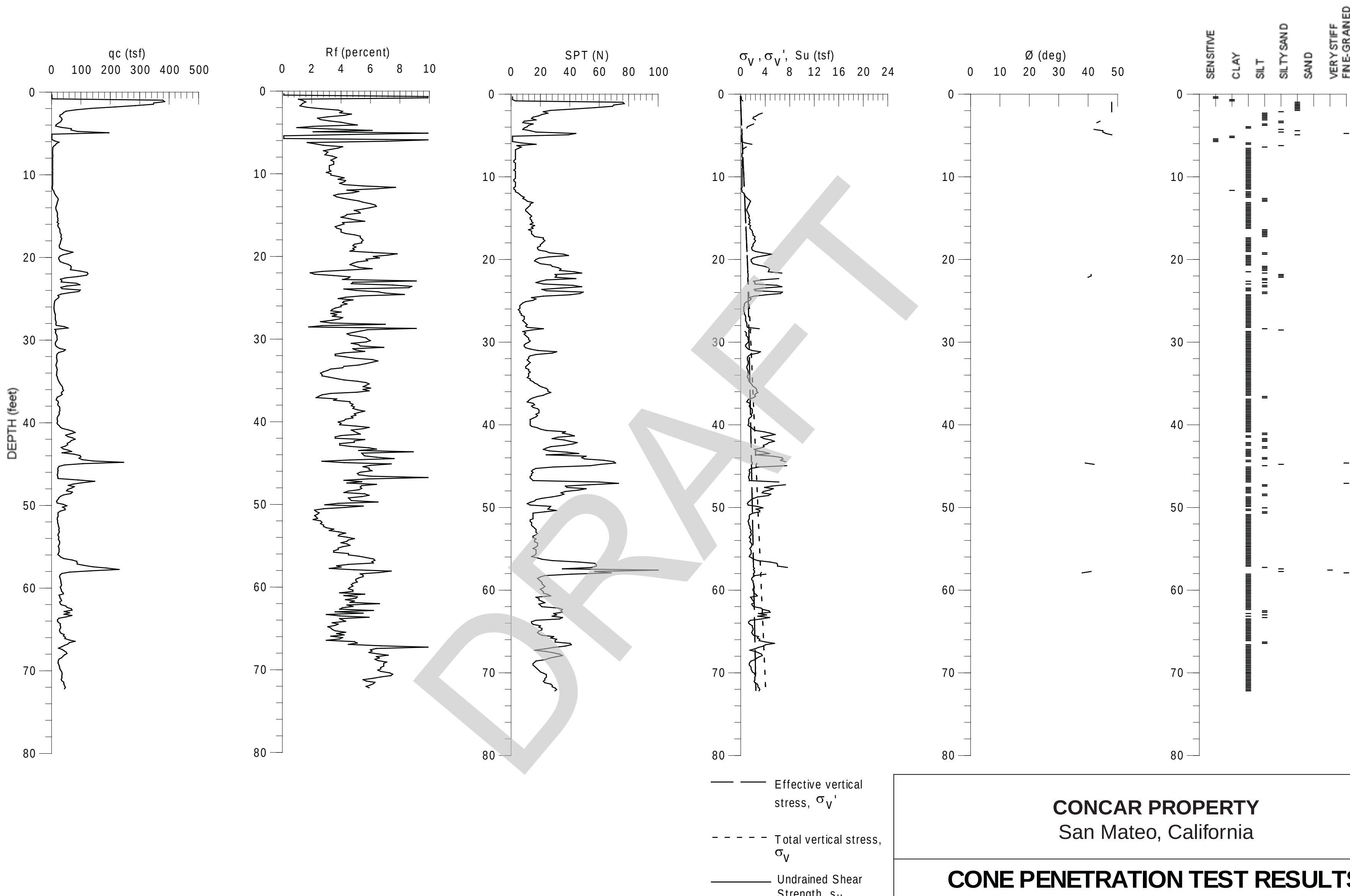
CONE PENETRATION TEST RESULTS
CPT-8

Date 01/05/16	Project No. 770626301	Figure B-8

LANGAN TREADWELL ROLLO



DRAFT

Terminated at a depth of 72.2 feet.
Groundwater calculated at a depth of 21.0 feet, see Figure B-19 (PPDT).
Date performed 12/11/15.
Ground surface elevlation: 102.4 feet, San Mateo City datum plus 100 feet.

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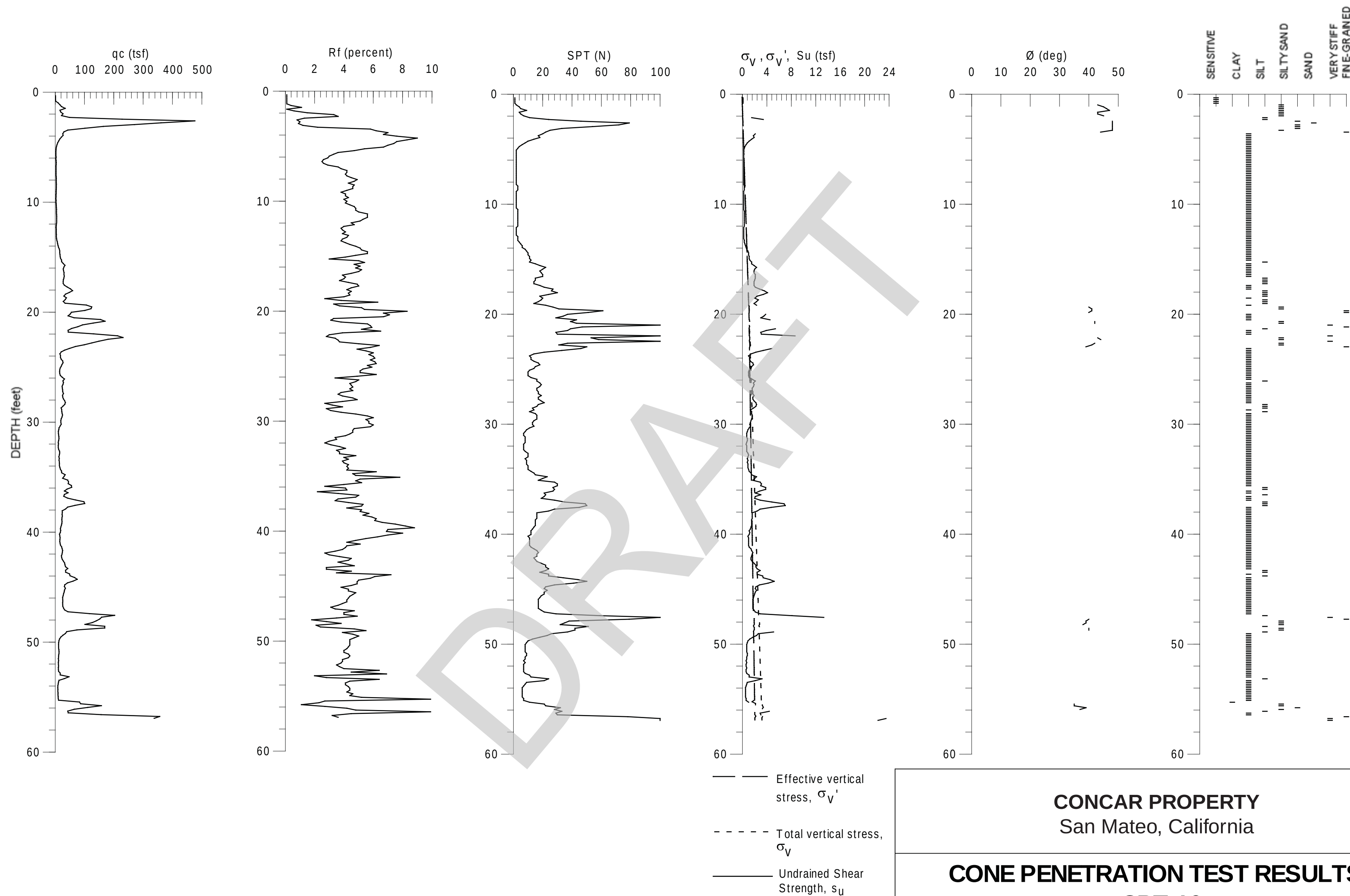
CONE PENETRATION TEST RESULTS
CPT-9

Date 01/05/16	Project No. 770626301	Figure B-9

LANGAN TREADWELL ROLLO



Terminated at a depth of 56.9 feet.
Groundwater not measured.
Date performed 12/11/15.
Ground surface elevlation: 102.8 feet, San Mateo City datum plus 100 feet.

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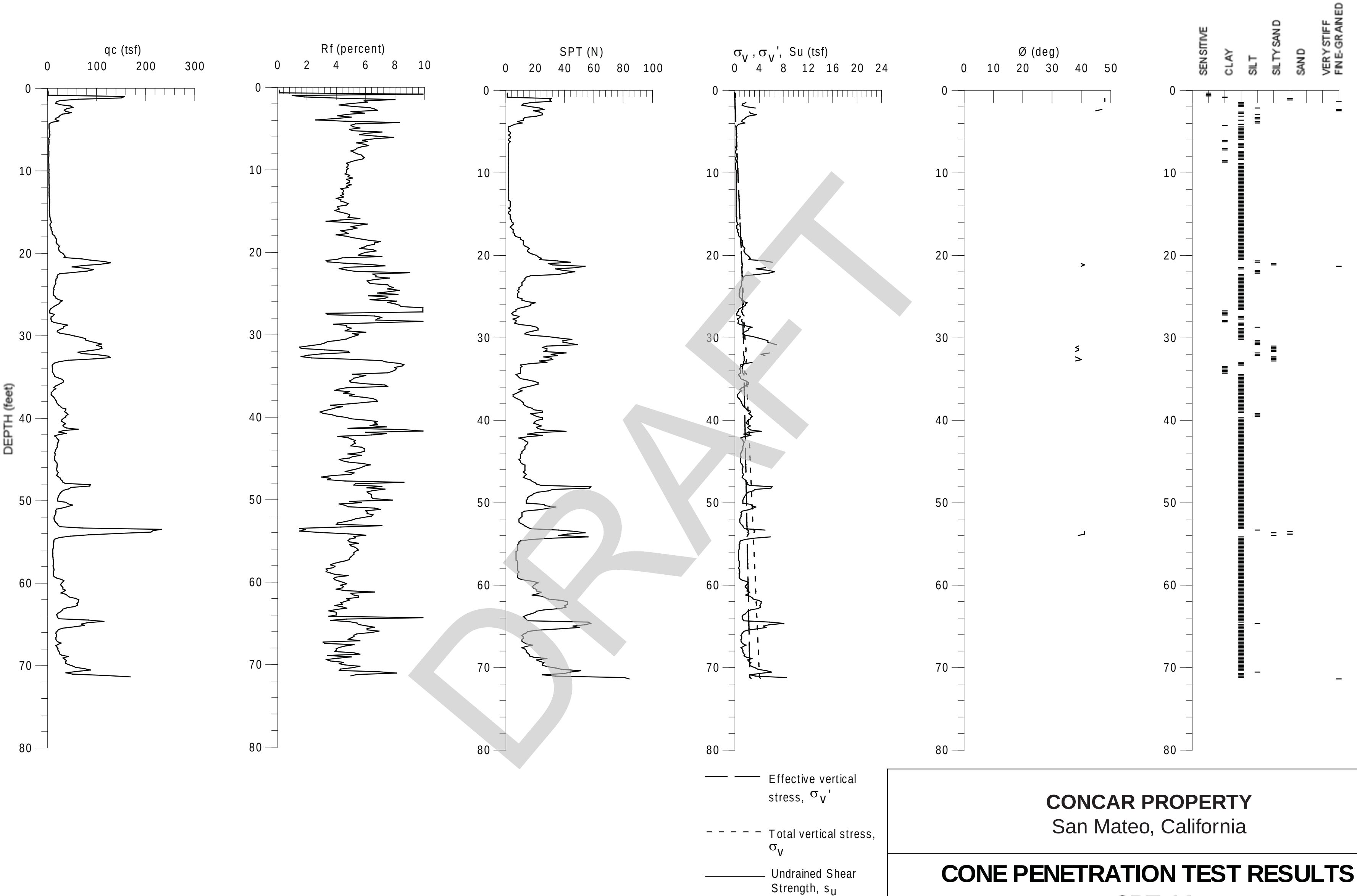
CONE PENETRATION TEST RESULTS
CPT-10

Date 01/05/16	Project No. 770626301	Figure B-10

LANGAN TREADWELL ROLLO



Terminated at a depth of depth of 71.4 feet.
Groundwater calculated at a depth of 21.5 feet, see Figure B-20 (PPDT).
Date performed: 12/11/15.
Ground surface elevation: 102.6 feet, San Mateo City datum plus 100 feet.

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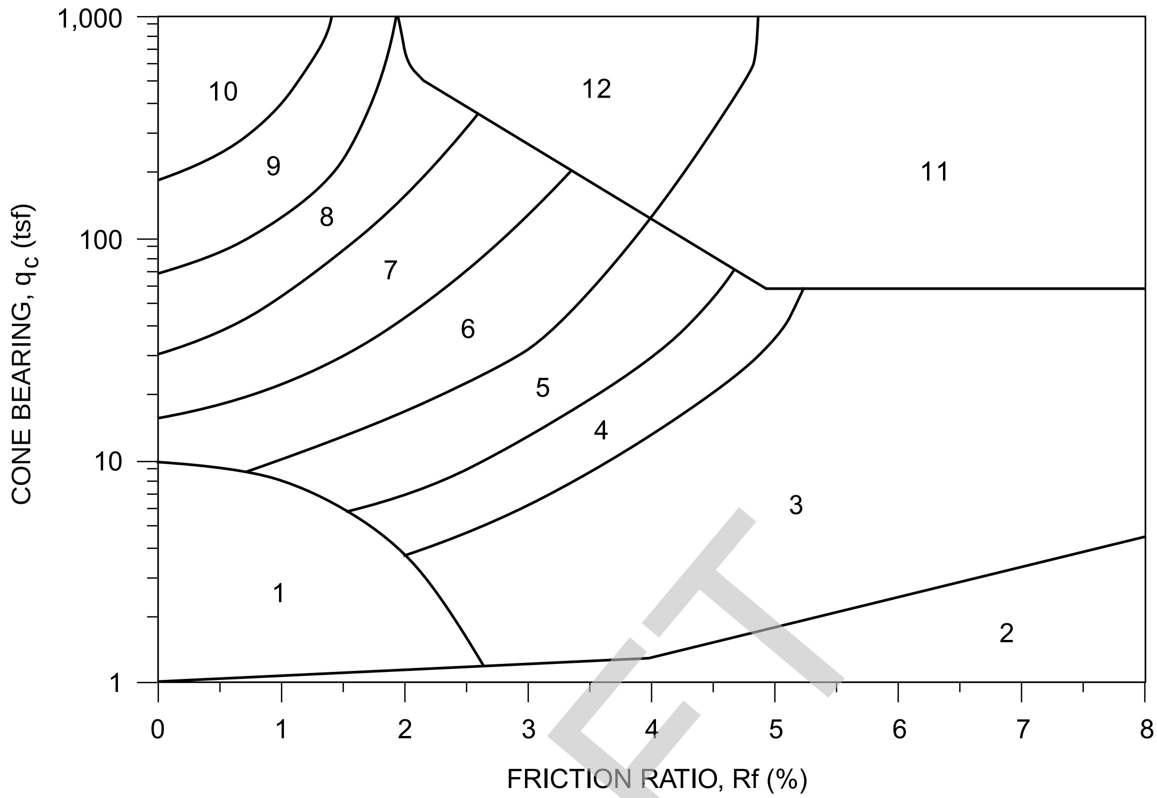
San Mateo, California

CONE PENETRATION TEST RESULTS
CPT-11

Date 01/05/16	Project No. 770626301	Figure B-11

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
$Rf = f_s/q_c \times 100$ = 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 $q_c > 9$).

DRAFT

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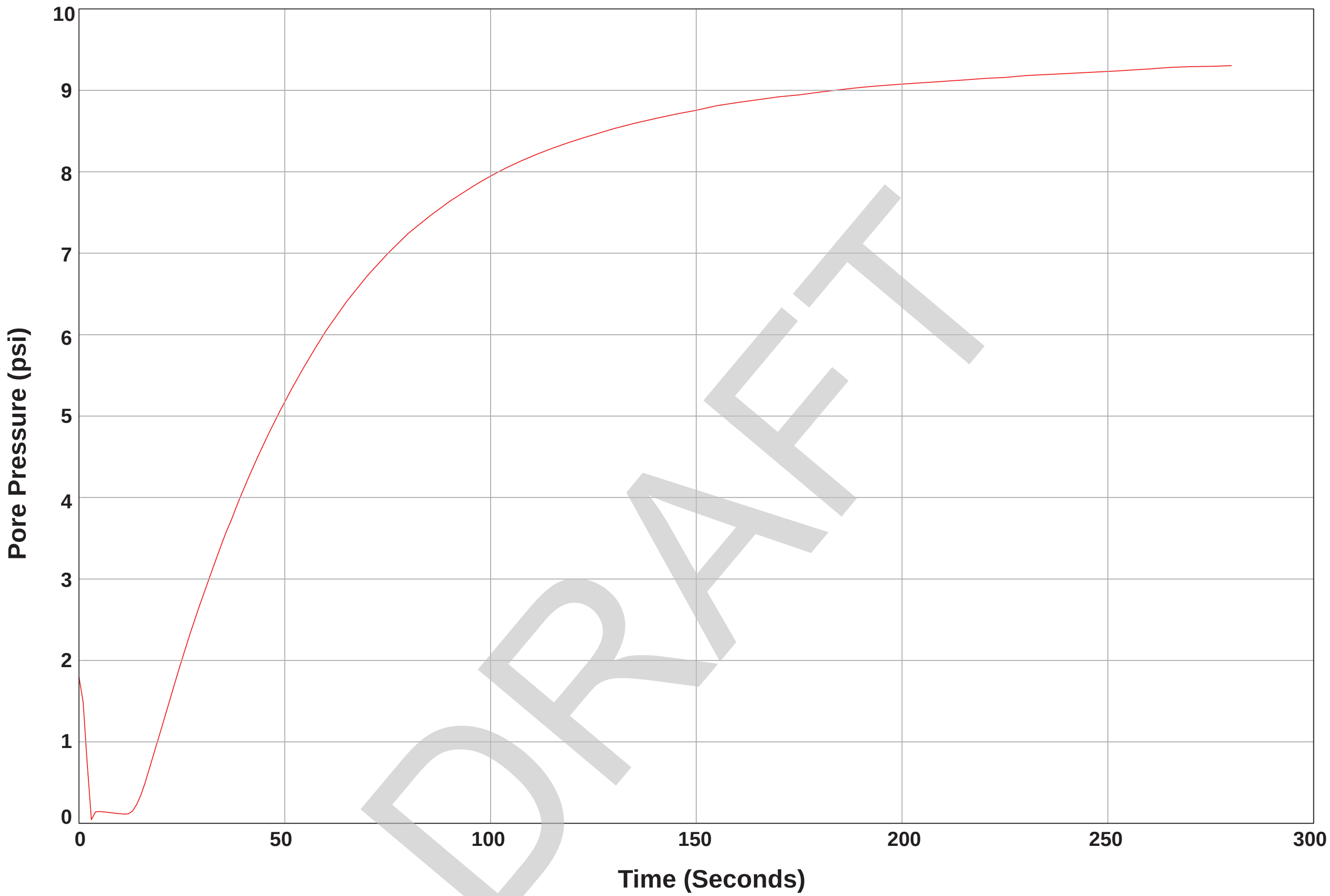
LANGAN TREADWELL ROLLO

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 01/15/16 | Project No. 770626301 | Figure B-12



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-1	42.3	9.3	21.5	20.8	81.7

Note:
1. Ground surface elevlation: 102.5 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/10/15.

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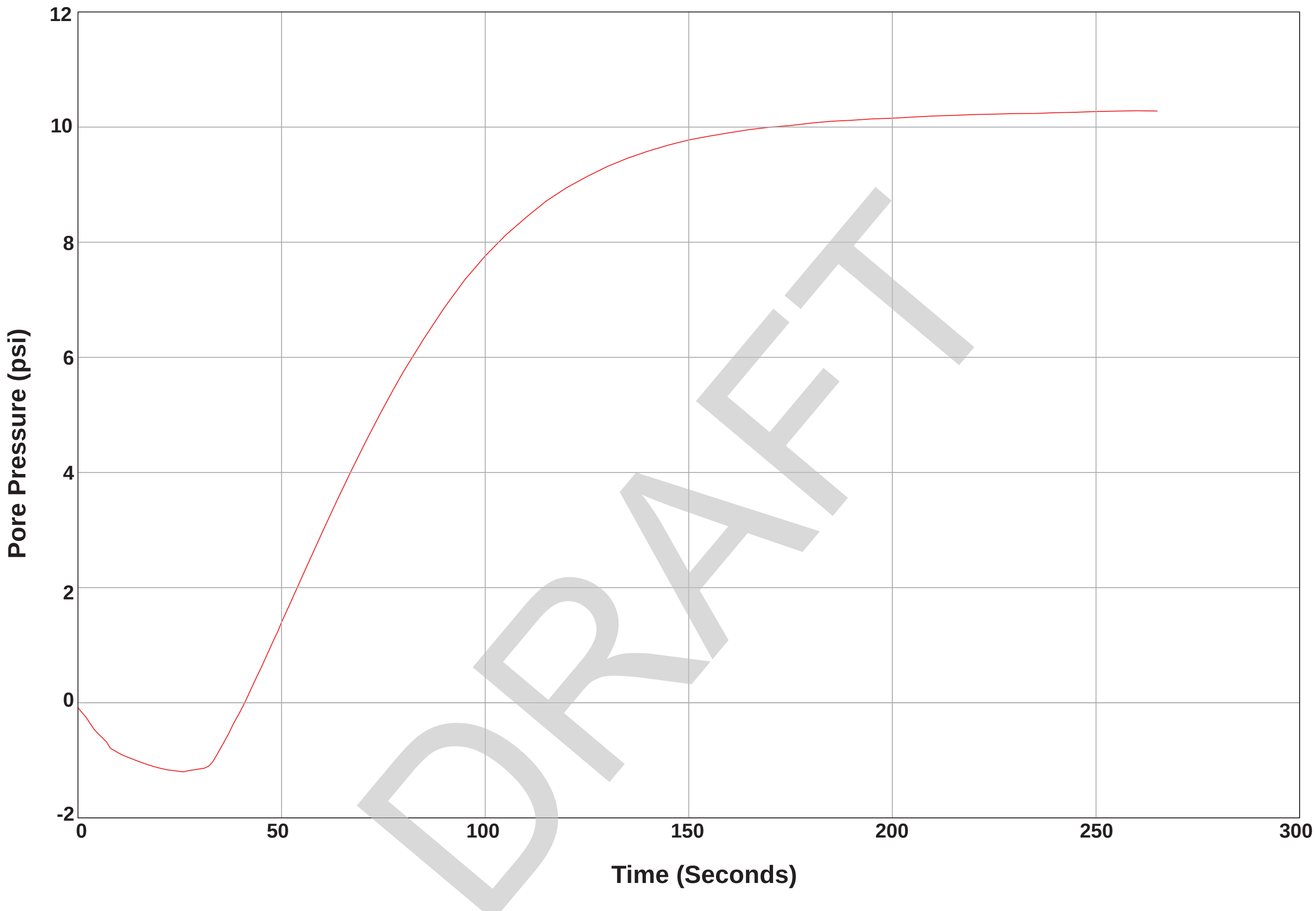
San Mateo, California

PORE PRESSURE DISSIPATION TEST
CPT-1

Date 01/06/15	Project No. 770626301	Figure B-13

LANGAN TREADWELL ROLLO



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-2	41.7	10.2	23.5	18.2	83.8

Note:
1. Ground surface elevlation: 102.0 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/10/15.

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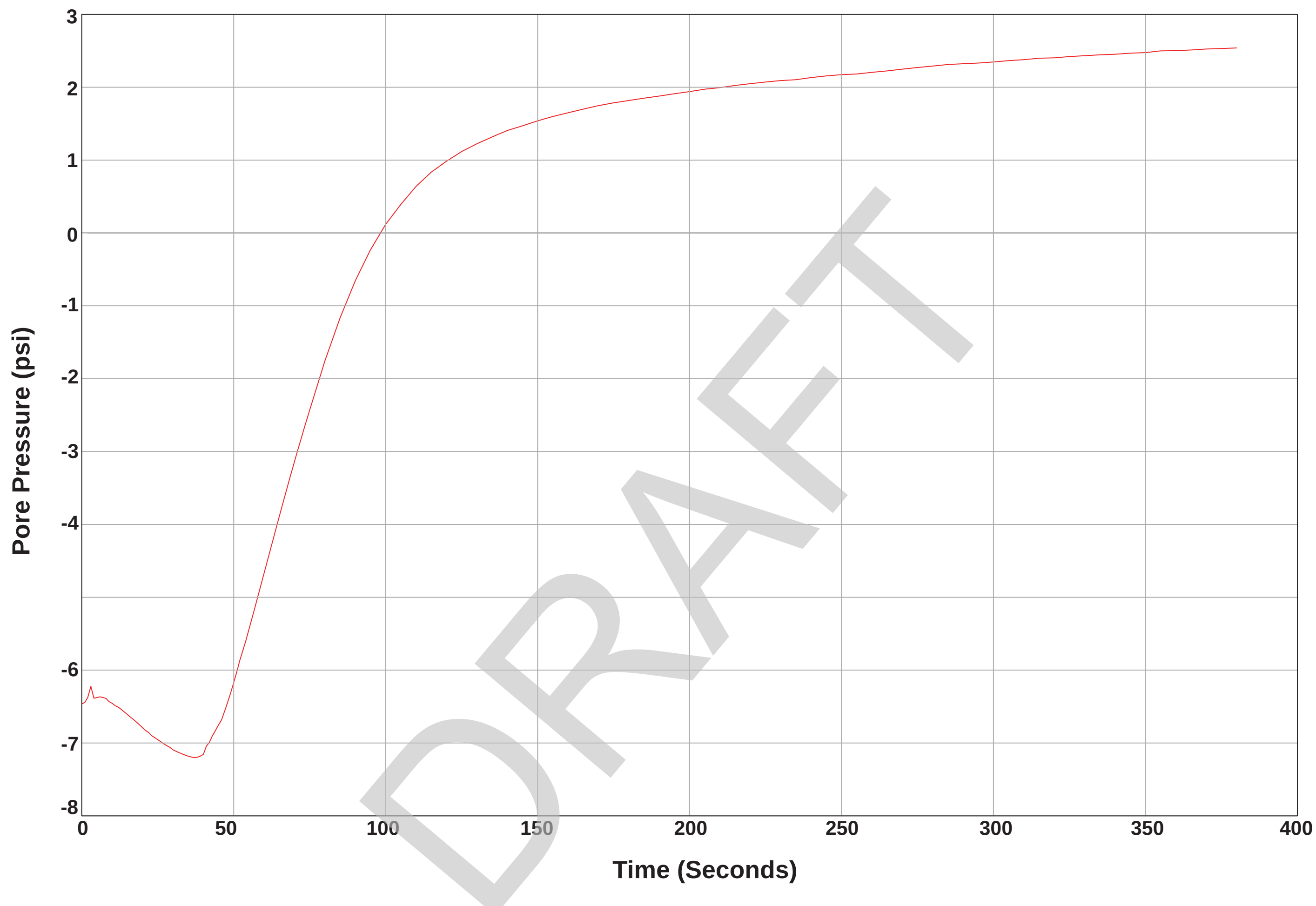
PORE PRESSURE DISSIPATION TEST
CPT-2

Date 01/06/15	Project No. 770626301	Figure B-14

LANGAN TREADWELL ROLLO



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-5	33.3	2.5	5.8	27.5	75.9

Note:

1. Ground surface elevation: 103.4 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/10/15.

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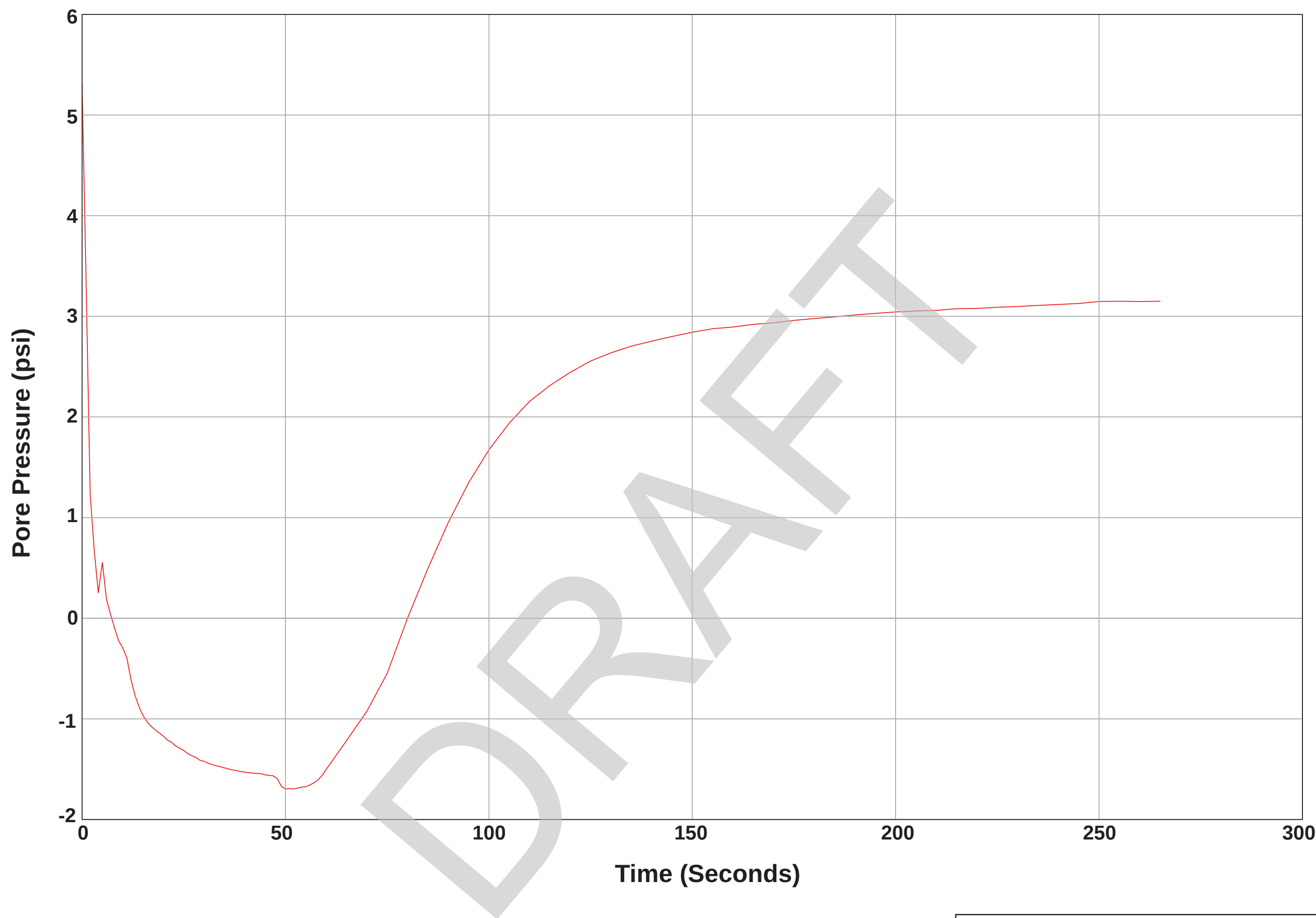
PORE PRESSURE DISSIPATION TEST
CPT-5

Date 01/06/15 | Project No. 770626301 | Figure B-15

LANGAN TREADWELL ROLLO



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-6	33.1	3.1	7.2	25.9	76.6

Note:
1. Ground Surface Elevation: 102.5 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/10/15.

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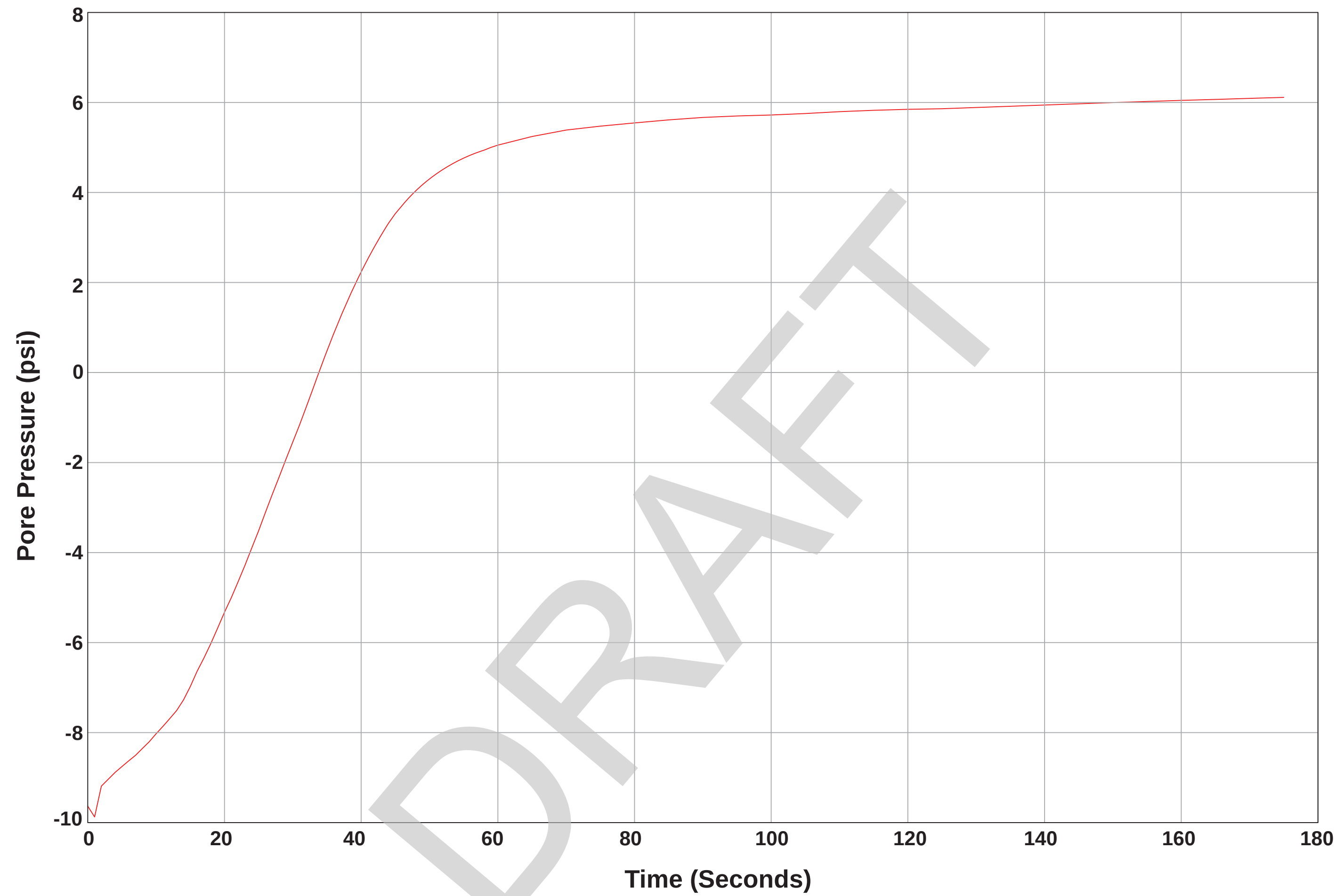
PORE PRESSURE DISSIPATION TEST
CPT-6

Date 01/06/15	Project No. 770626301	Figure B-16

LANGAN TREADWELL ROLLO



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-7	38.1	6.1	14.1	24.0	78.5

Note:
1. Ground Surface Elevation: 102.5 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/10/15.

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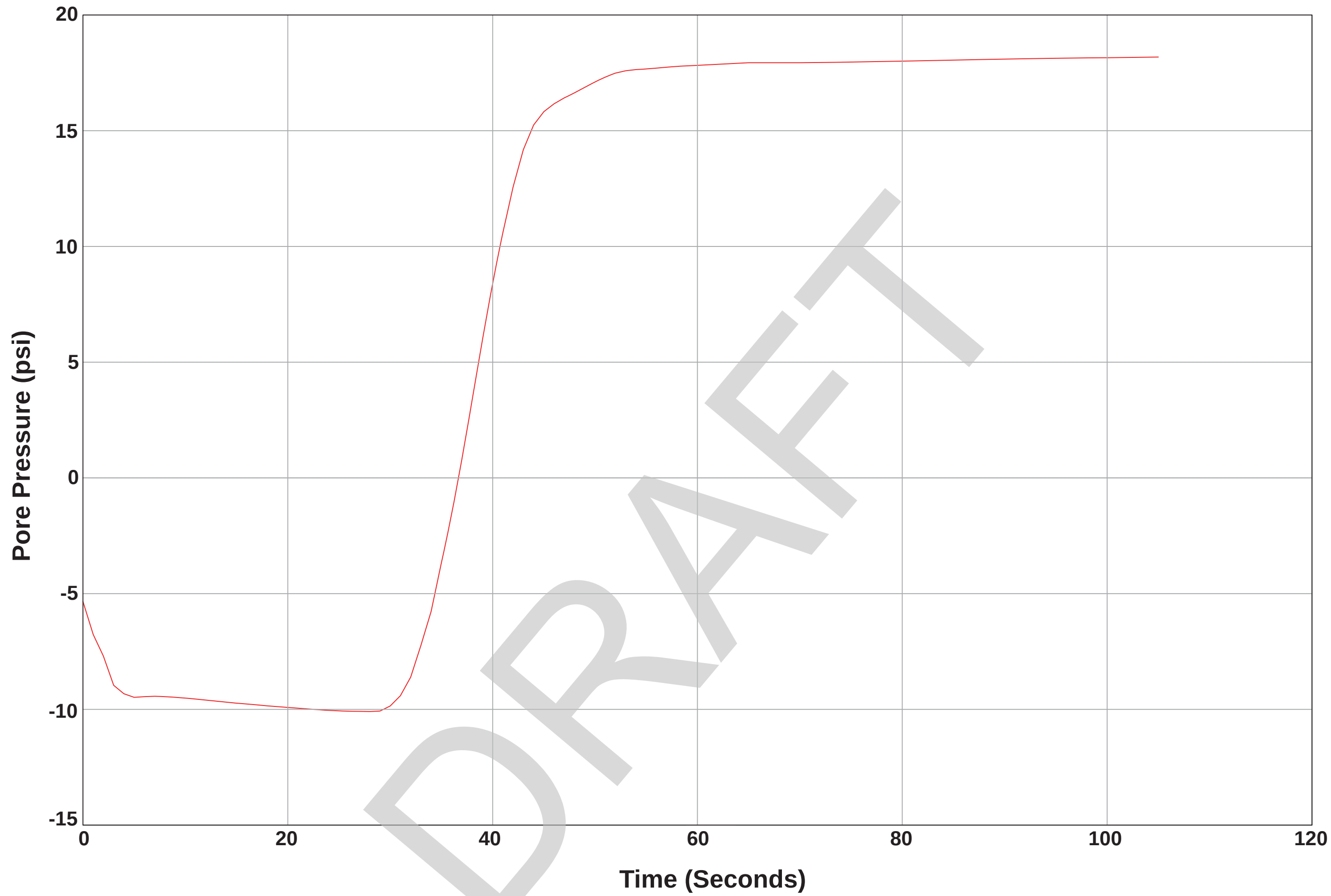
PORE PRESSURE DISSIPATION TEST
CPT-7

Date 01/06/15	Project No. 770626301	Figure B-17

LANGAN TREADWELL ROLLO



DRAFT

CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-8	64.1	18.0	41.5	22.6	79.7

Note:
1. Ground Surface Elevation: 102.3 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/11/15.

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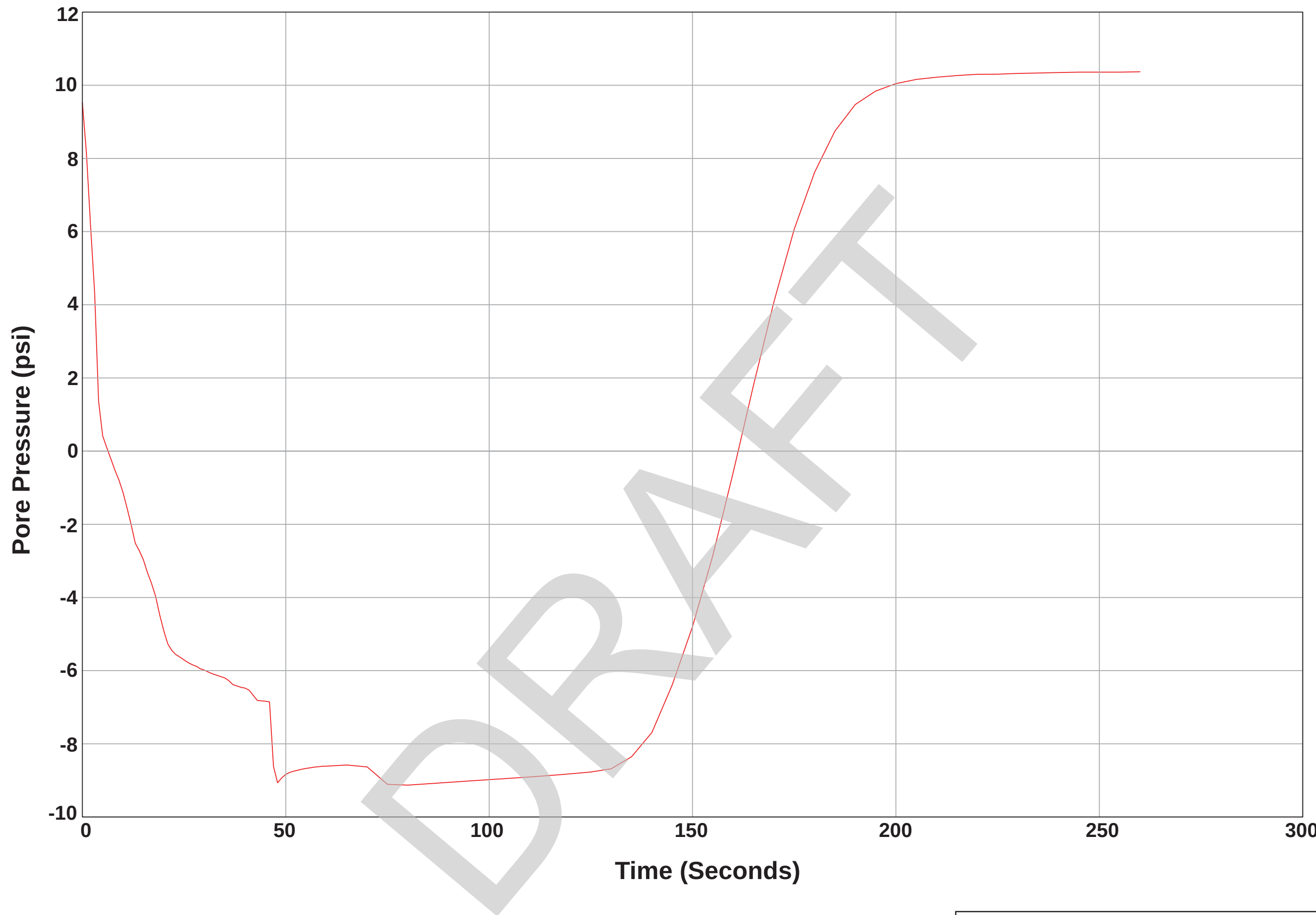
PORE PRESSURE DISSIPATION TEST
CPT-8

Date 01/06/15	Project No. 770626301	Figure B-18

LANGAN TREADWELL ROLLO



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-9	44.8	10.3	23.8	21.0	81.4

Note:
1. Ground Surface Elevation: 102.4 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/11/15.

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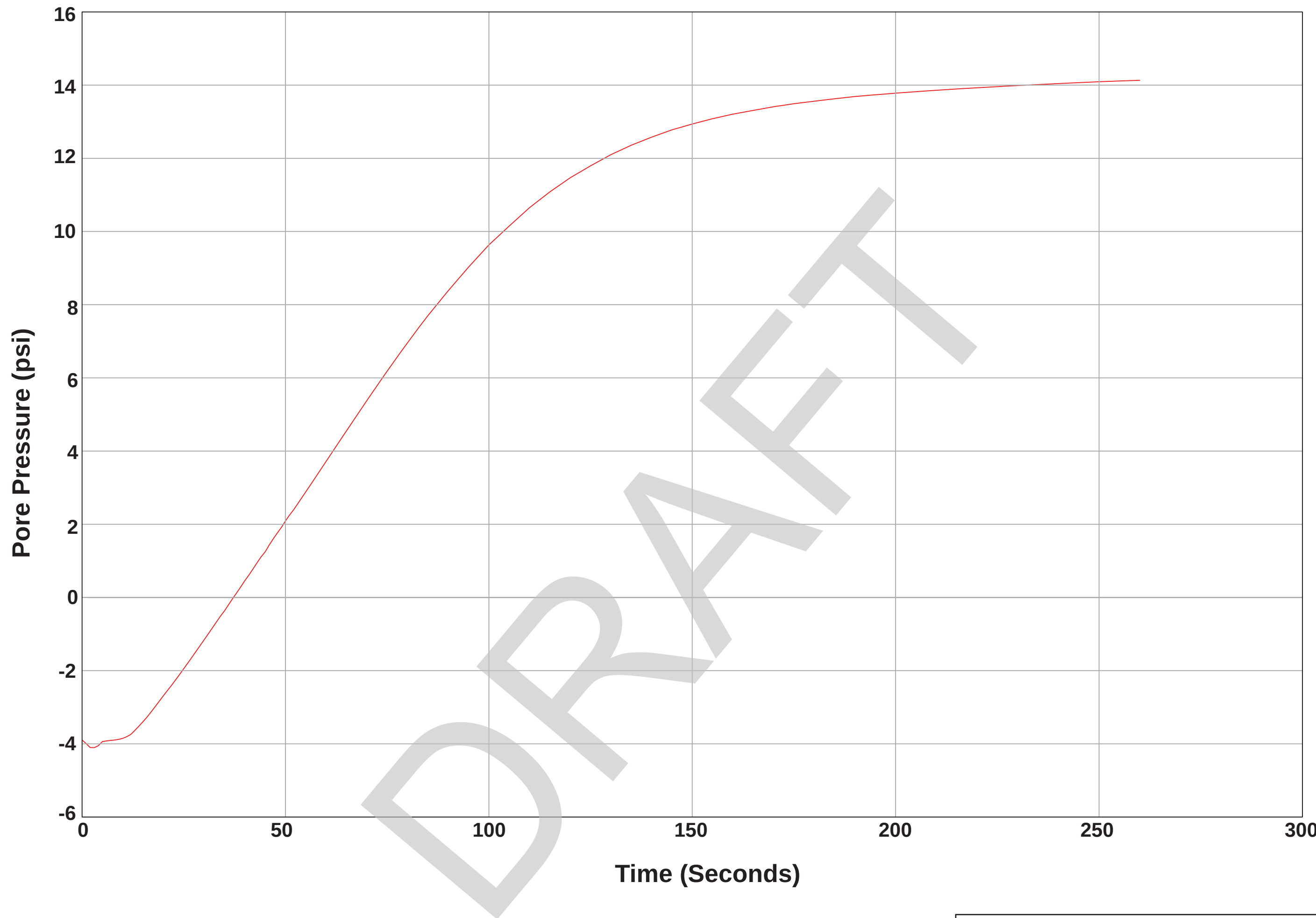
PORE PRESSURE DISSIPATION TEST
CPT-9

Date 01/06/15 | Project No. 770626301 | Figure B-19

LANGAN TREADWELL ROLLO



CPT (no.)	Approximate Depth (feet)	End Point (psi)	Calculated Head (feet)	Calculated GW Depth (feet)	Calculated GW Elevation (feet)
CPT-11	54.0	14.1	32.5	21.5	81.1

Note:
1. Ground Surface Elevation: 102.6 feet, San Mateo City datum plus 100 feet.
2. PPDT performed on 12/11/15.

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PORE PRESSURE DISSIPATION TEST
CPT-11

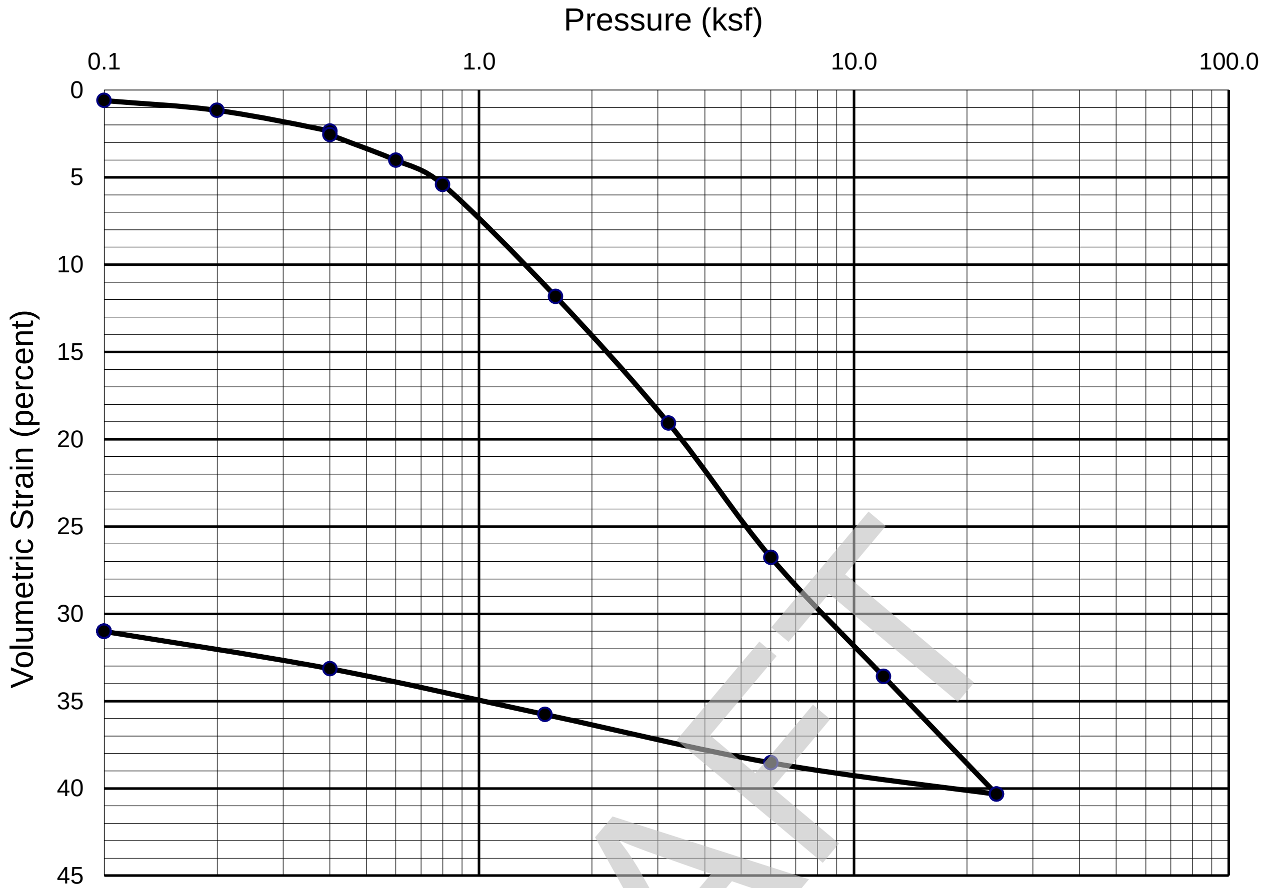
Date 01/06/15 | Project No. 770626301 | Figure B-20

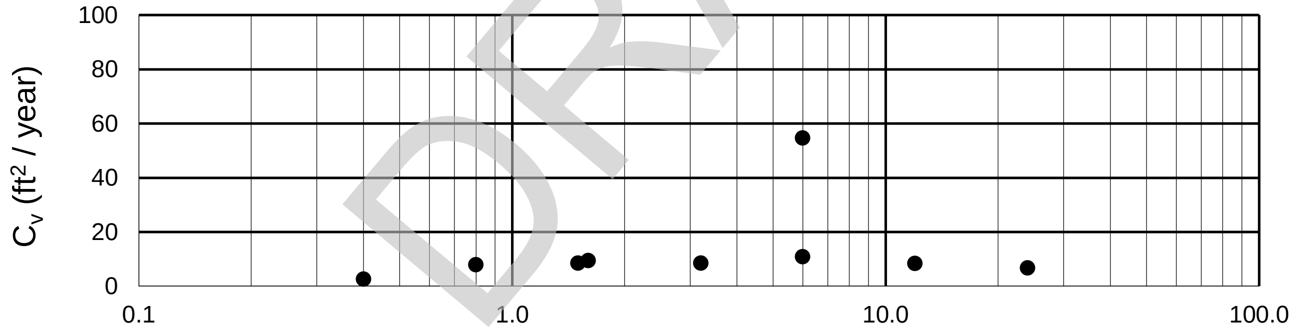
LANGAN TREADWELL ROLLO

APPENDIX C

LABORATORY DATA

DRAFT





Sampler Type: Shelby Tube		Condition	Before Test			After Test		
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	95.1	%	w_f	55.0	%
Overburden Pressure, p_o 860	psf	Void Ratio	e_o	2.50		e_f	1.42	
Preconsol. Pressure, p_c 860	psf	Saturation	S_o	103	%	S_f	105	%
Compression Ratio, $C_{\varepsilon c}$ 0.25		Dry Density	γ_d	48	pcf	γ_d	70	pcf
LL - -	PL - -	PI - -		G_s	2.70	(assumed)		
Classification SILT (MH), olive-gray		Source	B-1 at 7.5 feet					

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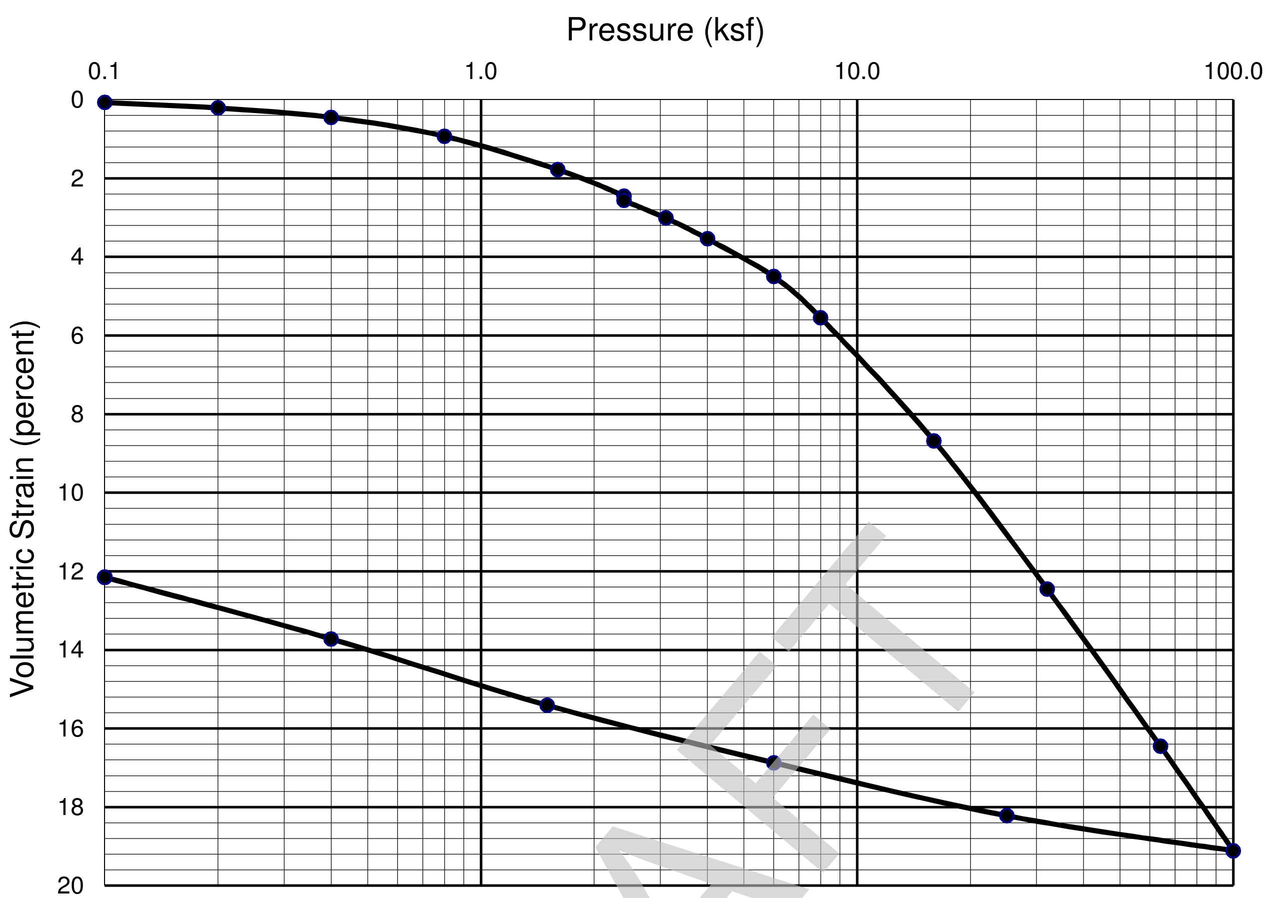
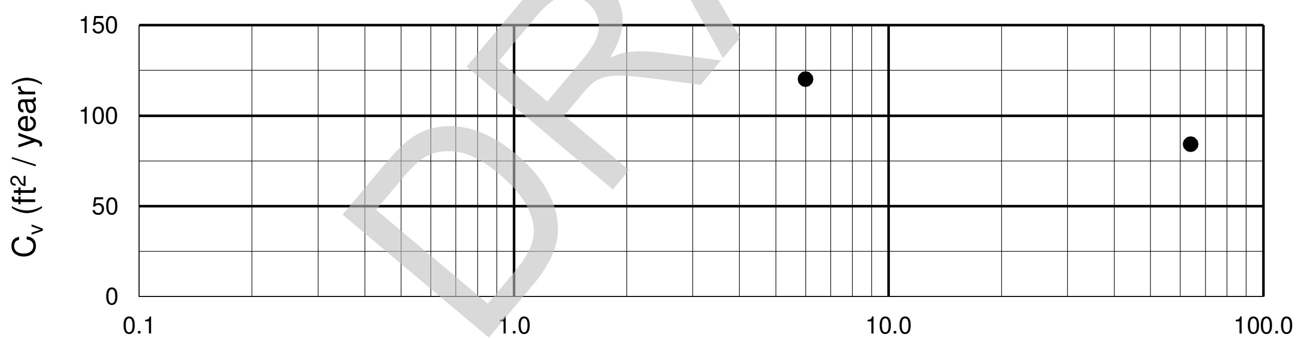
LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date	01/07/16	Project No.	770626301	Figure	C-1





DRAFT

Sampler Type: Dames & Moore			Condition	Before Test		After Test		
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	25.6 %	w_f	20.2 %	
Overburden Pressure, p_o	3,640	psf	Void Ratio	e_o	0.76	e_f	0.55	
Preconsol. Pressure, p_c	7,000	psf	Saturation	S_o	91 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.13		Dry Density	γ_d	96 pcf	γ_d	109 pcf	
LL - -	PL - -		PI - -			G_s	2.70	(assumed)
Classification CLAY with SAND (CL), olive-gray				Source	B-3 at 35 feet			

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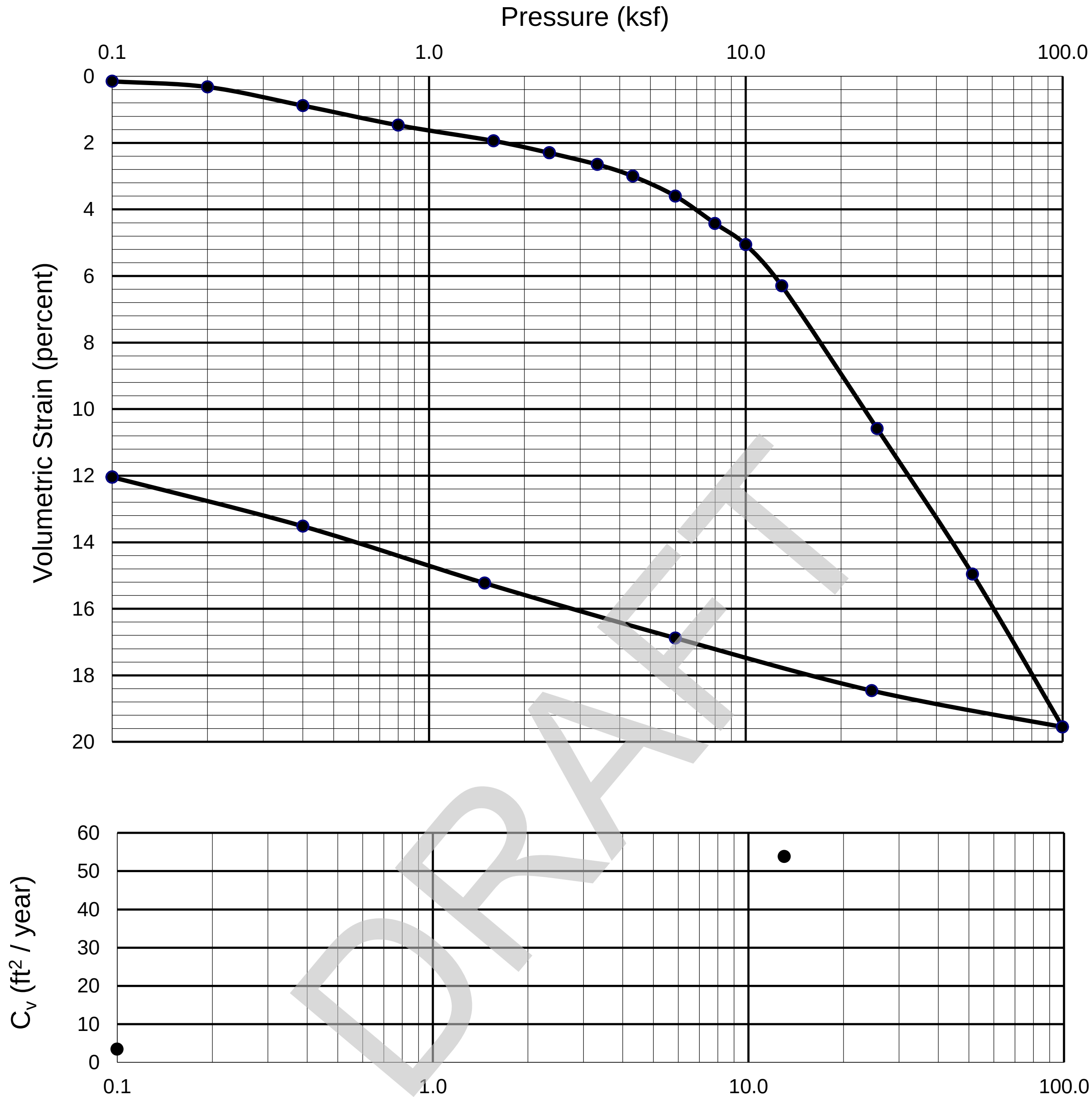
LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date	12/29/15	Project No.	770626301	Figure	C-2



Sampler Type: Shelby Tube				Condition	Before Test		After Test	
Diameter (in) 2.42		Height (in) 1.00		Water Content	w_o	26.9 %	w_f	19.6 %
Overburden Pressure, p_o	2,210	psf		Void Ratio	e_o	0.73	e_f	0.52
Preconsol. Pressure, p_c	10,000	psf		Saturation	S_o	100 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.15			Dry Density	γ_d	98 pcf	γ_d	111 pcf
LL - -		PL - -		PI - -			G_s 2.70	(assumed)
Classification SANDY CLAY (CL), light brown				Source	B-5 at 25 feet			

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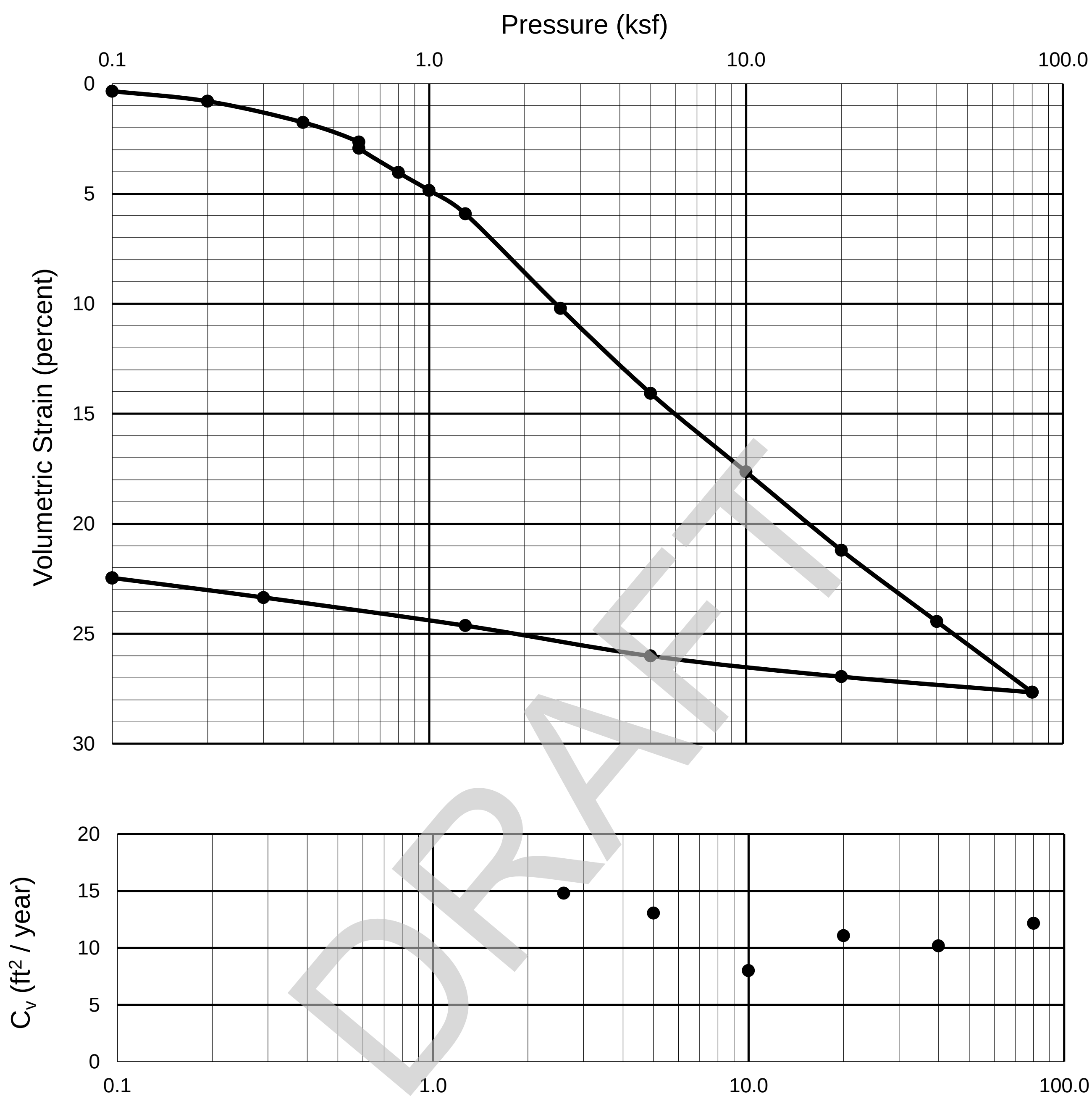
LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date	12/29/15	Project No.	770626301	Figure	C-3



DRAFT

Sampler Type: Shelby Tube				Condition		Before Test		After Test	
Diameter (in) 2.42		Height (in) 1.00		Water Content	w_o	32.8 %		w_f	17.1 %
Overburden Pressure, p_o	1,335	psf		Void Ratio	e_o	0.83		e_f	0.42
Preconsol. Pressure, p_c	710	psf		Saturation	S_o	107 %		S_f	111 %
Compression Ratio, $C_{\varepsilon c}$	0.12			Dry Density	γ_d	92 pcf		γ_d	119 pcf
LL - -		PL - -		PI - -			G_s	2.70	(assumed)
Classification SANDY CLAY (CL), olive				Source		B-6 at 12.5 feet			

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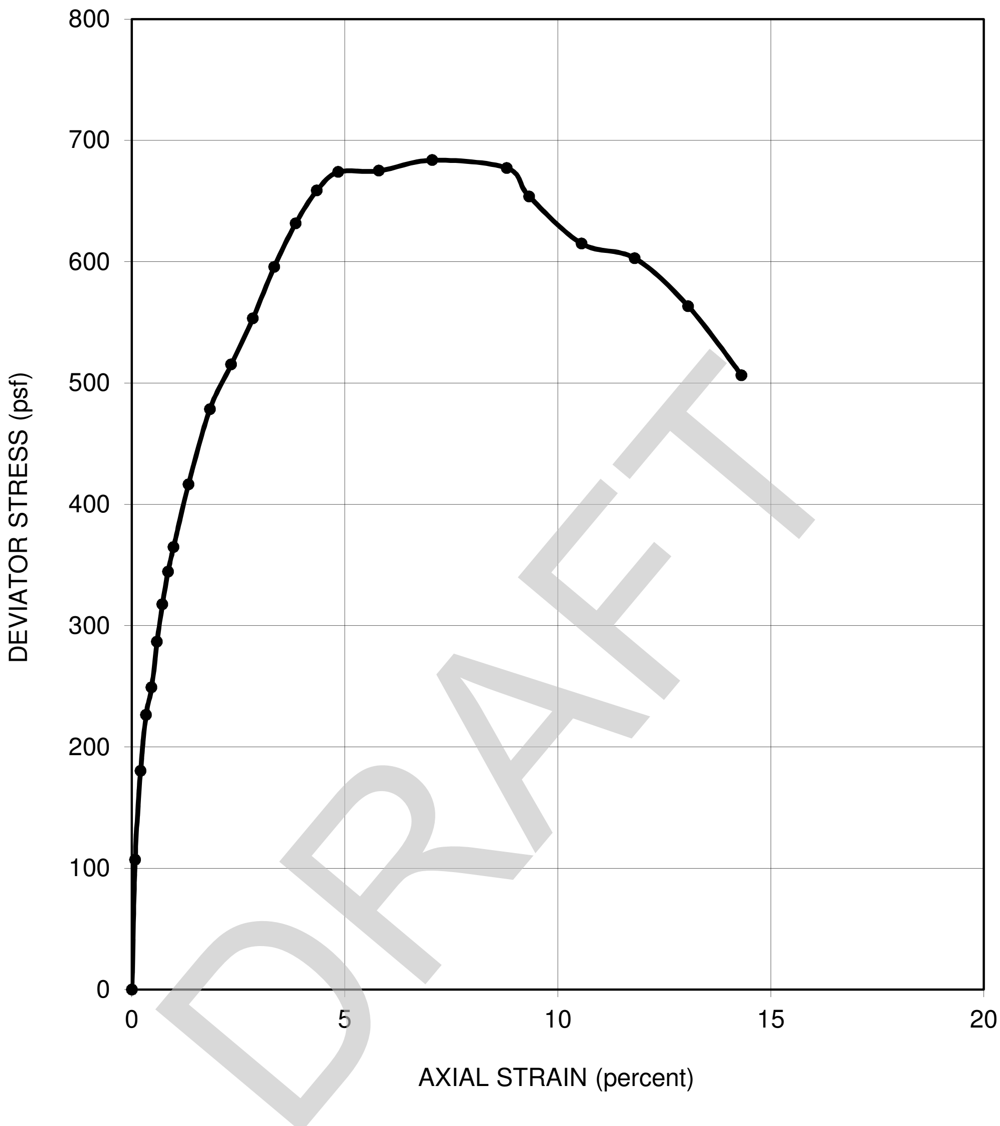
LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date	12/31/15	Project No.	770626301	Figure	C-4



SAMPLER TYPE	Shelby Tube		SHEAR STRENGTH	340	psf
DIAMETER (in.) 2.9	HEIGHT (in.) 6.1		STRAIN AT FAILURE	7.0	%
MOISTURE CONTENT	84.3	%	CONFINING PRESSURE	900	psf
DRY DENSITY	51	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SILT (MH), olive-gray		SOURCE	B-1 at 7.5 feet	

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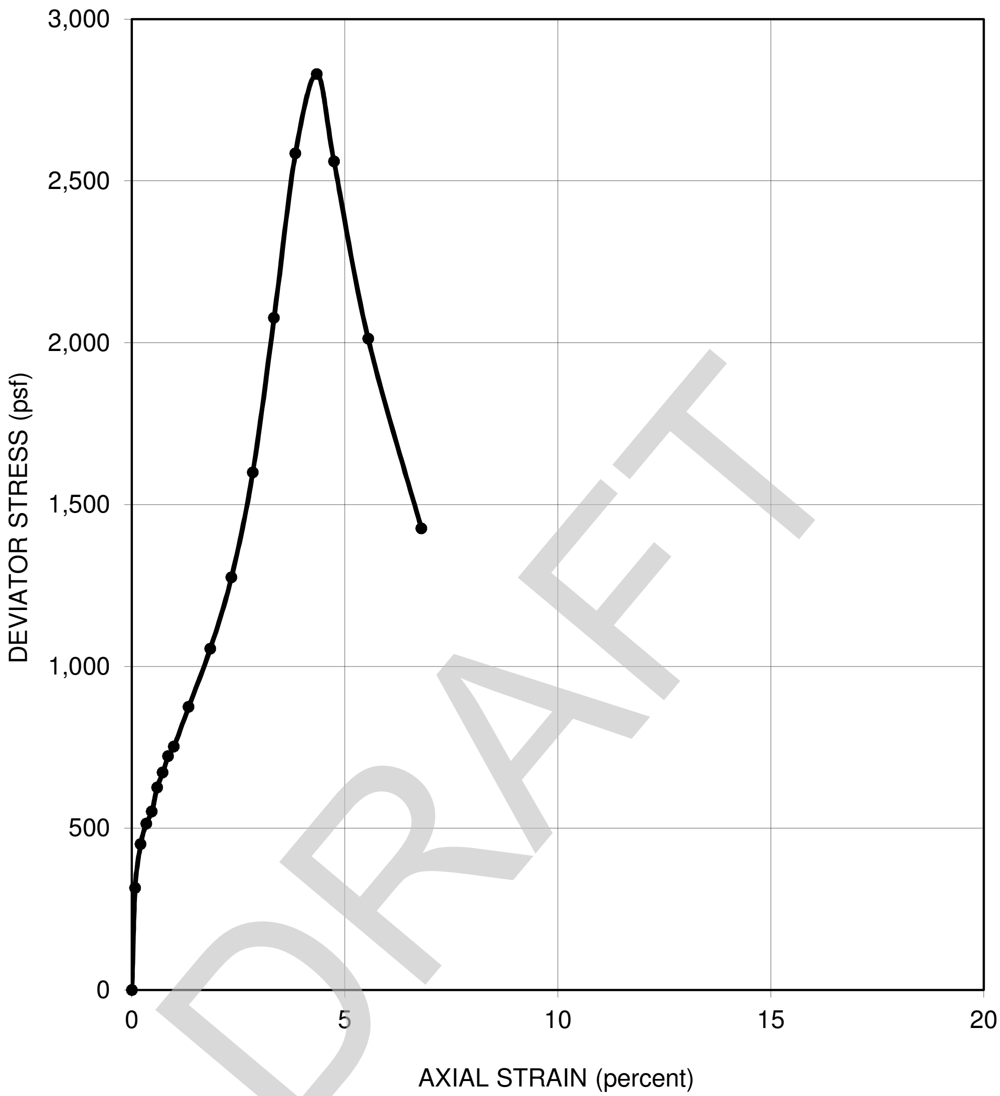
LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 12/21/15 | Project No. 770626301 | Figure C-5



SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	1,410	psf
DIAMETER (in.)	2.4	HEIGHT (in.) 5.7		STRAIN AT FAILURE	4.3	%
MOISTURE CONTENT		33.8	%	CONFINING PRESSURE	3,500	psf
DRY DENSITY		86	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), blue-gray				SOURCE	B-2 at 50 feet

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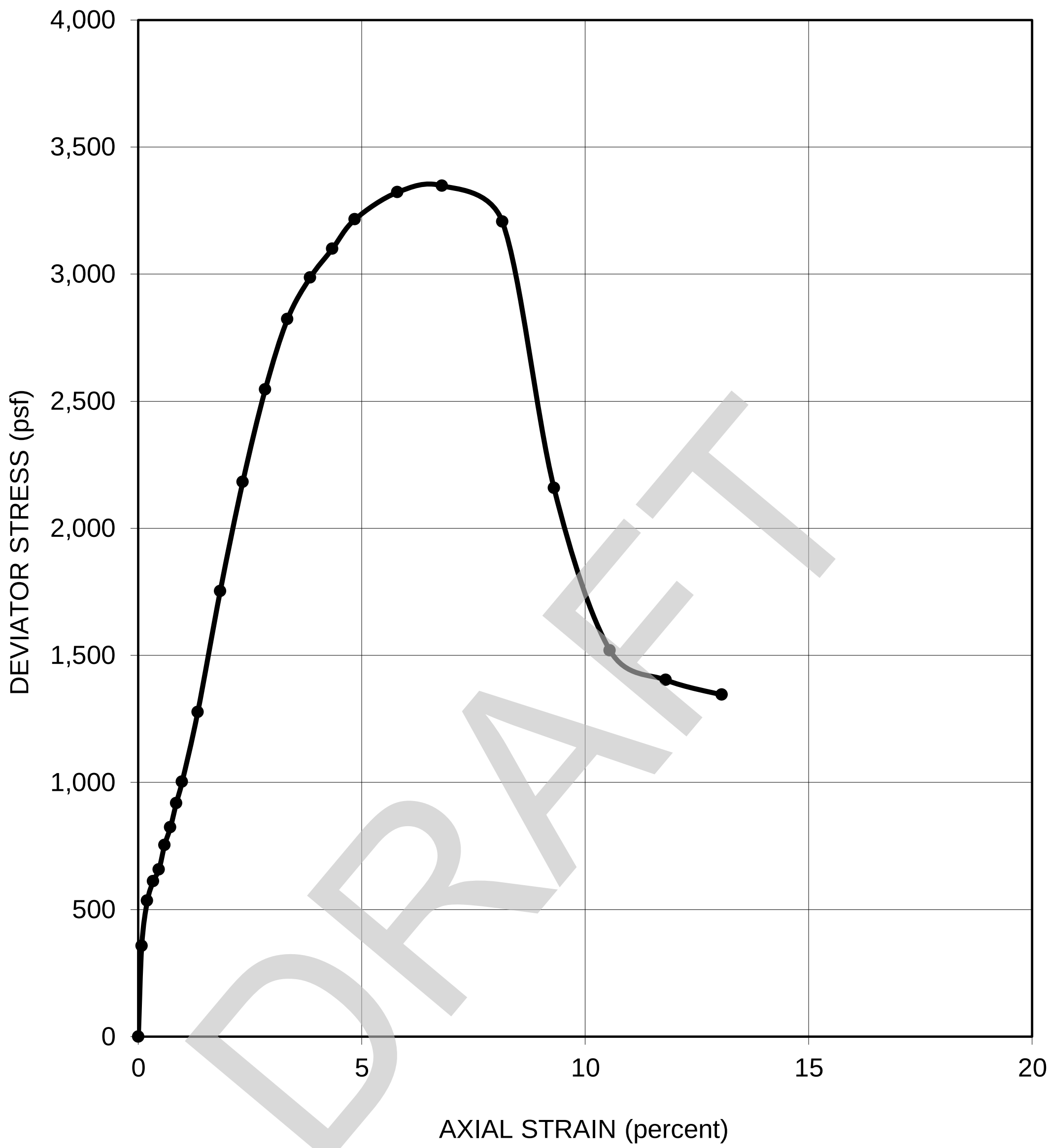
LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 12/21/15 | Project No. 770626301 | Figure C-6



SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	1,670	psf
DIAMETER (in.)	2.4	HEIGHT (in.) 5.6		STRAIN AT FAILURE	6.8	%
MOISTURE CONTENT		24.1	%	CONFINING PRESSURE	2,600	psf
DRY DENSITY		100	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY with SAND (CL), olive-gray				SOURCE	B-3 at 35 feet

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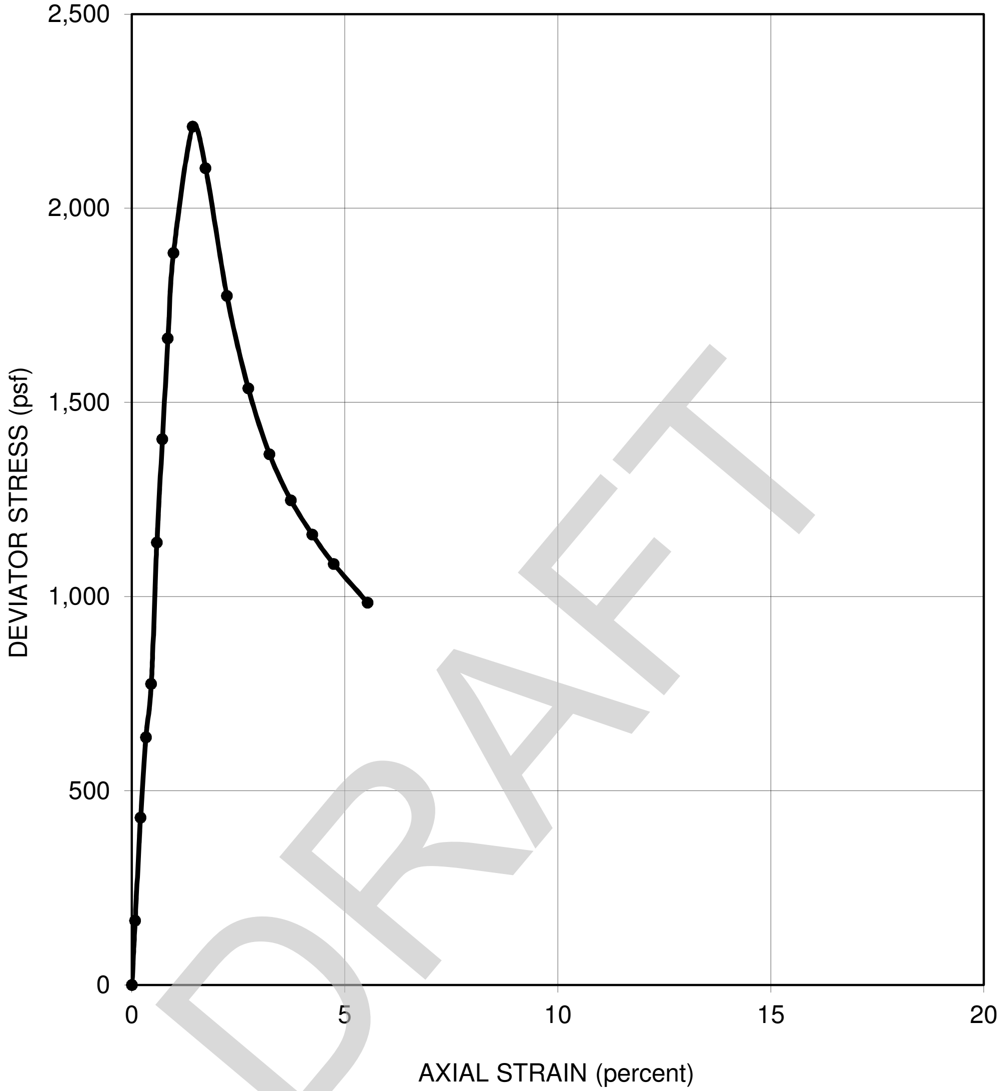
LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date	12/21/15	Project No.	770626301	Figure	C-7



SAMPLER TYPE	Shelby Tube		SHEAR STRENGTH	1,100	psf
DIAMETER (in.) 2.9	HEIGHT (in.) 6.1		STRAIN AT FAILURE	1.4	%
MOISTURE CONTENT	23.0	%	CONFINING PRESSURE	2,100	psf
DRY DENSITY	102	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), light brown			SOURCE	B-5 at 25 feet

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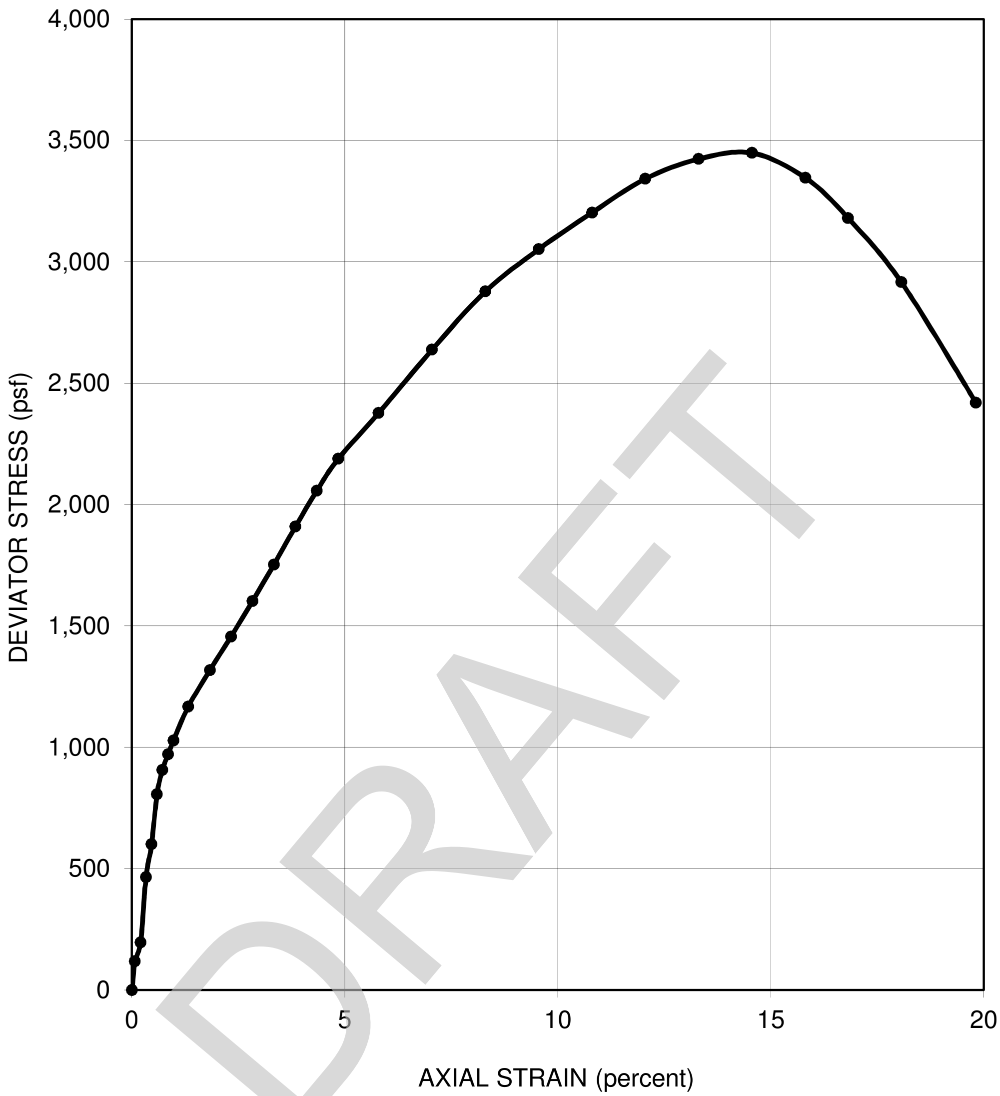
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 12/21/15 | Project No. 770626301 | Figure C-8



SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	1,720	psf
DIAMETER (in.)	2.9	HEIGHT (in.)	6.1	STRAIN AT FAILURE	14.6	%
MOISTURE CONTENT		12.7	%	CONFINING PRESSURE	1,300	psf
DRY DENSITY		120	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), olive			SOURCE	B-6 at 11.5 feet	

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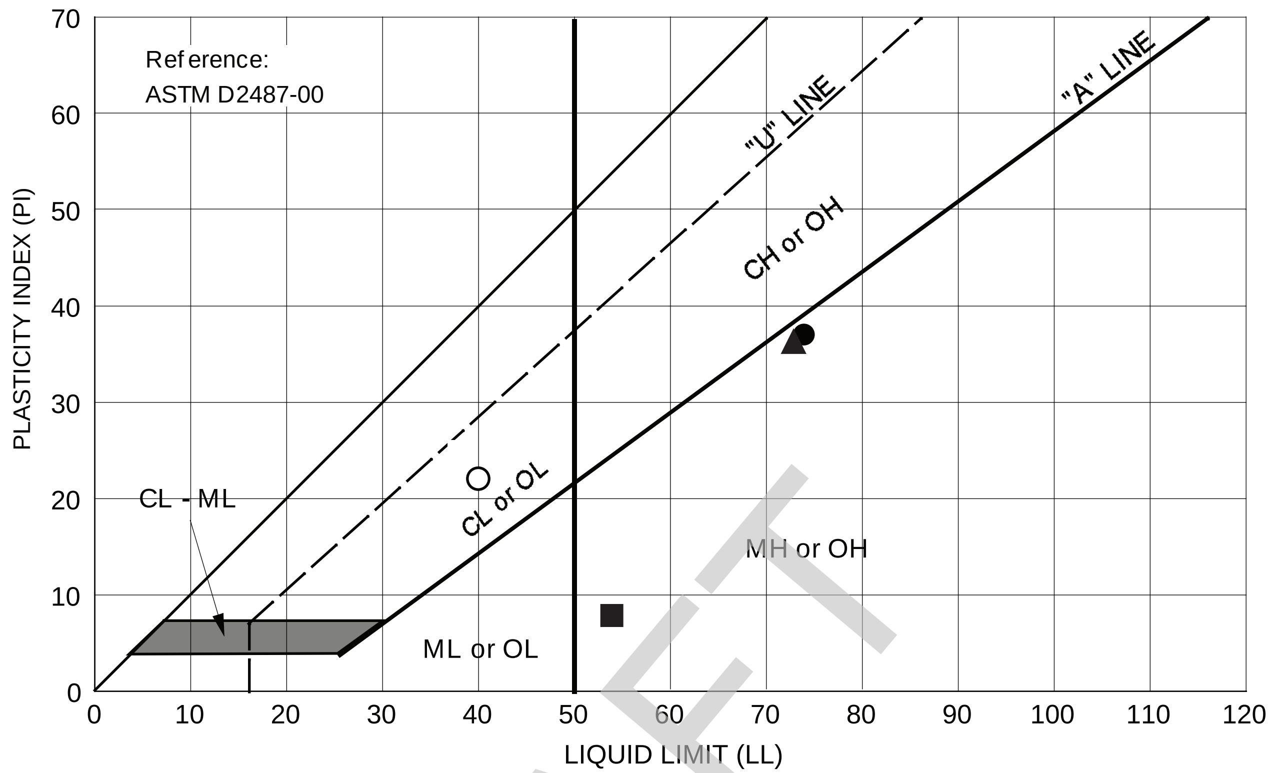
San Mateo, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 12/21/15 | Project No. 770626301 | Figure C-9



DRAFT

Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 6 feet	SILT (MH), olive-gray	--	74	37	--
■	B-3 at 8 feet	SILT (MH), gray-brown	--	54	8	--
▲	B-6 at 8 feet	SILT (MH), gray	--	73	36	--
○	B-6 at 1-2.5 feet	CLAY (CL), black	--	40	22	--

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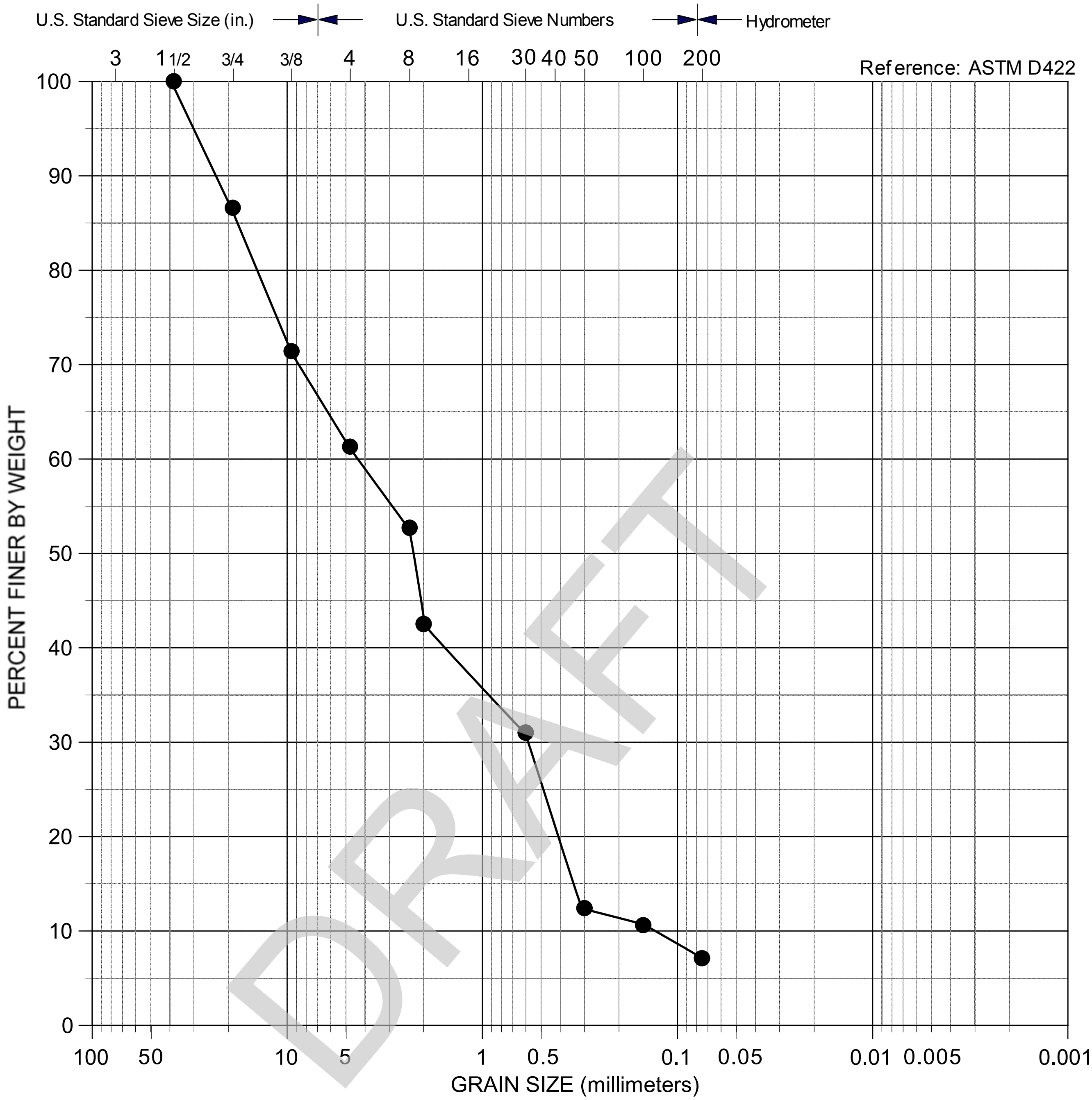
PLASTICITY CHART

LANGAN TREADWELL ROLLO

Date 01/06/16 | Project No. 770626301 | Figure C-10



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	% Gravel		%Sand			% Fines	
Sample	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Symbol	Sample Source	Classification
●	B-2 at 1 to 2 feet	SAND with SILT and GRAVEL (SP-SM), brown

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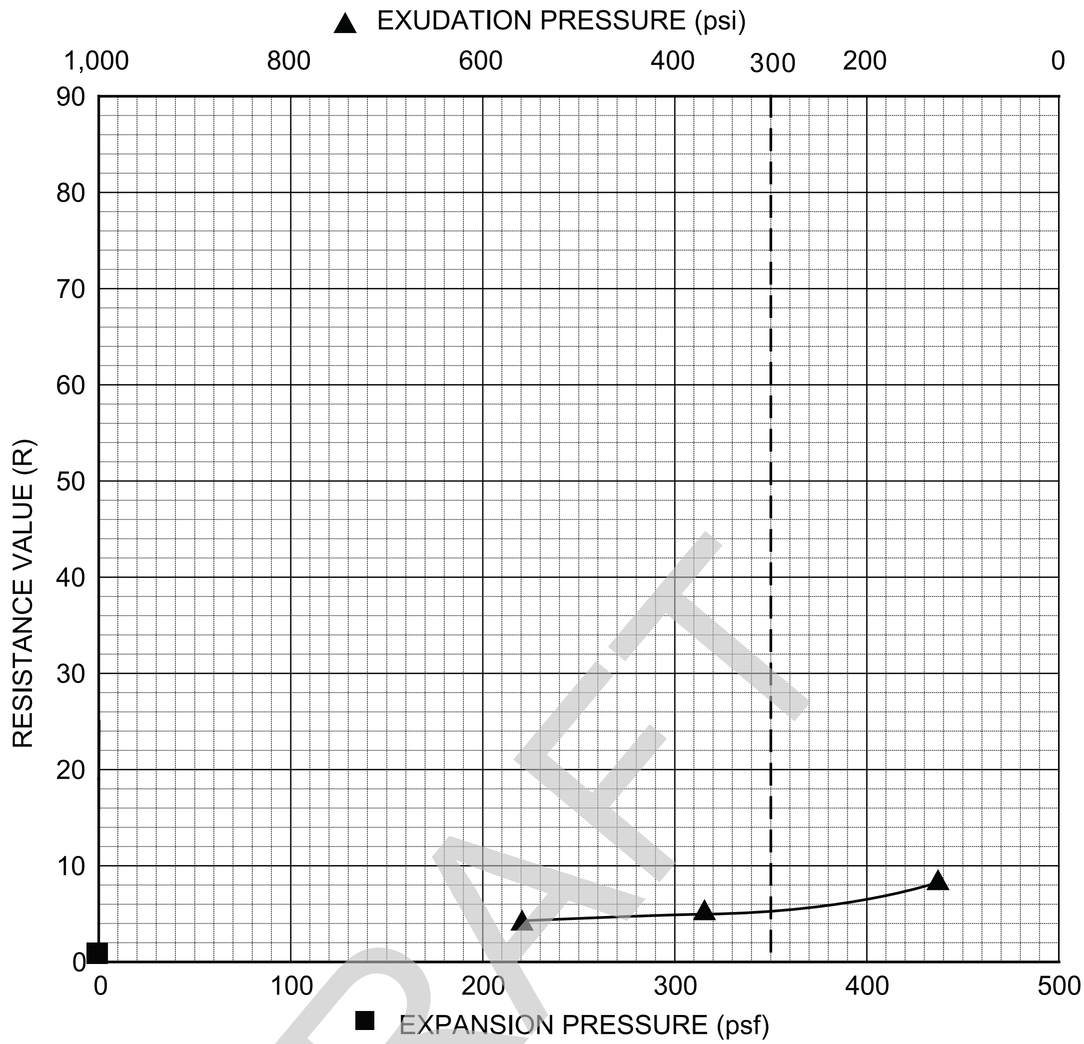
LANGAN TREADWELL ROLLO

PARTICLE SIZE ANALYSIS

Date 1/14/16 | Project No. 770626301 | Figure C-11



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Specimen ID:	A	B	C	D
Water Content (%)	17.3	16.4	15.5	--
Dry Density (pcf)	110.7	113.6	118.8	--
Exudation Pressure (psi)	240	332	467	--
Expansion Pressure (psf)	0	0	0	--
Resistance Value (R)	4	5	8	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-4 at 1 to 5 feet	SANDY CLAY with GRAVEL (CL), red-brown	--	--	5

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LANGAN TREADWELL ROLLO

RESISTANCE VALUE TEST DATA

Date 1/14/16	Project No. 770626301	Figure C-12

APPENDIX D

CORROSIVITY RESULTS

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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

30 December, 2015

Job No.1512142
Cust. No.12242

Mr. Matt Lattin
Langan Treadwell Rollo
4030 Moorpark Avenue, Suite 210
San Jose, CA 95117

Subject: Project No.: 770626301.700.315
Project Name: Concar Property
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Lattin:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on December 17, 2015. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are 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 were 300 & 500 mg/kg. Chloride ion concentrations greater than 300 mg/kg are considered corrosive to embedded reinforcing steel; and, as such, the concrete mix design shall be adjusted accordingly by a qualified corrosion engineer.

The sulfate ion concentrations were 63 & 240 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 were 7.79 & 7.81, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials for both samples was 330-mV, which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

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

JDH/jdl
Enclosure

DRAFT

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.: 770626301.700.315
Client's Project Name: Concar Property, San Mateo
Date Sampled: 12/10 & 11/15
Date Received: 17-Dec-15
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 30-Dec-2015

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)*
1512142-001	B-2-2, 3'	330	7.79	-	770	-	300	240
1512142-002	B-3-1, 1-5'	330	7.81	-	660	-	500	63

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	29-Dec-2015	29-Dec-2015	-	29-Dec-2015	-	29-Dec-2015	29-Dec-2015

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Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

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

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

1512142 12242

Chain of Custody

Page of

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

Job No.	CU#		Client Project I.D.
770626301	700	315 ~~324~~	Concar Property

Schedule

Analyte

ASTM w/Brief Evaluation

Date Sampled: 12/10/15

Date Due: Normal TAT

Full Name: Matt Lattin

email ~~Fax~~: mlattin@langan.com

Phone:

Company: Langan Treadwell Rollo

Cell: 916-844-8148 ☒

Sample Source: San Mateo

ANALYSIS

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation							
001	B-2-2, 3ft	12/11/15		S	PL			1	x	x	x	x	x		X							
002	B-3-1, 1-5ft	12/10/15		S	PL			1	X	X	X	X	X		X	Bag Broke open during Transport						
																GSO						

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

		Date	Time
Relinquished By:	[signature]	12/16/15	930am
Received By:	Kim McDonald	12/17/15	9:30
Relinquished By:			
Received By:			
Relinquished By:			
Received By:			

Comments:

THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES

8/6/2009

DRAFT

DISTRIBUTION

1 copy: Mr. Brian Myers
Coastal California Properties, LLC
520 Newport Center Drive, Suite 610
Newport Beach, California 92660

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QUALITY CONTROL REVIEWER

____

John Gouchon, GE #2282
Principal/Vice President