GEOTECHNICAL INVESTIGATION
BLOCKS 29-32
MISSION BAY
San Francisco, California

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San Francisco, California

21 December 2011
Project 750603902

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Mr. Ford Fish
salesforce.com
Landmark at One Market Street, Suite 300
San Francisco, California 94105

Subject: Geotechnical Investigation
Blocks 29-32, Mission Bay
San Francisco, California

Dear Mr. Fish:

Treadwell & Rollo is pleased to present this geotechnical investigation report for the proposed development at Blocks 29-32 project in Mission Bay in San Francisco, California. Copies have been distributed as indicated at the end of this report.

The proposed development will consist of four main structures, as summarized below:

- Block 29 Building (also known as Olive 29) will consist of a six- to nine-story structure, with a below-grade service area occupying the eastern portion of the structure.
- Block 30 Building (also known as Purple 30) will consist of a seven-story structure, with two basement levels for parking occupying the entire footprint of the structure.
- Block 31 Building (also known as Jacaranda 31) will consist of a six-story structure, with below-grade auditorium and service areas. The auditorium is located in the western portion of the structure, while the service area is in the eastern portion of the structure.
- Block 32 Building (also known as Yellow 32) will consist of a seven-story structure, with a below-grade service area occupying the western portion of the structure.

In addition to the four main buildings, a below-grade service tunnel is proposed to run in the north-south direction through the center of the site between Buildings 29 and 30 and between Buildings 31 and 32. The tunnel will connect the service areas of all four buildings. The remainder of the site will include several landscaping and infrastructure improvements, including water features, reflection pools, pavilions and other structural elements.

Subsurface conditions at the site consist of heterogeneous fill, underlain by Bay Mud, Colma Sand, Old Bay Clay, alluvium, and Franciscan Complex bedrock. The fill at the site is potentially liquefiable; erratic and unpredictable settlement may occur during a moderate to large earthquake. Furthermore, the Bay Mud is compressible and would consolidate under the weight of the main structures, causing excessive total and differential settlements. Therefore, we recommend the four main buildings be supported on pile foundations gaining support in dense sand or bedrock below the Bay Mud. Because the potential for liquefaction extends across the site and site vicinity, there is a potential for significant lateral movement of the ground surface during a major earthquake. Accordingly, we have recommended that the liquefaction potential be mitigated using ground modification techniques.

The existing fill and Bay Mud are not suitable for support of the buildings; pile foundations are recommended herein. At this time, the structural loads of the service tunnel, plaza and the landscape structures are unknown. The existing fill, which overlies the Bay Mud, was not compacted during placement, but with some overexcavation and recompaction should have sufficient strength to support light loads, provided some settlement is tolerable; therefore, for lightly loaded structures that can tolerate some settlement, spread footings or mat foundations may be utilized. For heavier loaded structures or structures with high lateral loads, low settlement tolerance or high uplift loads, pile foundations should be used.

This summary omits detailed recommendations; therefore, anyone relying on the report must read it in its entirety.

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

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

Sincerely yours,
TREADWELL & ROLLO, A LANGAN COMPANY

Serena T. Jang, GE
Senior Project Manager

Lori A. Simpson, GE
Vice President/Senior Associate

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed development at Blocks 29, 30, 31 and 32 (Blocks 29-32) in the Mission Bay area of San Francisco.

Blocks 29-32 is located within an area bound by Third Street on the west, South Street on the north, the future Terry Francois Boulevard on the east, and 16th Street on the south, as shown on Figure 1. The project area has approximate plan dimensions of 770 feet by 600 feet. The site is mostly vacant, with paved parking lots in the western and northern portions of the site. With the exception of an area in the southern portion of the site, the site is relatively flat, with the ground surface elevations ranging from 99.1 to 102.9 feet[1,2]. There is a depressed area in the southern portion of the site due to an excavation previously performed at the site for an environmental cleanup; the area has a plan dimension of approximately 320 feet by 280 feet and the ground surface elevation ranges from Elevation 91.4 feet to 96 feet.

According to schematic plans dated 30 August 2011 provided by Flad Architects a preliminary elevation plan provided by Tom Leader Studio dated 17 October 2011, structural sections by Rutherford & Chekene dated 28 November 2011 and email correspondences between the design team, the proposed development will consist of four main structures, as shown on Figure 2 and described below.

- *Block 29 Building* (also known as Olive 29) will consist of a six- to nine-story structure, with a below-grade service area occupying the eastern portion of the structure. The proposed finished ground floor elevation for the building is Elevation 103.42 feet. The finished floor for the below-grade service area will be at approximately Elevation 84.5 feet.
- *Block 30 Building* (also known as Purple 30) will consist of a seven-story structure, with two basement levels for parking occupying the entire footprint of the structure. The proposed finished floor elevation for the ground level is Elevation 101.63 feet. The finished floor for the lowest level of basement will be at approximately Elevation 74 feet.

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1 All elevations reference San Francisco City Datum plus 100 feet.
2 Based on "x-site-survey.dwg" by Sherwood Design, emailed to T&R on 10/19/11 from Tom Leader Studio.

- *Block 31 Building* (also known as Jacaranda 31) will consist of a six-story structure, with a below-grade auditorium and service area. The auditorium is located in the western portion of the structure, while the service area is in the eastern portion of the structure. The proposed finished ground floor elevation for the ranges from Elevation 103.43 to Elevation 103.68 feet. The finished floor for the auditorium is approximately 15 feet below finished floor elevation, at approximately 88.4 feet, and the finish floor of the service area will be at approximately Elevation 84.5 feet.
- *Block 32 Building* (also known as Yellow 32) will consist of a seven-story structure, with a below-grade service area occupying the western portion of the structure. The proposed finished floor elevation for the building is Elevation 101.63 feet. The finished floor for the below-grade service area will be at approximately Elevation 84.5 feet.

In addition to the four main buildings, a below-grade service tunnel is proposed to run in the north-south direction between Buildings 29 and 30 and between Buildings 31 and 32. The service tunnel will connect the service areas of all four buildings. The entry ramp for the service tunnel is at 16th Street. The proposed finished floor elevation of the tunnel is at approximately Elevation 77 feet.

The remainder of the site will include landscaping and infrastructure improvements, including water features, reflection pools, pavilions and other structural elements.

2.0 SCOPE OF SERVICES

The scope of our services was outlined in our proposal dated 9 June 2011. We reviewed existing subsurface data from the site and the vicinity and further explored subsurface conditions at the site. In July and October 2007, Treadwell & Rollo prepared geotechnical investigation reports for Blocks 30 and 32 for the previous site owner. When the current owner purchased the site from the previous owner, they acquired the data from the 2007 investigations, which included ten borings and two Cone Penetration Tests (CPTs). In addition, we drilled three borings for the current owner to obtain additional subsurface information for a preliminary evaluation of the subsurface conditions at the site. We also maintain a database of historical borings and CPTs in Mission Bay by others. For this investigation, we supplemented the previously obtained data by drilling eleven test borings, advancing three CPTs and performing laboratory tests on samples recovered from the borings. We relied on the data from these current and previous investigations by us and others to develop the conclusions and recommendations presented in this report.

Engineering studies were performed based on the soil and groundwater conditions defined by the borings and CPTs and engineering parameters developed from the laboratory testing program. On the basis of field and laboratory tests, our engineering studies and the results of recent experience on similar sites in Mission Bay, we developed conclusions and recommendations regarding:

- soil, bedrock, and groundwater conditions at the site
- the most appropriate foundation types for the proposed structure
- design criteria for the most appropriate foundation types, including values for vertical and lateral pile capacities
- estimated foundation and surrounding ground surface settlements
- floor slabs
- fill quality and compaction criteria
- soil subgrade preparation
- corrosion potential and mitigation of corrosivity
- seismic hazards, including ground rupture, liquefaction, and differential compaction
- seismic design criteria in accordance with 2010 San Francisco Building Code (SFBC)
- recommended site-specific response spectra
- site improvement
- construction considerations.

3.0 FIELD INVESTIGATION

We began our investigation by reviewing the results of previous studies by T&R, as well as by others, at and in the vicinity of the site. The approximate locations of these points of exploration are presented on Figure 2. Nearby borings and CPTs, including Borings B31-1, C29-1, C29-2, and C31-1, are shown on the site plan but the boring logs are not included in this report, as they were developed for other clients.

The approximate locations of the eleven additional borings and three CPTs done for the current phase of work are shown on Figure 2.

Prior to performing the field investigation, we:

- obtained a soil boring permit from the Monitoring Wells Section of the San Francisco Department of Public Health (SFDPH)
- notified Underground Service Alert
- checked the boring and CPT locations for underground utilities using an independent utility locating contractor.

3.1 Test Borings

During the previous and current investigation of the site, twenty-four test borings were drilled in four different mobilizations:

1. From 25 through 30 April 2007, five test borings, designated B32-1 through B32-5, were drilled using a truck-mounted, rotary-wash drill rig operated by Pitcher Drilling Company. The test borings were drilled to bedrock to depths of approximately 44.5 to 99 feet below the existing ground surface (bgs).
2. From 3 through 6 May 2007, five test borings, designated B30-1 through B30-5, were drilled using a truck-mounted, rotary-wash drill rig operated by Pitcher Drilling Company. The test borings were drilled to bedrock to depths of approximately 79.5 to 129 feet bgs.
3. From 7 through 11 June 2011, three test borings, designated as B29-1, B29-2, and B31-2 were drilled using a truck-mounted, rotary-wash drill rig operated by Pitcher Drilling Company. The test borings were drilled to bedrock to depths of approximately 87 to 136 feet bgs.
4. From 22 August 2011 through 11 September 2011, 11 test borings, designated B29-3 through B29-8 and B31-3 through B31-7, were drilled using two truck-mounted, rotary-wash drill rigs operated by Pitcher Drilling Company. The test borings were drilled to bedrock to depths of approximately 45.5 to 170.5 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 on Figures A-1 through A-24 in Appendix A. The soil and rock encountered in the borings were classified in accordance with the Classification Chart and Physical Properties Criteria for Rock Descriptions, presented on Figures A-25 and A-26, respectively.

Soil and rock samples were obtained using four different types of samplers: two driven split-barrel samplers and two pushed thin-walled samplers. 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 sampler with a 3.0-inch outside diameter and 2.875-inch inside diameter
- Dames & Moore (D&M) sampler with a 2.5-inch outside diameter thin walled tube.

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

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or less if the blow count approached 50 blows. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, for Borings B30-1 through B30-5 and B32-1 through B32-5, and factors of 0.7 and 1.2, respectively, for Borings B29-1 through B29-8 and B31-2 through B31-7, to account for sampler type and hammer energy. The N-values are shown on the boring logs. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

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

Upon completion, the boreholes were backfilled with grout consisting of cement, bentonite, and water in accordance with the requirements of the SFDPH. The grouting was completed under the observation of a SFDPH Inspector. The soil cuttings from the borings were collected in 55-gallon drums and bins, which were stored temporarily at the site, tested, and eventually transported off-site for proper disposal.

3.2 Cone Penetration Tests

Five CPTs, designated C29-1,C29-2, C30-1, C31-2 and C32-1, were advanced at the site. The CPTs were performed by two CPT contractors:

- On 3 and 4 May 2007, two CPTs, designated C30-1 and C32-1, were advanced by John Sarmiento and Associates with a truck-mounted, 20-ton push CPT rig. The CPTs were advance through the existing fill and into the underlying Bay Mud to a depth of approximately 35 feet bgs.
- On 25 August 2011, three CPTs, designated C29-1, C29-2 and C31-2 were advanced by Gregg Drilling and Testing with a truck-mounted, 20-ton push capacity CPT rig. The CPTs were advanced through the Bay Mud to depths of 52.7 to 69.9 feet.

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 and friction ratio by depth, as well as interpreted SPT N-Values, and interpreted soil classification, are presented in Appendix B as Figures B-1 through B-5. For C30-1 and C32-1, soil types were estimated using the classification chart on Figure B-6. For C29-1, C29-2 and C312, soil types were estimated using the classification chart on Figure B-7.

Upon completion of the field investigation, the CPT holes were backfilled with cement-bentonite grout in accordance with the requirements of SFDPH under the intermittent observation of an inspector from SFDPH.

3.3 Subsurface Information Database

T&R has performed several geotechnical investigations for surrounding projects. The approximate locations of the borings and Cone Penetration Tests (CPTs) closest to the Blocks 29-32 project site from the previous investigations are shown on Figure 2. The data is maintained in our files. In addition to exploration performed by T&R at this site and in the surrounding area, we have developed and maintain in our files a database of boring logs from various sources for the Mission Bay area. Their approximate locations of these borings are shown on Figure 2.

4.0 LABORATORY TESTING

The soil and bedrock samples recovered from the recent field exploration program were re-examined in the office for soil and rock classifications, and representative samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, fines content, plasticity (Atterberg limits), shear strength, and compressibility (consolidation). Results of the laboratory tests are included on the boring logs and in Appendix C.

Because corrosive soil can adversely affect underground utilities and below-grade elements, corrosion testing was performed as part of the detailed corrosion study of the upper soils. The results of the corrosivity evaluation and recommendations for corrosion protection are presented in Appendix D.

5.0 SITE CONDITIONS

Our understanding of the site conditions is based on a review of published literature, our site subsurface exploration, and our earlier research of the entire Mission Bay development area.

5.1 Site Conditions

Originally, the site was below water in a shallow bay known as Mission Bay. The tip of historic Point San Quentin was located just south of the site, along the 1859 San Francisco shoreline. Starting in the late 1860s, the bay was reclaimed by placing fill. A review of historic maps (Rumsey, 2003) and documents (ESA, 1990) indicates that the site was reclaimed starting around 1869 with soil and rock from nearby Irish Hill and the Second Street cut, and the filling completed between 1906 and 1910s with fill and building rubble from the 1906 San Francisco earthquake.

Starting in the early 1900s, the northwest portion of the site was occupied by the structures supporting Santa Fe Railroad, including store houses and coach and machine shops, and was also used as a stock corral. This area was relatively vacant from the 1940s through the 1970s. From 1902 to 1962, the Associated Oil Company occupied the southeast portion of the site with oil-related facilities including crude oil storage and several large above-ground storage tanks, offices and railroad tracks. In addition, the area was used for lumber storage and trucking-related activities from 1910s through the mid-1970s. The Bode concrete plant occupied the western portion of the site from the 1970s until 2002 and was demolished in the 2003. In addition, the northern end of Illinois Street extended into the project site, as did as a segment of El Dorado Street extending from the west.

Based on environmental studies, oil-related site activities caused contamination of the fill and groundwater at the southern portion of the site. In 2005, the southern portion of the site was remediated by the Pier 64 Group and ENTACT. Site remediation included demolition and removal of structures and pavements, removal of abandoned underground utilities, and excavation and off-site disposal of selected soil. From 2005 through 2008, portions of the remediated areas of site were backfilled in various stages up to the existing ground surface elevations. One area remains low as backfilling was not completed.

Currently, the site is relatively flat, with the ground surface elevations ranging 99.1 to 102.9 feet. Paved parking lots occupy the western and northern portion of the site, with the remainder of the site unpaved. In an area in the southern portion of the site, the ground surface is low due to the previous remediation excavation and is at approximately Elevation 91.4 feet to 96 feet. Ponding groundwater has been found near the bottom of the low areas. Other than this ponded water, no springs or seepages were observed on site.

5.2 Subsurface Conditions

Two idealized subsurface profiles (Figures 3 and 4) illustrate the general subsurface conditions at the site, consisting of fill, Bay Mud, Colma Formation sand, clay and sand layers and bedrock. The locations of the profiles are shown on Figure 2.

Fill: Where explored, the site is blanketed by approximately 9 to 33.5 feet of fill. The thickness of fill varies significantly throughout the site. The fill consists of gravel, sand, and clay mixtures, with brick, rock (including serpentinite), and other rubble. The sands and gravel are loose to very dense, and the clay is soft to stiff. The fill likely also includes cobble- and boulder-sized pieces of serpentinite that were apparent from the drilling but could not be recovered within the 1.5- to 2.5-inch diameter samplers. Where tested in the upper 10 feet, the fill has low expansion potential[3]. Corrosivity analyses indicate the fill is classified as generally corrosive; see Appendix D for more detail.

Bay Mud: A weak and compressible marine clay deposit, referred to as Bay Mud, is present beneath the fill. Where explored within the project site, this layer ranges from 2.5 to 46.5 feet thick, generally becoming thicker to the north.

Laboratory test results from this and nearby investigations indicate it has a compression ratio of 0.20 to 0.35. CPTs and lab test indicate the Bay Mud is normally consolidated to overconsolidated[4]. Where tested, the undrained shear strength is 275 to 1,500 pounds per square foot (psf). In general, the undrained shear strength of the Bay Mud increases with depth.

Corrosivity analyses indicate the Bay Mud is severely corrosive; see Appendix D for more detail.

Sand and Clay: A medium dense to very dense clayey sand, silty sand and sand with clay and stiff to hard sandy clay, clay with sand and clay was encountered below the Bay Mud. Where encountered the sand and clay layer is 3 to 31 feet thick. Where tested, the undrained shear strength of the clay varies from 1,540 to 7,480 psf.

Colma: Formation A medium dense to very dense sand, sand with clay, clayey sand, silty sand and sand with silt, known as the Colma Formation, was encountered below the sand and clay, except in borings B30-5, B31-6, B31-7, B32-2, B32-3, B32-4, B32-5, 127 and 361. Where encountered and tested, the sand is approximately 5 to 35 feet thick with percent fines ranging from 5.6 to 29.2. In Boring B29-6, a one-foot-thick layer

3 Highly expansive soil undergoes large volume changes with changes in moisture content.

4 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.

of hard clay with sand was encountered embedded in the Colma Formation sand. The Colma Formation generally becomes thicker to the north and west. Figure 5 presents a summary of the very dense sand (SPT-N value is greater than 50 blows per foot) portion of the Colma Formation, indicating the thickness of the very dense sand layer and the contours of the top of the very dense sand elevation.

Clay and Gravel: In Borings B29-1 through B29-7 and B30-1 a stiff to hard clay known as Old Bay Clay was encountered below the Colma Formation. The layer is approximately 6 to 21 feet thick. The Old Bay Clay is overconsolidated.

Borings B30-3 through B30-5 and B31-3 through B31-7 encountered very stiff to hard sandy clay, clay, gravelly clay with sand and clay with gravel and dense to very dense sand with silt and clayey sand below the Colma Formation.

Bedrock: Bedrock consists of serpentinite, greenstone, shale, and claystone of the Franciscan Complex. The rock is crushed to intensely fractured, soft to moderate hardness, and friable to weak, with deep to moderate weathering. Bedrock was encountered at depths ranging from 33 feet (Elevation 63 feet) in Boring B32-4 to 130 feet (Elevation -29 feet) in Boring B29-1. Bedrock generally becomes deeper to the northwest. Approximate bedrock elevation contours are presented on Figure 6.

Groundwater: Groundwater was encountered during our field investigation and the level was measured in several of the boreholes prior to switching from auger drilling to rotary wash method. The groundwater level was encountered at about 6.6 feet bgs in Boring B29-7 (Elevation 93.4 feet) to about 7 feet bgs in Boring B32-4 (Elevation 89 feet). The groundwater level is influenced by rainfall and tides; therefore, the groundwater level measurements may not represent stabilized groundwater levels.

6.0 GEOLOGY AND SEISMICITY

Our evaluation of the geology and seismicity of the area is based on our review of published reports and information in our files from other sites in the vicinity.

6.1 Regional Geology

The site is in the northeast portion of the San Francisco peninsula, which lies within the Coast Ranges geomorphic province. The northwesterly trend of ridges and valleys characteristic of the Coast Ranges is obscured in San Francisco, except for features such as Russian Hill, Telegraph Hill, Hunters Point, and Potrero Hill. San Francisco Bay and the northern portion of the peninsula lie within a down-dropped crustal block bounded by the East Bay Hills and the Santa Cruz Mountains. The San Francisco Bay depression resulted from interaction between the major faults of the San Andreas fault zone, particularly the Hayward and San Andreas faults east and west of the bay, respectively (Atwater, 1979).

San Francisco's topography is characterized by relatively rugged hills formed by Jurassic- to Cretaceous-aged[5] bedrock (Schlocker, 1974). The bedrock consists of highly deformed and fractured sedimentary rocks of the Franciscan complex. The present topography resulted mainly from east-west compression of coastal California during the late Pliocene and Pleistocene[6] epochs (Norris and Webb, 1990).

The low-lying areas of the San Francisco peninsula are underlain by Quaternary[7] sediments deposited on eroded Franciscan bedrock. Sediment deposition within the pre-historic[8] bay margin was influenced by oscillating late-Quaternary sea levels that resulted from the advance and retreat of glaciers worldwide.

The resulting sequence of alternating estuarine[9] and terrestrial[10] sediments corresponds to high and low sea-level stands, respectively. In contrast, Quaternary sediments in the plains landward of the bay are predominantly terrestrial.

By late Pleistocene time, the high sea level associated with the Sangamon (about 125,000 years ago) interglacial resulted in deposition of the Yerba Buena Mud (Sloan, 1992). Also known locally as "Old Bay Clay", the Yerba Buena Mud was deposited in an estuarine environment similar in character and extent to

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5 The *Jurassic* and *Cretaceous* periods spanned the time period from approximately 160 to 70 million years ago.

6 The *Pliocene* epoch spans from approximately 5 to 2 million years ago, while the *Pleistocene* epoch spans from approximately 2 million to 11,000 years ago.

7 The *Quaternary* period spans from approximately 2 million years ago to present, and includes the Pleistocene and Holocene epochs.

8 The present margin of San Francisco Bay is generally located seaward of its original location as a result of extensive land reclamation over the last century.

9 *Estuarine* sediments typically consist of silt and clay, sometimes rich in organic matter and with occasional sand, deposited in inland marine areas affected by fresh water. Represents present environment of San Francisco Bay and includes the bay and adjacent tidal marshlands.

10 *Terrestrial* sediments generally consist of variable mixtures of clay, silt, sand and gravel deposited by rivers and streams ("alluvial deposits" or "alluvium"), and fine sand deposits deposited by wind ("eolian deposits" such as dune sands).

the present bay. Sea level lowering associated with the onset of Wisconsin glaciation exposed the bay floor and resulted in terrestrial sedimentation, such as the Colma formation, on the Yerba Buena Mud. Sea level rose again starting roughly 20,000 years ago, fed by the melting of Wisconsin-age glaciers.

The sea re-entered the Golden Gate about 10,000 years ago (Atwater, 1979). Inundation of the present bay resulted in deposition of estuarine sediments, called Bay Mud, which continue to accumulate.

Historical development of the San Francisco Bay area resulted in placement of artificial fill material over substantial portions of modern estuaries, marshlands, tributaries, and creek beds in an effort to reclaim land (Nichols and Wright, 1971).

Potrero Hill immediately southwest of the site is comprised of serpentinite. The serpentinite bedrock is associated with ancient shear zones within and bounding portions of the Franciscan Complex bedrock units. The shear zones generally consist of a mixture of hard blocks of bedrock, from less than an inch to 25 feet or more in diameter, contained within a matrix of soft, intensely sheared shale. Serpentinite is the most common rock type, however, hydrothermally altered rocks such as calc-silicate compositions are common locally.

The bedrock underlying Mission Bay predominantly consists of sandstone, serpentinite, greenstone, chert, and shale. It is covered by colluvium and marine deposits. Fill of highly variable quality and density blankets the site.

6.2 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 7. For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[11] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

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

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
N. San Andreas – Peninsula	12.7	West	7.23
N. San Andreas (1906 event)	12.7	West	8.05
Total Hayward	16	East	7.00
Total Hayward-Rodgers Creek	16	East	7.33
N. San Andreas – North Coast	17	West	7.51
San Gregorio Connected	19	West	7.50
Mount Diablo Thrust	33	East	6.70
Total Calaveras	34	East	7.03
Rodgers Creek	36	North	7.07
Green Valley Connected	38	East	6.80
Monte Vista-Shannon	39	Southeast	6.50
Point Reyes	44	West	6.90
West Napa	46	Northeast	6.70
Greenville Connected	50	East	7.00
Great Valley 5, Pittsburg Kirby Hills	55	East	6.70
Great Valley 4b, Gordon Valley	70	Northeast	6.80
N. San Andreas – Santa Cruz	74	Southeast	7.12
Great Valley 7	76	East	6.90
Hunting Creek-Berryessa	78	North	7.10
Zayante-Vergeles	84	Southeast	7.00
Great Valley 4a, Trout Creek	92	Northeast	6.60
Maacama-Garberville	94	North	7.40
Monterey Bay-Tularcitos	97	South	7.30

Figure 7 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 8) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 93 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, 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 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

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

7.0 GEOLOGIC HAZARDS

During a major earthquake, strong and intense ground shaking is expected to occur at the project site (USGS, 2008).

These levels of ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[12], lateral spreading[13], and seismic densification[14]. Each of these conditions has been evaluated based on our literature review, field investigation, and analysis and is discussed in this section.

The project site is relatively flat, except in the one low area of the site. Once developed, the project site will be relatively level and should not be subject to landslide or erosion.

7.1 Ground Shaking and Probabilistic Seismic Hazard Analysis

We expect this site will experience strong and intense ground shaking during a major earthquake on any of the nearby faults. We developed site-specific response spectra corresponding to the Maximum Considered Earthquake (MCE) and Design Earthquake (DE) per the 2010 San Francisco Building Code (SFBC) and ASCE 7-05. The MCE spectrum is defined as the lesser of the probabilistic spectrum having 2 percent probability of exceedance in 50 years or 150 percent of median deterministic event on the governing fault. The DE is defined in the 2010 SFBC as 2/3 of the Maximum Considered Earthquake (MCE) spectrum.

Because of the subsurface conditions at different blocks, we performed probabilistic seismic hazard analysis (PSHA), deterministic analysis and ground response analysis to develop site-specific spectra. The probabilistic seismic hazard analysis (PSHA) was performed using the computer code EZFRISK 7.62 (Risk Engineering 2011). This approach is based on the probabilistic seismic hazard model developed by Cornell (1973) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources and earthquake activities were assigned to the faults based on historical and geologic data. The site-specific effects of the overburden soil for Blocks 29, 30 and 31 were evaluated using the ground response program SHAKE91 (Idriss and Sun 1992) as part of a computational module in EZFRISK; we assumed for our ground response analyses that the liquefaction potential in the upper 15 feet of fill will be mitigated.

Details of our analyses are discussed in Appendix E. The recommended horizontal ground surface spectra for Blocks 29, 30, 31 and 32 are shown on Figures 9, 10, 11 and 12, respectively. Digitized values of the recommended horizontal MCE and DE spectra for these blocks for a damping ratio of 5 percent are presented in Tables 3 through 6.

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

13 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.

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

TABLE 3
Recommended Spectra for Block 29
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.419	0.357
0.10	0.542	0.434
0.20	0.735	0.618
0.30	0.924	0.791
0.40	1.075	0.908
0.50	1.183	0.973
0.60	1.270	1.010
0.70	1.325	1.040
0.80	1.350	1.070
0.90	1.360	1.085
1.00	1.365	1.070
1.10	1.365	1.020
1.20	1.300	0.929
1.30	1.200	0.778
1.40	1.065	0.651
1.50	0.925	0.563
1.60	0.809	0.506
1.70	0.720	0.468
1.80	0.664	0.442
1.90	0.628	0.418
2.00	0.596	0.398
2.10	0.568	0.379
2.20	0.543	0.362
2.30	0.519	0.346
2.40	0.497	0.332
2.50	0.478	0.318
2.60	0.459	0.306
2.70	0.442	0.295
2.80	0.426	0.284
2.90	0.412	0.274
3.00	0.398	0.265
3.10	0.385	0.257
3.20	0.373	0.249
3.30	0.362	0.241
3.40	0.351	0.234
3.50	0.341	0.227
3.60	0.332	0.221
3.70	0.323	0.215
3.80	0.314	0.209
3.90	0.306	0.204
4.00	0.298	0.199

TABLE 4
Recommended Spectra for Block 30
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.420	0.264
0.10	0.725	0.623
0.20	0.991	0.869
0.30	1.100	0.950
0.40	1.100	0.950
0.50	1.100	0.950
0.60	1.100	0.950
0.70	1.100	0.950
0.80	1.100	0.950
0.90	1.100	0.950
1.00	1.100	0.896
1.10	1.100	0.810
1.20	1.060	0.727
1.30	0.980	0.650
1.40	0.901	0.583
1.50	0.824	0.535
1.60	0.754	0.498
1.70	0.702	0.468
1.80	0.664	0.442
1.90	0.628	0.418
2.00	0.596	0.398
2.10	0.568	0.379
2.20	0.543	0.362
2.30	0.519	0.346
2.40	0.497	0.332
2.50	0.478	0.318
2.60	0.459	0.306
2.70	0.442	0.295
2.80	0.426	0.284
2.90	0.412	0.274
3.00	0.398	0.265
3.10	0.385	0.257
3.20	0.373	0.249
3.30	0.362	0.241
3.40	0.351	0.234
3.50	0.341	0.227
3.60	0.332	0.221
3.70	0.323	0.215
3.80	0.314	0.209
3.90	0.306	0.204
4.00	0.298	0.199

TABLE 5
Recommended Spectra for Block 31
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.554	0.446
0.10	0.775	0.582
0.20	0.982	0.763
0.30	1.154	0.929
0.40	1.280	1.048
0.50	1.356	1.111
0.60	1.387	1.124
0.70	1.379	1.099
0.80	1.341	1.049
0.90	1.282	0.989
1.00	1.211	0.927
1.10	1.130	0.868
1.20	1.059	0.811
1.30	0.985	0.737
1.40	0.912	0.664
1.50	0.843	0.597
1.60	0.780	0.537
1.70	0.724	0.487
1.80	0.674	0.442
1.90	0.628	0.418
2.00	0.596	0.398
2.10	0.568	0.379
2.20	0.543	0.362
2.30	0.519	0.346
2.40	0.497	0.332
2.50	0.478	0.318
2.60	0.459	0.306
2.70	0.442	0.295
2.80	0.426	0.284
2.90	0.412	0.274
3.00	0.398	0.265
3.10	0.385	0.257
3.20	0.373	0.249
3.30	0.362	0.241
3.40	0.351	0.234
3.50	0.341	0.227
3.60	0.332	0.221
3.70	0.323	0.215
3.80	0.314	0.209
3.90	0.306	0.204
4.00	0.298	0.199

TABLE 6
Recommended Spectra for Block 32
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.758	0.505
0.05	0.940	0.627
0.10	1.319	0.880
0.20	1.500	1.000
0.30	1.500	1.000
0.40	1.500	1.000
0.50	1.500	1.000
0.60	1.384	0.923
0.75	1.200	0.800
1.00	0.900	0.600
1.50	0.600	0.400
2.00	0.450	0.300
3.00	0.300	0.200
4.00	0.225	0.150

7.2 Liquefaction and Associated Hazards

When a saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of strength as a result of a transient rise in 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 liquefaction hazard zone as designated by the California Geological Survey (CGS) seismic hazard zone map for the area titled *State of California Seismic Hazard Zones, City and County of San Francisco, Official Map,* dated 17 November 2001. However, there was no documented observation of liquefaction at this site during the 1906 Earthquake or the 1989 Loma Prieta Earthquake.

CGS has recommended the content for site investigation reports within seismic hazard zones be performed in accordance with Special Publication 117A titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California,* dated September 11, 2008. Our evaluation of site seismic hazards was performed in general accordance with these guidelines.

Borings B29-1, B29-2, B29-4, B29-5, B29-6, B29-7, B29-8, B30-1, B30-2, B30-3, B30-4, B30-5, B31-3, B31-4, B31-5, B31-6, B31-7, B32-2 and B32-3, C29-3, C29-4, C31-2 and C32-1 encountered loose to medium dense sand and gravel layers with varying silt and clay content just above or below the water table. The combined layers ranged from about 1½ to 16 feet thick. The results of our studies indicate these sand and gravel layers could liquefy during a major earthquake. Using the Tokimatsu and Seed (1987) method for evaluating earthquake-induced liquefaction settlement, we estimate settlement ranging of 1 to 6 inches may occur, depending upon the layer thickness and relative density. This settlement is expected to be erratic and vary significantly across the site.

Because of the shallow groundwater table and the relatively shallow liquefiable deposits, we conclude ground failure, such as lurch cracking and/or the development of sand boils, could occur. The ground-surface settlement will likely be larger than estimated in areas where sand boils and associated ground failure occur.

7.3 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 bay, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes.

According to Youd, Hansen, and Bartlett (1999), for significant lateral spreading displacements to occur, the soils should consist of saturated cohesionless sandy sediments with $(N_1)_{60}$ less than 15, where liquefaction of the soils is likely to occur based on standard liquefaction analysis. The potentially liquefiable fills underlying the site were determined to consist of silty sands and gravels with cobbles and boulders of serpentinite. Where serpentinite cobbles and boulders are in the fill, the fill should not undergo sustained loss of shear strength resulting from pore pressure increases, and hence, should not develop widespread shear zones for significant lateral displacements to occur during liquefaction. However, if the site is not mitigated against liquefaction, there are sufficient continuous zones of liquefiable material where sand is present to induce lateral spreading, causing significant damage to foundations. Therefore we are recommending the site be improved to mitigate liquefaction and lateral spread potential. If the site is improved, we anticipate liquefaction in the upper 15 feet of the fill will be mitigated, and the remaining liquefiable fill will generally have discontinuous layers with $(N_1)_{60}$ less than 15. Therefore, we do not anticipate large scale lateral spreading at the site should the top 15 feet be improved; however, localized areas of lateral spreading may occur.

7.4 Seismic Densification

Seismic densification (also referred to as cyclic densification and differential compaction) can also occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. In general, granular deposits encountered above the groundwater table were dense or clayey. Up to 4.5 feet of loose to medium dense sand was encountered in Borings B30-1, B30-2, B30-4, B30-5, B31-3, B31-5, B32-2 and B32-3 above the groundwater table. Using the Tokimatsu and Seed (1987) method for evaluating seismically induced settlement in dry sand, we estimate settlements of up to 2½ inches of seismic densification at Boring B32-3 during a major earthquake; in Borings B30-1, B302, B30-4, B30-5, B31-3, B31-5 and B32-2 we estimate less than ½ inch is likely to occur during a major earthquake. The soil above the groundwater table encountered in the other borings is either very clayey or has sufficient density to resist seismic densification; therefore, we conclude the potential for seismic densification to occur is low at these locations.

7.5 Tsunami

According to published data (URS Blume, 1974) the maximum recorded run-up (tsunami wave) at the Presidio occurred after the 1964 Alaskan earthquake. The wave measured 7.5 feet at the Golden Gate; no damage was reported along the San Francisco shoreline. Based on recent published maps (California Emergency Management Agency, 2009), the eastern property line borders the edge of the tsunami inundation area.

8.0 DISCUSSION AND CONCLUSIONS

Geotechnical issues of concern that should be addressed during the selection and design of a safe economical foundation system include:

- static and seismic settlement of the ground surface
- potential for liquefaction and lateral spreading
- the presence of weak, compressible soils and adequate foundation support
- variability in depth to supporting soil/bedrock
- soil corrosivity.

Each of these issues is discussed in the following subsections.

8.1 Settlement

The results of consolidation testing indicate the Bay Mud is normally to overconsolidated. Where the Bay Mud is normally and overconsolidated, consolidation is complete under the existing fill loads that were placed in the late 1800s to early 1900s; where the Bay Mud is overconsolidated, the Bay Mud has been under a greater load in the past than it is currently under. These results are consistent with the thickness of the Bay Mud, the length of time the fill has been in place, and the history of site use. Therefore, primary settlement is complete under the weight of the existing fill and secondary compression (strain-related movements) is occurring. Where new fill is placed, a new cycle of primary consolidation will begin and additional settlement will occur. Considering the variable thickness of new fill, existing fill, and Bay Mud, and the variable stress history of the Bay Mud, the amount of settlement will differ across the site.

Based on our review of Schematic Design Drawings (Flad Architects, 2011), Preliminary Elevation Plans (Tom Leader Studio, 2011) and the current topographic survey (Sherwood Engineers, 2011), approximately one to six feet of new fill will be placed to achieve the proposed grades for the "at-grade" portion of the building pads and landscape areas. For the service tunnel, service areas in Block 29 and 32 Buildings, the auditorium in the Block 31 building and the parking basement for the Block 30 Building, excavations of approximately 15 to 28 feet below the existing ground surface are proposed. Settlement will occur under the load of new fill and rebound will occur within and adjacent to the proposed excavation. We have used the drawings and results of our studies to estimate the amount of settlement that could occur at the site over the next 50 years. Specific estimates of consolidation-related settlements over the next 50 years at settlement points SP-1 through SP-34 (shown on Figure 13) are presented in Table 7. At the ground surface above the footprint of pile-supported structures, settlement is not expected; however, we anticipate if new fill is added to form the floor slab of pile supported structures, the ground will settle away from the slab.

TABLE 7
Settlement Estimates

Settlement Point No. [1]	2011 Existing Ground Elevation [2,3] (feet)	Proposed Grade [2,4] (feet)	Existing Fill Thickness[5] (feet)	Bay Mud Thickness[5] (feet)	Settlement [6,7] (inch)
SP-1	102.2	104.0	20	40	0.4
SP-2	102.2	104.0	20	29	0.3
SP-3	102.2	104.0	15	29	2.8
SP-4	100.5	104.0	25	43	1.1
SP-5	101.5	103.0	28	21	5.9
SP-6	91.0	103.5	11	13	1.7
SP-7	100.8	103.0	32	15	0.9
SP-8	96.0	104.0	21	17	2.9
SP-9	96.5	104.0	13	23	0
SP-10	96.0	104.0	13	23	-2.1
SP-11	100.2	101.5	23	12	-2.1
SP-12	101.3	103.0	12	25	0.4
SP-13	99.0	101.0	15	11	-0.1
SP-14	100.3	101.0	31	22	-0.4
SP-15	99.0	101.0	14	23	0.5
SP-16	100.5	101.5	12	10	-2.0
SP-17	100.8	103.0	27	18	0.2
SP-18	100.8	103.0	19	31	2.3
SP-19	100.5	104.0	23	29	0.4
SP-20	102.2	102.4	20	34	0
SP-21	102.2	102.4	20	34	0
SP-22	102.2	104.0	20	40	0.3
SP-23	102.2	104.0	20	40	0.3
SP-24	102.2	104.0	20	40	0.4
SP-25	102.2	104.0	20	40	0
SP-26	101.5	102.4	15	29	1.4
SP-27	101.5	102.4	17	6	0
SP-28	101.5	102.4	17	6	0
SP-29	101.5	102.7	22	3	0
SP-30	101.5	102.7	22	3	0
SP-31	100.0	100.5	12	3	-0.9
SP-32	101.5	100.6	12	3	0
SP-33	100.3	101.0	31	22	-0.7
SP-34	100.5	101.5	31	22	-2.0

Notes:
1. Refer to Figure 13 for Settlement Point locations.
2. All elevations reference San Francisco City Datum plus 100 feet.
3. Ground elevations are from drawing titled "x-site-survey.dwg" by Sherwood Design, emailed to T&R.
4. Proposed grades from plan titled "Proposed Preliminary Elevation Study", dated 10/17/11, by Tom Leader Studio.
5. Based on investigations by Treadwell & Rollo and others within site and site vicinity. Thickness estimated to nearest one foot and interpolated between borings.
6. Does not include seismically-induced settlement or secondary compression.
7. Positive indicates downward settlement; negative settlement indicates rebound.

As discussed previously, we estimate 1 to 6 inches of liquefaction-induced settlement and less than ½ inch of seismic densification may occur during a major earthquake (except at B32-5 where 2½ inches of seismic densification may occur). This settlement is in addition to the predicted consolidation settlement shown in Table 7. Even if the fill beneath the site is improved to mitigate liquefaction (as discussed in Section 8.2), up to six inches of liquefaction-induced settlement may occur in surrounding unimproved areas. In addition, because portions of the liquefiable layer are likely too deep to economically improve, liquefiable zones will remain which could result in one to five inches of liquefaction induced settlement.

Therefore, static and seismic settlement could affect various aspects of the planned development, including lateral resistance of piles, pile caps and grade beams, utilities, building entrances, sidewalks, and improvements in the plaza. Abrupt differential settlement will occur where elements connect to or cross over pile-supported structures. Design of the project should incorporate features to mitigate the effects of the predicted settlements.

Flexible connections and hangers should be used for utilities that connect to and/or extend beneath the buildings where settlement is expected. Additionally, exterior slabs and ramps attached to buildings should be designed to accommodate differential settlement between the buildings and exterior ground at all entrances and sidewalks. Other on-grade improvements should be designed to accommodate the expected differential settlement where they cross pile-supported structures. Maintenance of utilities, sidewalks and entry slabs should be expected throughout the life of the project. This may include periodically replacing some of the improvements at the building/exterior area interface. Because of the potential for large settlements during an earthquake, it may be necessary to replace exterior on-grade improvements after an earthquake.

8.2 Ground Improvement

Because potentially liquefiable soils are present across the entire project site, large vertical displacement (up to six inches) and lateral spreading could occur at the site during a large earthquake. If liquefaction occurs, the ability of the piles to resist lateral loads will be reduced, induced moments in the piles will be increased significantly, and passive pressure for basement walls, pile caps and grade beams will be significantly reduced. Where lateral spreading occurs, additional loading on the piles and basement walls will occur due to the soil movement, causing significant damage.

We therefore conclude the most practical and economical solution to reduce movements and provide pile resistance is to improve the consistency of the liquefiable layer. On the basis of our experience with the different methods of improvement, we judge rapid impact compaction[15] or stone columns[16] throughout the site would be the most appropriate methods to improve the fill and mitigate against liquefaction and lateral spreading. Another method is compaction grouting[17]; however, because of the pressures required for this method, we believe there is insufficient overburden (weight) to resist heave and properly improve the fill. Therefore, we rejected this method of improvement for this site.

From our discussions with a local soil improvement specialty contractor, we conclude that rapid impact compaction (RIC) is the most economical method for this at the site. RIC employs a tamping device mounted on an excavator. The tamping device consists of a five-foot-diameter "foot" that is placed on the ground and struck with a 7.5-ton weight that is dropped from a height of about one meter to impart energy to the ground surface. The energy is delivered at a rate of about 40 to 60 blows per minute. RIC is generally performed in a grid pattern with application points spaced approximately 6 to 8 feet on center. On other sites within Mission Bay, RIC has proven effective at densifying granular soils typically within the upper approximately 15 feet. Where loose to medium dense sandy soils are deeper than 15 feet bgs, RIC may not be able to mitigate their liquefaction potential. Therefore, we have assumed in our analyses that only the upper 15 feet of fill will be mitigated against liquefaction, and estimate up to five inches of liquefaction-induced settlement may still occur in the deeper fill in improved areas. Where potentially liquefiable fill is deeper than 15 feet, the site grade can be lowered for the RIC work to allow improvement of the deeper fill. If deeper improvement is needed and the site cannot be lowered sufficiently, stone columns should be used.

Because of existing improvements surrounding the site, vibrations from ground improvement should be monitored; setbacks and/or trenches may be warranted to reduce the potential of damage. The results of vibration monitoring on a nearby site showed that setbacks of about 10 to 15 feet are effective.

15 The rapid impact compaction method uses a Rapid Impact Compactor (RIC) to impart energy by dropping a 7.5 ton weight from a controlled height of about 1 m onto a patented foot. Some applications for which the method is used include compaction of loose soils to improve bearing capacity and mitigate liquefaction potential.

16 Stone columns are a liquefaction mitigation measure in which crushed rock columns are installed, usually using vibratory equipment, which displaces the potentially liquefiable soil with compacted crushed rock. This technique densities the in-place granular soil and provides a path for rapid dissipation of excess pore water pressures.

17 Compaction grouting is a ground improvement technique in which cement grout is injected under high pressure to increase the density of the soil, thereby reducing the liquefaction potential.

If RIC is performed, it will cause uneven settlement of the ground surface. Consequently, additional fill will be required to raise site grades where no basements are planned. Our experience at other Mission Bay sites, where this method was used, indicates one to two additional feet of fill may be required to raise site grades in addition to the fill needed for the building pad.

If deeper improvement is needed, stone columns may be used. Stone columns can be installed to the full depth of the fill. As with RIC, vibrations should be monitored and a setback may be needed to avoid damage to nearby improvements.

Any site improvement technique should be performed under our observation, and test sections will be required to confirm levels of improvement.

8.3 Foundations

The factors influencing the selection of a safe, economical foundation system with adequate capacities are:

- the presence of heterogeneous and potentially liquefiable fill
- the presence of weak, compressible Bay Mud
- concerns regarding total and differential settlement if building loads are imposed on the fill and Bay Mud
- the variations in thickness, density, and depth of potential bearing layers.

The fill in its present condition is not capable of providing adequate support for a shallow foundation system; erratic and unpredictable settlement could occur during an earthquake. Even if the fill is improved, the Bay Mud beneath the site is weak, varies in thickness, and will consolidate under the weight of building loads. The settlement caused by the new building and fill loads would be excessive and would damage the building. Considering the poor quality of the fill and the anticipated total and differential settlement created by Bay Mud consolidation, we conclude a deep foundation system consisting of driven piles is the most appropriate and economical method for support of the buildings and floor slabs. To provide sufficient capacity and limit settlement of the buildings, we conclude the piles should extend below the fill and Bay Mud and gain support primarily from end bearing in the very dense sand or bedrock. Alternative types of deep foundations may be considered.

Based on our review of the proposed elevation plan, we anticipate portions of the footprints of Buildings 29 and 31 will be raised. Because of the additional settlement caused by the placement of new fill at these buildings, piles for these two buildings will experience downdrag loads in addition to building loads, except within the footprint of proposed basements. Downdrag is the additional load transferred to the piles when the Bay Mud surrounding them consolidates. The downward movement of the compressible soil layer and the overlying soils with respect to the pile imposes negative (downward) frictional stresses on the pile. Downdrag loads are developed where sufficient strain occurs in the soil to transfer load to the pile. Accordingly, the piles at this site should be designed to support these downdrag loads in addition to the building loads.

Driven piles, especially displacement type piles, typically encounter refusal in very dense, clean sand layers greater than about 10 feet thick. If a significant amount of fines (greater than about 10 percent of either clay or silt) are present, the pile will generally not achieve refusal in the layer. Furthermore, if silt or clay layers are present below a thin layer of sand, the pile may punch through the sand. Where piles extend to bedrock, the bedrock surface varies in elevation as does the hardness of the rock. Consequently, pile lengths may vary dramatically across the site. Where piles do not meet refusal in dense sand, they should be driven to refusal in bedrock. A detailed discussion for each block is presented in Sections 8.3.1 through 8.3.4.

8.3.1 Building 29

A dense to very dense sand of the Colma Formation with varying amount of clay and silt was encountered below the Bay Mud in all the borings drilled within the footprint of the proposed Block 29 building. The very dense sand encountered in these borings (where SPT-N value is greater than 50 blows per foot) is approximately 18 to 35 feet thick with percent fines ranging from 8.1 to 10.8, where tested. From our experience during pile driving in similar soil in Mission Bay, we anticipate the piles will meet refusal and achieve end-bearing capacity in the sand layer. Figure 5 shows contours of the elevation of the top of the very dense portion of the Colma Formation sand, where encountered, in addition to the estimated layer thickness at our boring locations.

We considered several pile types for the Block 29 building, including prestressed, precast concrete piles (PSPC), steel pipe piles, and steel H-piles. On the basis of our past experience at sites in the vicinity, we judge 14-inch-square, PSPC piles would be the most economical driven pile type, although their design will need to account for the variability expected in the length of the piles due to some variation in the depth to and density of the sand. Steel piles or alternative piles, such as auger cast or torqued in piles, would also be appropriate.

The depth of pile embedment into the sand depends on its density and percent fines; for planning purposes, we estimate PSPC piles should encounter refusal approximately 5 to 8 feet into the very dense sand layer of the Colma Formation (approximate pile tip elevation ranging from Elevation 17 to 26 feet).

8.3.2 Building 30

A dense to very dense sand of the Colma Formation, with varying amount of clay and silt, was encountered in Borings B30-1 through B30-4 and B32-1 within the footprint of the proposed Block 30. The very dense sand encountered in these borings, where SPT-N value is greater than 50 blows per foot, is approximately 12 to 23 feet thick with percent fines ranging from 5.6 to 29.2, where tested. From our experience during pile driving in similar soil in Mission Bay, we anticipate the piles will meet refusal and achieve adequate end-bearing capacity in the sand layer. Figure 5 shows contours of the elevation of the top of the very dense portion of the Colma Formation sand,, where encountered, in addition to the estimated layer thickness at our boring locations. The sand was not encountered in the southeastern portion of the building footprint, specifically at Borings B30-5 and B32-2; in this area, bedrock becomes somewhat shallower. Therefore, where the sand becomes thin or is not present, piles should be driven to refusal in bedrock. Where piles transition from end bearing in the dense sand to end bearing in bedrock, there could be sudden changes in pile length and the piles in the transition zone will need to be designed with higher cutoff allowances.

We considered several pile types for the Block 30 building, including PSPC piles, steel pipe piles, and steel H-piles. On the basis of our past experience at sites in the vicinity, we believe 14-inch-square, PSPC piles would be the most economical driven pile type, although their design will need to account for the variability expected in the length of the piles due to variations in bearing conditions. Steel piles or alternative piles, such as auger cast or torqued in piles, would also be appropriate.

Where piles are driven to refusal in very dense sand, the depth of pile embedment into the sand depends on its density and percent fines; for planning purposes, we estimate PSPC concrete piles should encounter refusal after penetrating 5 to 8 feet into the very dense sand layer of the Colma Formation (approximate pile tip elevation ranging from Elevation 17 to 35 feet). Where the very dense sand layer is thin or was not encountered, piles should be driven to refusal in bedrock; for planning purposes, we estimate PSPC piles should encounter refusal 5 to 10 feet into bedrock (approximate pile tip elevation ranging from Elevation 5 to 40 feet). Approximate top of bedrock elevations contours are shown on Figure 6.

8.3.3 Building 31

During our investigation, we encountered the very dense sand layers of the Colma formation in Borings B31-2 through B31-5. However, the sand layers encountered in Borings B31-2 through B31-5 are not sufficiently thick, dense, and/or clean to provide adequate pile capacity. Therefore, we judge piles at Block 31 should be driven to refusal in the bedrock.

The elevation of bedrock varies across the building footprint, as does the hardness of rock, and pile lengths will vary significantly across the building footprint. We considered several pile types for the Block 31 building, including PSPC piles, steel pipe piles, and steel H-piles. With the large variations in pile lengths expected, however, we judge that PSPC piles would not be appropriate. Steel piles can more easily accommodate these variations as they can be cut off or spliced as needed, thereby reducing waste.

Therefore, we judge steel piles are the most appropriate pile type for Building 31. Fourteen-inch steel H-piles are typically the most economical steel pile type in the region; however, other steel pile types are acceptable. Alternative pile types, such as auger cast or torqued in piles, are also appropriate.

For planning purposes, we estimate 14-inch steel H-piles will typically encounter refusal approximately 10 to 15 feet into bedrock (approximate pile tip elevation ranging from Elevation -10 to 50 feet), although some piles may extend up to 30 feet into rock, based on driving steel H-piles at a nearby site. Approximate top of bedrock elevations contours are shown on Figure 6.

8.3.4 Building 32

During our investigation, the very dense sand layers of the Colma Formation were not encountered within the footprint of the propose Building 32. Therefore, we judge piles at Block 32 should be driven to refusal in the bedrock. For budgeting purposes, we estimate piles should encounter refusal after penetrating 5 to 20 feet into bedrock (approximate pile tip elevation ranging from 30 to 55 feet). Approximate top of bedrock elevations contours are shown on Figure 6.

We considered several pile types for this project, including PSPC and steel H-piles. Based on the subsurface information from our borings, the bedrock surface does not vary more than about 15 feet across the building footprint. We recommend the building be supported on driven, 14-inch square PSPC with a 10-foot-long steel H-pile stinger; a steel H-pile stinger at the tip of the concrete pile will allow deeper embedment into the bedrock and protect the concrete pile tip when transitioning the shorter piles from soil to rock. Steel piles and alternative piles, such as auger cast or torqued in piles, would also be appropriate.

8.3.5 Service Tunnel

A below-grade service tunnel is proposed to run in the north-south direction from 16th to South Street, between Buildings 29 and 30 and between Buildings 31 and 32. The service tunnel will connect the service areas of all four buildings. The proposed finished floor elevation of the tunnel is approximately 25 feet below the proposed ground surface, at approximately Elevation 75 feet.

Based on the subsurface conditions encountered in the vicinity of the service tunnel structure, the very dense sand encountered in borings along the northern portion of the tunnel (where SPT-N value is greater than 50 blows per foot) is approximately 20 to 28 feet thick. From our experience during pile driving in similar soil in Mission Bay, we anticipate the piles will meet refusal and achieve adequate end-bearing capacity in the sand layer. In the southern portion of the tunnel, the very dense sand layer thins out and is not present; therefore, the southern portion of the tunnel should be supported on piles driven to bedrock. Both 14-inch PSPC concrete piles and 14-inch steel HP-piles may be considered. For budgeting purposes, we estimate piles on the northern segment of the tunnel should encounter refusal after penetrating 5 to 8 feet into the very dense sand layer of the Colma Formation (approximate pile tip elevation ranging from 17 to 35 feet) and piles in the southern segment of the tunnel should encounter refusal after penetrating 5 to 20 feet into bedrock (approximate pile tip elevation ranging from 0 to 55 feet). Approximate top of very dense sand and top of bedrock elevations contours are shown on Figures 5 and 6, respectively.

8.3.6 Hardscaped Plaza and Ancillary Landscape Structures

We understand there will be a hardscaped plaza with several landscape structures planned between the four buildings. At this time, the structural loads of the plaza and the landscape structures are unknown. The existing fill, which overlies the Bay Mud, was not compacted during placement, but with some overexcavation and recompaction should have sufficient strength to support light loads (up to 1000 pounds per square foot (psf)). For lightly loaded structures, spread footings or mat foundations may be utilized, however settlement will occur. For heavier structures or structures with low settlement tolerance, pile foundations should be used.

We understand the hardscaped plaza may be supported on piles, rather than on grade. These piles will be relatively lightly loaded, although most will have loads due to downdrag in addition to the plaza loads. We understand the loading of the proposed hardscaped plaza and ancillary structures in the plaza are not yet known. Until more design is developed, foundations for these structures can be assumed to be similar to foundations for the nearest building. Once more is known about these structures, we can provide additional recommendations for the foundation systems if requested.

8.3.7 Foundation Construction Considerations

Even with an extensive indicator pile program, it will be difficult to predict actual pile lengths for production, as piles should be driven to refusal and the bearing layers are variable. In many cases we anticipate that concrete and steel piles will meet refusal prior to being driven their full production length because of the variations in depth and hardness of rock and density and thickness of the Colma Formation sand layer over short distances. Consequently, most of the piles will likely require cutoff. Cutoff allowances will vary, depending on the building. For example, at Blocks 29, 31 and 32, we would expect an average cutoff allowances of approximately 10 to 15 feet, depending on the embedment of the piles into the very dense sand or bedrock. In Block 30, if piles do not penetrate the sand layer where expected, up to about 20 feet of cutoff could result.

8.3.8 Foundation Settlement

Piles will transfer building loads to relatively incompressible dense sand or bedrock; however, some settlement of the piles will still occur. The primary contributions to settlement for shorter piles bearing in the sand will consist of elastic compression and consolidation of the underlying clays from the foundation stresses transferred through the sand, while settlement of piles bearing in bedrock will primarily consist of elastic compression. We estimate total settlement of both pile types will range between approximately 1 to 1-1/2 inches, depending on the load on the pile, the type and length of the pile and the presence of compressible soil below the pile tip. Most of the settlement of piles bearing in bedrock is anticipated to occur during construction, while about 1/2 to 2/3 of the settlement for piles bearing in sand is anticipated to occur during construction, with the remainder occurring within a few years after construction is complete. We estimate differential settlement will be less than 1/2 inch between adjacent columns supported on new piles, both during construction and at the completion of foundation settlement.

8.4 Soil Corrosivity

A detailed corrosivity evaluation was performed by JDH Corrosion, and the results of its study are presented in Appendix D. The results of the JDH analysis of various soil samples indicate the fill at the site is classified as generally corrosive whereas Bay Mud is generally classified as severely corrosive.

Unprotected steel elements placed below grade will corrode; protection of foundations, utilities, and other structural elements, which extend into these layers, will be required. For more detail, see the recommendations by JDH Corrosion Consultants in Appendix D.

8.5 Groundwater

The groundwater level was encountered between Elevation 89 to 93.4 feet during our investigation. In other borings drilled in the area, groundwater was encountered several feet higher. The groundwater elevation is likely influenced by tidal fluctuations, as well as by wet and dry seasons. We conclude a design groundwater elevation of 95 feet should be used in design. .

Assuming a four foot thick structural mat slab and two-foot thick crushed rock working pad, excavations for below grade portion of the structures will extend as deep as approximately 12.5 feet (Block 31 Building's Auditorium) to approximately 24 feet (Block 30 Building's parking basement) below the design groundwater table; groundwater will need to be removed from the excavations during construction. The method of dewatering will depend to an extent on the method of shoring. Groundwater seepage through the Bay Mud should be slow, though flow through the fill could be high. If a secant pile wall, sheet pile wall or other type of tight shoring system is used, the groundwater flow should be reduced and groundwater removal can be achieved using sumps and pumps. Other shoring systems that are less resistant to the flow of groundwater would require dewatering wells to lower the groundwater in the fill; however, dewatering wells are not effective in Bay Mud or in close vicinity to San Francisco Bay. Therefore, shoring systems that do not cut off the majority of water flow, such as soldier pile and lagging, are not feasible.

8.6 Excavation and Shoring

Where there is not sufficient room to allow temporary, sloped cuts and where excavations extend below groundwater, the excavations should be retained by shoring. A properly braced cutoff shoring wall should provide satisfactory temporary support and limit seepage. There are several key considerations in selecting suitable shoring and underpinning systems. Those we consider of primary concern are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures
- the ability of the shoring system to minimize the inflow of groundwater and required dewatering
- the ability of the shoring system to reduce potential for ground movement
- cost.

We understand several shoring systems are being considered, including soil-cement-mixed walls and secant pile or stitch walls with internal bracing and/or tiebacks. We judge that both systems are viable for the project. The issues associates with these systems are discussed in the remainder of this section.

Soil-cement mixed walls are installed by advancing hollow-stem augers and pumping cement slurry through the tips of the augers during auger withdrawal or by jet-grout methods. The soil is mixed with the cement slurry in situ, forming continuous, overlapping, soil-cement columns. Steel beams are placed in some of the soil-cement columns to provide rigidity. Soil-cement walls are considered temporary; permanent walls are usually built inside of the soil-cement walls. Because these walls are continuous they will act to temporarily cut-off groundwater infiltration, resulting in less dewatering. In addition, soil-cement walls are generally more rigid than soldier-piles and lagging and usually result in less shoring deformations.

Secant piles are drilled shafts that overlap to form a continuous wall. The wall is constructed by drilling alternate shafts and then "back stepping" to drill the intervening shafts in order to interlock the two adjacent shafts. Every second shaft is reinforced usually with a wide flanged steel section or reinforcing steel cage. The reinforced shafts are called "primaries". The alternate shafts, which are not reinforced, are called "intermediates" or "secondaries". The concrete used for the secondary piles is usually lean concrete that remains soft enough for the drilling and interlocking of the primary shafts. The primaries are usually poured with structural concrete.

The shoring will likely require either grouted tiebacks or internal bracing, depending on whether encroachment permits can be obtained to drill beneath the city streets, Port of San Francisco property and/or adjacent structures.

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

The design, construction, and performance of the shoring system should be the responsibility of the contractor and should be designed by a structural engineer knowledgeable in this type of construction. The design of the selected dewatering system should be provided to the shoring designer so that the temporary groundwater elevation can be incorporated in the shoring design. Geotechnical recommendations for the shoring design are provided in Sections 9.6 of the report.

8.7 Construction Considerations

The fill soil at the site consists mainly of sand, gravel and clay that can be excavated with conventional earth-moving equipment such as loaders and backhoes. The fill is easily remolded and loses strength when wet. Therefore, site preparation and grading may be difficult if performed during the rainy season. In addition, heavy vibratory equipment should not be used during site preparation and grading; a vibratory compactor will likely cause a capillary rise of the groundwater, creating a wet subgrade.

Brick, rock, concrete, and other building rubble may be encountered in the fill. Boulders and cobbles are likely present. Installation of shoring and foundations may be difficult in some areas of the site. Piles may be damaged if driven into obstructions. To mitigate the potential for damage, predrilling or vibrating (steel pile only) the initial pile section should be evaluated during the indicator pile programming; removal of obstructions may still be needed.

The fill most likely contains heavy metals and petroleum hydrocarbons. Handling and disposal of the fill material should be performed in accordance with a site mitigation plan (SMP) that includes health and safety criteria; preparation of an SMP is not within the scope of this investigation.

Where excavations extend below the design groundwater elevation of Elevation 95 feet, or about 6 to 9 feet below final site grades, dewatering will be required. Prior to construction, the groundwater should be tested to evaluate if it can be discharged directly to the storm drain system or if it must be treated on-site prior to discharge.

9.0 RECOMMENDATIONS

From a geotechnical standpoint, we conclude 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 and are implemented during construction. Criteria for foundation design, together with recommendations for site preparation, floor slabs, and seismic design are presented in this section of the report.

9.1 Pile Foundations

We recommend the structures be supported on driven, 14-inch square prestressed precast (PSPC) concrete piles or 14-inch, steel H-piles gaining capacity through friction in the soil below the Bay Mud and end bearing in the dense sand and/or in bedrock.

In the past, we performed static pile load tests at other Mission Bay sites with similar subsurface conditions. Static pile load tests were performed on both 14-inch square PSPC concrete piles and HP14x117 steel piles driven to refusal in dense sand or bedrock. Pile driving analyzer (PDA) load tests have also been performed on a number of driven piles in similar conditions. Correlating the results of the earlier static load tests with the subsurface conditions at this site, we developed recommended pile capacities for 14-inch piles driven to refusal in dense sand or bedrock. The capacities presented in the following sections represent the capacities as determined by load tests on 14-inch square PSPC concrete piles and HP14x117 section steel piles. The piles should be driven into the bearing layer and have sufficient structural capacity to support building loads and downdrag loads, if any. To avoid axial capacity reduction due to group effects, piles should be spaced at least three pile widths apart, measured center to center.

Concrete piles should be designed to mitigate the effects of corrosion and steel piles should either be cathodically protected or include an allowance for corrosion; recommendations regarding corrosion are included in Appendix D. The structural engineer should check that the piles have sufficient structural capacity after the section is reduced due to corrosion.

Where noise and/or vibrations due to driving piles are unacceptable, alternative pile types, such as auger cast piles or torqued-in-place steel pipe piles, may be used for foundation support. These specialty piles are typically design-build items; their design and capacity should be determined by experienced specialty foundation subcontractors.

9.1.1 Axial Capacity for Block 29, 30, 31, and 32

The following subsections present the recommended axial pile capacities. The capacity of the pile section should be checked for the dead plus live load plus downdrag load; the pile cross section for steel piles will be governed either by this loading combination or the lateral load demand.

9.1.1.1 Block 29

The building at Block 29 should be supported on PSPC piles driven below the fill and Bay Mud to refusal in the very dense sand. Table 8 presents the recommended axial pile capacities for the Block 29 building support. Due to the planned below grade service area, we have divided the site into zones, designated as Zone 29-1 and Zone 29-2, corresponding to different downdrag loads and uplift capacities, as shown on Figure 14. Zone 29-1 is the at-grade portion of the building and Zone 29-2 is the below-grade service area.

TABLE 8
Recommended Pile Capacities
Block 29

Pile Type	Zone	Bearing Layer	Assumed Top of Pile Elevation (feet)	Estimated Pile Tip Elevation (feet)	$Q_{ultimate}$ Axial Capacity (kips)	$Q_{ultimate, DD}$ Downdrag Load (kips)	$Q_{net\ allowable}$ Dead Plus Live Loads[1,2] (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift[3] (kips)
14-inch Square PSPC Concrete Pile	29-1	Very Dense Sand	94	17 to 26	800	95	350	470	90
	29-2	Very Dense Sand	79	17 to 26	800	0	400	530	75

Notes:
1. Includes a Factor of Safety (FS) of 2.0. Loads on pile, including downdrag, should not exceed ultimate structural capacity of pile.
2. To apply downdrag reduction factors (DRF) for a single pile in a group (Table 9), use: NET $Q_{allowable} = [Q_{ultimate} – (Q_{ultimate,DD} \times DRF)]/FS$ (see Note 1 above).
3. The allowable uplift capacity includes a FS of 2.0 (temporary load). For permanent allowable uplift capacity, a FS of 3.0 should be applied.

The capacities shown in Table 8 represent the capacity of the soil only; the load placed on the pile plus the downdrag load should not exceed the structural capacity of the piles, as determined by the 2010 SFBC code. Several factors should be considered when evaluating the length of the piles. As previously discussed, because some of the piles will achieve refusal in the dense sand with various embedment lengths, it is difficult to predict their finished length throughout the site. Accordingly, piles may have cutoff lengths; we judge cutoff lengths will be up to about 20 feet. A cutoff allowance should be incorporated into the pile design. The actual cutoff allowance should be evaluated during an indicator pile program. Where piles will be installed in groups, the downdrag load can be reduced to account for group effects, using the factors presented in Table 9.

TABLE 9
Downdrag Reduction Factors (DRF)

Number of Piles within Pile Cap	Downdrag Reduction Factor[1,2]
1	1.0
2	0.8
3	0.7
4 or more	0.6

Notes:
1. Multiply ultimate downdrag by reduction factor to obtain average downdrag per pile in a group.
2. Downdrag reduction factors assume a pile spacing of three diameters, center to center.

For budget purposes, the pile lengths should be estimated based on 5 to 8 feet of embedment in the very dense sand; consequently, we estimate production pile lengths will be approximately 70 to 80 feet within the at-grade portion of the structure (assuming pile cap depth of approximately nine feet) and approximately 55 to 65 feet within the below-grade service area. The pile length will depend on the thickness and density of the very dense sand. Better predictions of lengths and cutoff allowance will be determined using the results of an indicator program. However, we recommend anticipating 100 percent of the production piles will require some cutoff. For budgeting purpose, we anticipate an average cutoff allowance of 15 feet for each production pile.

9.1.1.2 Block 30

The building at Block 30 should be supported on PSPC piles driven below the fill and Bay Mud to refusal in very dense sand or bedrock. The recommended axial pile capacities for the Block 30 building piles are presented on Table 10. The estimated support zones are shown as Zone 30-1 in very dense sand of the Colma Formation and Zone 30-2 in bedrock, as shown on Figures 15 and 16, respectively.

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TABLE 10
Recommended Pile Capacities
Block 30

Pile Type	Zone	Bearing Layer	Assumed Top of Pile Elevation (feet)	Estimated Pile Tip Elevation (feet)	$Q_{ultimate}$ Axial Capacity (kips)	$Q_{ultimate, DD}$ Downdrag Load (kips)	$Q_{net\ allowable}$ Dead Plus Live Loads[1] (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift[2] (kips)
14-inch Square PSPC Concrete Pile	30-1	Very Dense Sand	74.5	17 to 35	750	0	375	500	70
	30-2	Bedrock	74.5	5 to 40	750	0	375	500	65

Notes:
1. Includes a Factor of Safety (FS) of 2.0. Loads on pile should not exceed ultimate structural capacity of pile.
2. The allowable uplift capacity includes a FS of 2.0 (temporary load). For permanent allowable uplift capacity, a FS of 3.0 should be applied.

The capacities shown in Table 10 represent the capacity of the soil and rock only; the structural capacity of the pile as determined by the 2010 SFBC code may be less. The load placed on the pile should not exceed the structural capacity of the piles, as determined by the 2010 SFBC code.

Several factors should be considered when estimating the length of the piles. As previously discussed, because some of the piles will achieve refusal in the dense sand or in bedrock with various embedment lengths, it is difficult to predict their finished length throughout the site. Accordingly, piles may have cutoff lengths up to 20 feet. A cutoff allowance should be incorporated into the pile design. The actual cutoff allowance should be evaluated during an indicator pile program.

For budget purposes, the pile lengths should be estimated based on 5 to 8 feet embedment in sand in Zone 30-1 and 5 to 10 feet embedment in bedrock in Zone 30-2; consequently, we estimate production pile lengths should be between approximately 40 to 60 feet Zone 30-1 and 35 to 70 feet in Zone 30-2. The pile length will depend on the thickness and density of the very dense sand and/or on the depth and consistency of the Franciscan Complex. Better predictions of lengths and cutoff allowance will be determined using the results of an indicator program. However, we recommend anticipating 100 percent of the production piles will require some cutoff. For budgeting purpose, we anticipated an average cutoff allowance of 10 feet for each production pile, although we anticipate cutoff allowances as high as 20 feet will be required in the transition from piles end bearing in sand to piles end bearing in rock.

9.1.1.3 Block 31

We recommend the Block 31 building be supported on driven, 14-inch, steel H-piles gaining capacity through friction in the soil below the Bay Mud and end bearing in bedrock. Steel piles should have a driving shoe on the tip. Due to the variation in the subsurface conditions at the site and the planned basement elevations, we have divided the site into various zones corresponding to different compression, downdrag and uplift capacities. We divided the site into two zones for compression capacity, designated Zone 31-C1 and Zone 31-C2, as shown on Figure 17. Downdrag loads are presented for four zones, designated Zone 31-DD1 through Zone 31-DD4, as shown on Figure 18. We determined uplift capacity in four zones, designated Zone 31-U1 through Zone 31-U4, as shown on Figure 19. In computing the downdrag loads and uplift capacities, we have assumed the top of pile elevation of 94.5 feet for the at-grade portion of the building, Elevation 86.5 feet within the auditorium footprint and Elevation 79 feet within the service area.

The recommended pile capacities for 14-inch steel H-piles driven to refusal in bedrock are presented in Table 11, and downdrag loads are presented in Table 12.

TABLE 11
Compression Pile Capacity
14-inch Steel H-Pile
Block 31

Zone	Estimated Pile Tip Elevation (feet)	$Q_{ultimate}$ Axial Capacity[1] (kips)
31-C1	-10 to 25	800
31-C2	25 to 50	750

Note: 1. Loads on pile should not exceed ultimate structural capacity of pile.

To determine the allowable capacity of a single pile in a group, the following equation may be used:

[Equation 1]: $NET\ Q_{allowable} = [Q_{ultimate} - (Q_{ultimate,DD} \times DRF]/FS$

where: $Q_{ultimate,DD}$ = Downdrag Load (Table 12)
DRF = Downdrag Reduction Factor (Table 9)
FS = Factor of Safety

TABLE 12
Recommended Downdrag Loads
Block 31

Zone	$Q_{ultimate.DD}$ Downdrag Load (kips)
31-DD1	85
31-DD2	55
31-DD3	25
31-DD4	0

The net allowable load may be increased by one-third for total design loads, including wind and earthquake loads.

The capacities shown in Table 11 represent the capacity of the soil and rock only; the structural capacity of the selected steel section as determined by the 2010 SFBC code may be less. Loads on pile should not exceed ultimate structural capacity of pile. Check by multiplying load on pile by appropriate load factor and adding $Q_{ultimate,\ DD}$. The same when compared to the axial capacities of Table 11, should have an appropriate factors of safety. We recommend the Factors of Safety (FS) to be applied in Equation 1 are 2.0 for Dead plus Live Loads and 1.5 for Total Design Loads.

For budget purposes, the pile lengths should be estimated based on 10 to 15 feet embedment in bedrock. Consequently, we estimate production pile lengths should be approximately 50 to 95 feet within the at-grade portion of the structure, approximately 65 to 80 feet long within the below-grade auditorium and approximately 30 to 65 feet long within the below-grade service area.

The actual pile length will depend on the depth and consistency of the Franciscan Complex. Better predictions of lengths will be determined using the results of an indicator program. However, we recommend anticipating 100 percent of the production piles will require some cutoff or add on. For budgeting purpose, we anticipated an average cutoff allowance of 10 feet for each production pile.

Several factors should be considered when determining the section size of the piles. The actual pile section may need to be larger in the upper 30 feet of the pile to account for corrosion (unless the piles are cathodically protected and a thicker section is not needed) and lateral pile capacity. With likely varying cutoff allowances throughout the site, the thicker section will need to be long enough to account for the minimum required length of section in the ground even if 10 feet is cut off because of pile stickup. Therefore, for budgeting purposes, assuming a required thicker section of 30 feet at the top of the pile and a cutoff allowance of 10 feet, we recommend the thicker pile section be planned to be at least 40 feet long. It should be noted that if lighter sections are used below the corrosive zone, less flexibility is available within the pile length if unexpected differences in pile lengths occur.

The recommended uplift capacities on a single pile are presented in Table 13.

TABLE 13
Recommended Uplift Pile Capacities
Block 31

Zone	$Q_{ultimate}$ Uplift (kips)	$Q_{allowable}$ Temporary Uplift[1] (kips)	$Q_{allowable}$ Permanent Uplift[2] (kips)
31-U1	70	35	23
31-U2	190	95	63
31-U3	170	85	56
31-U4	225	112	75

Notes: 1. Includes a Factor of Safety (FS) of 2.0.
2. Includes a Factor of Safety (FS) of 3.0.

9.1.1.4 Block 32

We recommend the Block 32 piles consist of driven PSPC piles with a steel stinger driven to refusal in bedrock. The steel stinger should have a driving shoe on the tip. The piles will gain capacity through friction in the soil below the Bay Mud and end bearing in the bedrock. Recommended pile capacities are presented below in Table 14. Due to the variation in the subsurface condition at the site and the planned basement elevation, we have divided the site into various zones corresponding to different uplift capacities, as shown on Figure 20.

TABLE 14
Recommended Pile Capacities
Block 32

Pile Type	Zone	Bearing Layer	Assumed Top of Pile Elevation (feet)	Estimated Pile Tip Elevation (feet)	$Q_{ultimate}$ Axial Capacity (kips)	$Q_{ultimate, DD}$ Downdrag Load (kips)	$Q_{net\ allowable}$ Dead Plus Live Loads[1] (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift[2] (kips)
14-inch Square PSPC Concrete Pile with a 10-foot-long steel stinger	32-1	Bedrock	92.6	30 to 55	750	0	375	500	50
	32-2	Bedrock	79	30 to 55	750	0	375	500	35

Notes:

1 Includes a Factor of Safety (FS) of at least 2.0. Loads on pile should not exceed ultimate structural capacity of pile.

2 The allowable uplift capacity includes a FS of 2.0 (temporary load). For permanent allowable uplift capacity, a FS of 3.0 should be applied.

The capacities shown in Table 14 represent the capacity of the soil and rock only; the load placed on the pile should not exceed the structural capacity of the piles, as determined by the 2010 SFBC code. Several factors should be considered when determining the length of the piles. As previously discussed, because the surface and hardness of the bedrock is variable, it is difficult to predict the finished length of the piles throughout the site. Accordingly, piles may have cutoff lengths up to about 20 feet. A cutoff allowance should be incorporated into the pile design. The actual cutoff allowance should be evaluated during an indicator pile program.

For budget purposes, the pile lengths should be estimated based on 5 to 20 feet embedment in the bedrock; because a 10-foot-long steel stinger will be cast into the pile, the embedment should average 10 feet. Consequently, we estimate production pile lengths should be approximately 40 to 65 feet within the at-grade portion of the structure and approximately 25 to 50 feet within the below-grade service area. The pile length will depend on the depth and consistency of the Franciscan Complex. Better predictions of lengths will be determined using the results of an indicator program. However, we recommend anticipating 100 percent of the production piles will require some cutoff or add on. For budgeting purpose, we anticipated an average cutoff allowance of 15 feet for each production pile.

9.1.1.5 Service Tunnel

We recommend the piles supporting the service tunnel consist of either PSPC concrete piles or steel H-piles gaining capacity through friction in the soil below the Bay Mud and, if additional capacity is needed, in end bearing in dense sand or bedrock. Steel piles should have a driving shoe on the tip. Recommended pile capacities are presented below in Table 15. Due to the variation in the subsurface condition at the site and the planned basement elevation, we have divided the site into various zones corresponding to different compression and uplift capacities, as shown on Figure 21.

TABLE 15
Recommended Pile Capacities
Service Tunnel

Pile Type	Zone	Bearing Layer	Assumed Top of Pile Elevation (feet)	Estimated Pile Tip Elevation (feet)	$Q_{ultimate}$ Axial Capacity (kips)	$Q_{ultimate, DD}$ Downdrag Load (kips)	$Q_{net\ allowable}$ Dead Plus Live Loads[1] (kips)	$Q_{allowable}$ Total Design Load (kips)	$Q_{allowable}$ Uplift[2] (kips)
14-inch Square Pile	T-1	Very Dense Sand	77	20 to 30	750	0	375	500	50
	T-2	Bedrock	77	5 to 35	750	0	375	500	45
	T-3	Bedrock	77	25 to 60	750	0	375	500	35

Notes:
1. Includes a Factor of Safety (FS) of at least 2.0. Loads on pile should not exceed ultimate structural capacity of pile.
2. The allowable uplift capacity includes a FS of 2.0 (temporary load). For permanent allowable uplift capacity, a FS of 3.0 should be applied.

The capacities shown in Table 15 represent the capacity of the soil and rock only; the load placed on the pile should not exceed the structural capacity of the piles, as determined by the 2010 SFBC code.

Several factors should be considered when determining the length of the piles. If the design requirement of the piles can be met through skin friction only, the pile length will be determined based on the length needed in friction. However, if the piles will be end-bearing, they will achieve refusal in the sand or bedrock with various embedment lengths and it will be difficult to predict their finished length throughout the site. Accordingly, piles may have cutoff lengths up to 30 feet. A cutoff allowance should be incorporated into the pile design. The actual cutoff allowance should be evaluated during an indicator pile program.

For budget purposes, the pile lengths should be estimated based on 10 feet of embedment in the very dense sand and 5 to 20 feet embedment in the bedrock; consequently, we estimate production pile lengths should be between approximately 45 to 55 feet within Zone T-1, 55 to 75 feet within Zone T-2 and 20 to 55 feet within T-3. The pile length will depend on the thickness and density of the dense to very dense sand and/or on the depth and consistency of the Franciscan Complex. Better predictions of lengths and cutoff allowance will be determined using the results of an indicator program. However, we recommend anticipating 100 percent of the production piles will require some cutoff. For budgeting purpose, we anticipated an average cutoff allowance of 15 feet for each production pile.

9.1.2 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 moment capacity of the pile.

The lateral capacity of piles is significantly influenced by the potential for liquefaction in the fill, as well as the depth to groundwater and the presence of weak Bay Mud.

For Block 29, we have calculated the lateral capacity for a 14-inch PSPC pile for fixed- and free-head conditions for a case where the liquefaction hazard has been mitigated in the upper 15 feet (measured from an existing ground surface elevation of 100.5 feet), as discussed in Section 8.2. If the upper 15 feet of soil is not improved to resist liquefaction, these values will be reduced. For the below-grade and at-grade condition, we developed deflection and moment profiles for ½ inch of lateral deflection at the top of the pile for a 14-inch-square PSPC pile, and the curves are presented on Figures 22 through 25.

The lateral capacity of piles in Block 30 is significantly influenced by the presence of the soft Bay Mud at the top of pile elevation. We calculated the lateral capacity for a 14-inch PSPC concrete pile for fixed- and free-head conditions and developed deflection and moment profiles for ½ inch of lateral deflection at the top of the pile for a 14-inch PSPC concrete pile. The results are presented on Figures 26 and 27.

At Block 31 we calculated the lateral capacity for a HP14x73 steel pile for fixed- and free-head conditions for a case where the liquefaction hazard has been mitigated in the upper 15 feet (based on an existing ground surface elevation of approximately 101.5 feet), as discussed in Section 8.2. In addition, the lateral load analyses for the steel pile were performed in both the strong and weak axis. For the at-grade condition and the below-grade auditorium and service areas, we developed deflection and moment profiles for ½ inch of lateral deflection at the top of an HP14x73 steel pile. The results are presented on Figures 28 through 33. For piles within the at-grade portion of the building, if the upper 15 feet of soil is not improved to resist liquefaction, these values will be reduced. The actual pile section chosen for construction, based on a sacrificial corrosion allowance, will depend on corrosion allowance for the desired life-span of the piles, as discussed in Appendix D.

We calculated the lateral capacity at Block 32, for a 14-inch PSPC pile for fixed- and free-head conditions also for a case where the liquefaction hazard has been mitigated in the upper 15 feet (based on an existing ground surface elevation of 100.5 feet). For the below-grade and at-grade condition, we developed deflection and moment profiles for ½ inch of lateral deflection at the top of the pile for a 14-inch PSPC pile. The results are presented on Figures 34 through 37. If the upper 15 feet of soil is not improved to resist liquefaction, the values for the at-grade condition will be reduced.

At the time of the publication of this report, the pile type for the tunnel foundations has not been selected. For estimating and preliminary design purposes, we have assumed 14-inch square PSPC concrete piles will be used and recommend the following lateral capacity profiles for the service tunnel design:

- Zone T-1: Block 30 Building deflection and moment profiles for ½ inch of lateral deflection at the top of the pile; the curves are presented on Figures 26 through 27.
- Zones T-2 and T-3: Block 32 Building below-grade service area deflection and moment profiles for ½ inch of lateral deflection at the top of the pile; the curves are presented on Figures 36 through 37.

These 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 on Figures 38a and 38b. The moment profile for a single pile with an unfactored load should be used to check the design of individual piles in a group.

Additional lateral load resistance can be developed as passive resistance acting against the faces of the pile caps, grade beams, and key and skirt walls and below grade walls. In the fill where liquefaction potential is mitigated, an equivalent fluid weight of 270 pounds per cubic foot (pcf) and 135 pcf may be used to compute passive resistances against pile caps, grade beams, and skirt walls above and below the water table (design groundwater level of Elevation 95 feet), respectively. In Bay Mud, an equivalent fluid weight of 95 pcf may be used to compute passive resistance. The upper 18 inches below final soil subgrade should be ignored in computing passive resistance to account for settlement and an unconfined soil surface.

For the below-grade portions of buildings, passive resistance against basement walls may be used. Detailed recommendations for lateral load resistance of below-grade walls are presented in Section 9.2.

If increased lateral resistance is needed, pile caps and grade beams could be deepened and/or additional "short piles" or intermediate grade beams could be added for lateral resistance. Downdrag loads, where present, will act on short piles or grade beams.

9.1.3 Ancillary Landscape Structures

Ancillary Landscape structures should be supported on a shallow mat foundation, where settlement is acceptable, or on piles gaining capacity in friction below the fill and Bay Mud and, where needed, end bearing in dense sand or bedrock.

Where these structures will be supported on a mat, the allowable bearing capacity for landscape structures should be selected to minimize settlement. The allowable bearing pressure of shallow foundation in the fill should not exceed 1,000 psf for dead plus live load and 1,300 psf for total loads; however, the static bearing pressure should be reduced as needed to limit settlement. We can provide settlement based on bearing pressures once the loads and sizes of the structures are known. At a minimum, we recommend the mat subgrade be overexcavated and recompacted to provide at least 24 inches of engineered fill below the mat. The overexcavation and recompaction should extend five feet beyond the limits of the mat.

If pile foundations are needed, we can provide recommendations once the location and the load requirements of the structures are known.

9.1.4 Construction Considerations

We recommend an indicator pile program be performed to provide data for developing production pile driving recommendations, including estimated pile tip elevations. Indicator piles should be installed near boring locations and may be installed at column locations and can be used for support of the building.

Indicator piles can be installed at production pile locations; indicator pile locations should be selected by us and approved by the structural engineer. They should be installed with the same equipment that will be used to install the production piles.

We recommend 20 indicator piles be driven for each of the buildings at Block 29, 31 and 32. For Block 30, we recommend 30 indicator piles be driven across the building footprint to better delineate the transition of pile end bearing in very dense sand and pile end bearing in bedrock. We also recommend 10 indicator piles driven for the service tunnel. The indicator piles will provide data regarding the pile lengths necessary to achieve refusal penetration in sand or bedrock. It is difficult to predict where piles will encounter refusal in dense sand or how far the piles will extend into the dense sand or bedrock before encountering refusal. During indicator pile driving, we recommend that many of the piles be driven to high blow counts to achieve maximum penetration in the dense sand or bedrock. We will use this data to further define the presence and thickness of the dense sand layer and the variation in depth and hardness of bedrock. Therefore, we recommend indicator pile lengths be chosen to extend at least 20 feet into very dense sand at Blocks 29 and 30 and the northern portion of the tunnel and at least 20 feet into bedrock at Blocks 30, 31 and 32 and the southern portion of the tunnel. Cutoff lengths up to 30 feet should be anticipated during the indicator program, although the cutoff length may be longer at Block 30 where there is a transition between piles bearing in sand and piles bearing in rock.

Determination of driving equipment and pile section 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. The hammer selected should be able to supply sufficient energy to the pile tip to penetrate very stiff to hard clay and intermittent sand layers encountered below the Bay Mud and to penetrate at least 10 feet into very dense sand at Blocks 29 and 30 and the northern portion of the tunnel and at least 10 feet into bedrock at Blocks 30, 31 and 32 and the southern portion of the tunnel. We recommend a WEAP analysis be performed to help determine the most appropriate hammer and pile size, and we should be provided with the opportunity to review the results.

Because of the potential for rubble and rock in the fill, pile locations should be predrilled or the first pile section should be vibrated in (H-piles only). Where obstructions are encountered that cannot be predrilled or vibrated through, the obstruction should be removed or piles relocated. Excavation with a backhoe or excavator may also be required to remove larger obstructions encountered in the fill. Piles should not be driven through obstructions. The predrill auger should have a diameter no greater than the minimum pile width to avoid reductions in lateral pile capacities. To reduce the amount of spoils, the predrilling should not extend more than a few feet into the Bay Mud. The cost of disposing of the fill and Bay Mud removed from the predrill holes should be considered when determining the foundation costs. Where steel piles are used, they can be vibrated through the fill. At a maximum, they should not be vibrated beyond the bottom of the Bay Mud; below the Bay Mud, the piles should be driven with an impact hammer. The effects of vibration on adjacent improvements may need to be monitored.

We recommend all of the indicator piles be monitored during driving with a pile driving analyzer (PDA). The PDA uses accelerometers to measure the propagation of compression waves through the pile during driving. When used in conjunction with the Case Pile Wave Analysis Program (CAPWAP), the PDA data can be used to:

- 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).

Half of the indicator piles at each building should be retapped at least four days after the initial drive. A CAPWAP analysis should be performed on a representative blow during the retap. A hammer capable of developing sufficient energy to mobilize the tip of the pile and/or develop the pile capacity should be used for the retaps.

The advantage of PDA testing is that it is relatively inexpensive to perform and several tests can be completed in one day. However, because the pile capacity information generated by this test is based on rapid rather than sustained loading, the results are somewhat approximate. In addition, recent experience indicates CAPWAP analyses can underestimate the ultimate capacity significantly if the pile does not displace sufficiently to mobilize the ultimate capacity, which is anticipated to be the case for long piles driven to bedrock.

Pile driving will cause vibrations on adjacent sites. These vibrations can cause settlement of the fill materials surrounding the site or could adversely affect nearby improvements, particularly freshly placed concrete. We recommend that the conditions of buildings and improvements within 150 feet of the site be photographed and surveyed to document existing conditions prior to the start of construction and that they be monitored periodically during construction.

9.2 Below-grade and Basement Walls

Below-grade walls should be designed to resist lateral pressures imposed by the adjacent soil and any surcharge loads. 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 an added seismic increment that should be added to active earth pressures (Lew et al. 2010). We used the procedures outlined in Lew et al. (2010) to compute the seismic active pressure. Figure 38 presents the static condition (at-rest) and seismic condition (active plus seismic pressure increment) for fill and Bay Mud assuming level backfill, for the improved and unimproved fill conditions. Figure 39 presents passive resistance for seismic conditions where the fill has not been improved and where the fill has been improved to a depth of 15 feet.

Where traffic is expected within 10 feet of the walls, a surcharge of 100 psf should be added to the top 10 feet of wall.

The lateral earth pressures given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend to the design groundwater level (Elevation 95 feet). We should check the manufacturer's specifications regarding the proposed prefabrication drainage panel material to verify it is appropriate for its intended use.

Another acceptable alternative is to backdrain the wall with 3/4- to 1-1/2-inch sized crushed rock at least one foot wide extending to the design groundwater table. Filter fabric should be placed between the gravel drain and the adjacent ground.

If the walls are not backdrained, the portion of the walls above the water table should be designed for the pressures given for fill below the water table.

9.3 Soil Improvement

As discussed in Section 8.2, the fill at the site should be improved to reduce the potential for liquefaction and lateral spreading during a major earthquake. As discussed, we judge the most economical method of improvement is Rapid Impact Compaction (RIC), although this method will likely only improve the fill to depths of about 15 feet below the ground surface. Where fill is currently thicker than about 15 feet, we recommend the site grade be lowered at least five feet to allow deeper improvement. The thickness of fill is varies considerably across the site; however, where explored, the fill in general is greater than

15 feet within Block 29 (at Borings B29-1 through B-29-8, and CPTs C29-2 through C29-4), the northern half of Block 30 (at Borings B30-2 through B30-5, 360) and the western half of Block 31 (at Borings B31-1 through B31-3, B31-5 and B31-6, and CPT C31-1).

The soil improvement should be performed by a specialty contractor experienced in this type of soil improvement. During RIC, or a similar type of dynamic compaction, we should monitor:

- average induced ground settlements
- crater settlement with adjacent ground heave measurements
- ground vibrations during tamping.

Once a soil technique has been chosen, its effectiveness should be verified by performing two test sections at each block, or a total of six sections throughout the site if the test sections are improved in one operation. We should choose the locations of the test sections. The test sections should be on the order of at least 30 by 30 feet in plan dimension. Where the fill is thick, the grade should be lowered in the area of the test section to confirm that deeper improvement can be accomplished. We should evaluate the results of CPTs before and after the dynamic compaction test sections to document the improvement of the fill material. We recommend post-RIC CPTs be performed a minimum of two weeks after the test sections have been completed. In addition, we recommend time be included in the construction schedule for the option to perform a second RIC pass in the case the first post-RIC CPTs do not show adequate improvement. Improvement of the fill should be verified at the test sections prior to continuing improvement throughout the site. Improvement of the site should be performed prior to installation of production piles.

The improved fill (where classified as sand or silty sand) should have minimum and average SPT blow counts $[(N1)_{60,CS}$[18], over three consecutive SPTs, of at least 20 and 25 blows per foot, respectively, corresponding to Cone Penetration Test (CPT) minimum and average tip resistances $[(q_{cIN})_{cs}]$[19], over an interval of three feet, of at least 100 and 125 tons per square foot (tsf), respectively. The acceptance criteria may need to be reevaluated depending on the soil types encountered. If improvement methods

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18 The Standard Penetration Test (SPT) blow count (N-value) is obtained by recording the number of blows required to drive an SPT sampler the final 12 inches of an 18-inch drive using a 140-pound safety hammer with a fall of 30 inches. (N 1)60,CS is an N-value that has been normalized to an overburden pressure of one atmosphere and corrected to account for the effects of fines content.

19 The $(ci_{ciN})_{cs}$ is tip resistance that has been normalized to an overburden pressure of 1 atmosphere and corrected to account for the effects of fines content.

are used that will cause heavy or excessive vibrations, setbacks may be required and/or monitoring of nearby improvements should be performed. Vibrations should be measured during the construction of the test section and may need to be monitored during production.

Prior to performing dynamic compaction, the existing surface improvements (i.e. asphalt pavement) should be demolished and removed from the site. Demolished asphalt pavement grindings may be stockpiled for reuse at the site. If thick aggregate base is present, or very hard near-surface soils are encountered, the layer should be ripped prior to performing the dynamic compaction, as hard layers may preclude the vibrations from penetrating and densifying the underlying loose sandy fill.

During improvement, the ground surface will settle. This settlement will not be uniform across the improved area; it will result in an uneven ground surface. At a nearby site, up to two feet of ground settlement was observed in the craters after RIC was performed. Additional fill will need to be brought on site to raise site grades following RIC. Settlements may also occur beyond the immediate work area. Adjacent structures within 150 feet of the project site should be monitored during production work.

9.4 Floor Slabs

Where new fill will be placed, we estimate consolidation settlements to vary from zero to six inches within the footprints of the proposed buildings. We recommend the ground floor slabs of the buildings be designed to span between pile caps and/or grade beams, and the fill should not be relied upon for support. Entrances to the building should be designed to transition from areas of structural support to areas of no support where up to six inches of static settlement and an additional six inches of seismically-induced settlement could occur. For below-grade areas, floor slabs will be below groundwater and should be designed to resist hydrostatic uplift based on a design groundwater elevation of 95 feet. We recommend using a factor of safety of 2 for permanent hydrostatic uplift.

Initially, at-grade slabs will be in contact with the ground. Moisture is likely to condense on the underside of the ground floor slabs, even though it will be above the design groundwater table. A moisture barrier should be considered if movement of water vapor through the slab would be detrimental to its intended use. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 16.

TABLE 16
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 — 100
3/4 inch	30 — 100
1/2 inch	5 — 25
3/8 inch	0 — 6
Sand	
No. 4	100
No. 200	0 — 5

The sand overlying the membrane should be dry at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.45 (see Appendix D). If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

Floor slabs for below-grade areas should be waterproofed.

The existing fill is generally corrosive. Floor slabs should be designed to mitigate the effects of corrosion. For more detail, see the recommendations by JDH Corrosion Consultants in Appendix D.

9.5 Seismic Design

In the development of site-specific response spectra, we assumed that the potentially liquefiable soil in the upper 15 feet of fill will be mitigated. Furthermore, where the potentially liquefiable soil extends below a depth of 15 feet, we recommend that site grades be overexcavated approximately five feet prior to performing site improvements to extend the improvement deeper. Because any remaining potentially liquefiable layers will be relatively thin and discontinuous, we conclude Blocks 29, 30 and 31 should be classified as Site Class S_E and Block 32 as Site Class S_D.

For seismic design in accordance with the provisions of 2010 SFBC, we recommend the following:

TABLE 17
Mapped Values per 2010 SFBC

Site Coefficients and Modification Factors	Blocks 29, 30 and 31	Block 32
Site Class	S_E	S_D
F_a	0.9	1.0
F_v	2.4	1.5
S_S	1.500	1.500
S_1	0.611	0.611
S_{MS}	1.350	1.500
S_{M1}	1.466	0.917
S_{DS}	0.900	1.000
S_{D1}	0.978	0.611

As discussed in Section 7.1, we performed PSHA, deterministic analysis and ground response analysis to develop recommended horizontal ground surface spectra for Blocks 29, 30, 31 and 32. The recommended horizontal response spectra are presented on Figures 9, 10, 11 and 12, respectively. Details of our analyses are presented in Appendix E.

9.6 Shoring

The proposed excavation for the Building 30, service areas in Buildings 29, 31 and 32 and the service tunnel should be shored. Soil-cement mixed walls and secant pile walls are acceptable methods of lateral support for adjacent improvement and properties. Typical lateral earth pressures for these shoring systems are presented on Figure 38.

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 selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The shoring system should be designed by a licensed structural engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring engineer 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.

Soil-cement mixed walls or secant walls with tiebacks or internal bracing should be designed using the lateral earth pressures presented on Figure 41. The pressure on the shoring should be computed by adding the water pressure to the soil pressure as shown on Figure 41. In computing the passive pressure, we have assumed the groundwater level within the site will be lowered to a depth of at the bottom of the excavation while the groundwater level outside the shoring remains close to its natural level (Elevation 95). The passive resistance and the active pressure are shown through stiff clay and sands below the Bay Mud. Penetration depth below the bottom of excavation should be determined based on the requirements to achieve lateral stability and resist downward loading of the tiebacks and/or internal bracing, if they are used.

9.6.1 Tiebacks

Temporary tiebacks may be used to restrain the shoring. The vertical load from the temporary tiebacks should be accounted for in the design of the vertical elements. Design criteria for tiebacks are presented on Figure 38.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks with bar and strand tendons should have a minimum unbonded length of 10 and 15 feet, respectively. All tiebacks should have a minimum bonded length of 15 feet and be spaced at least six times the grouted diameter of the bonded zone or four feet, whichever is less. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, post grouting, and workmanship. The use of solid-flight augers to install tiebacks in sand and the fill can result in loss of soil and settlement of structures or the ground surface located above the tiebacks. Therefore, solid flight augers or Titan type anchors should not be used for tieback installation. We recommend a smooth cased tieback installation method (such as a Klemm type rig) be used. For estimating purposes, we recommend using an allowable skin friction value of 1,000 psf for post-grouted tiebacks in fill, as shown on Figure 41.

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 our observation. Replacement tiebacks should be installed for tiebacks that fail the load tests.

9.6.2 Tieback Testing

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

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

9.6.2.1 Performance Tests

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

In the performance test, the load applied to the tieback and its movement is measured during several cycles of incremental loading and unloading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 1, 2, 3, 4, 5, 6 and 10 minutes. If the difference between the 1- and 10-minute 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.

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

9.6.2.2 Proof Tests

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

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

9.6.2.3 Acceptance Criteria

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

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

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

9.6.3 Penetration Depth of Shoring

The shoring designer should evaluate the required penetration depth of the shoring walls. The shoring walls should have sufficient axial capacity to support the vertical load component of the tiebacks and any other vertical load acting on the walls. Above the excavation level, the axial capacity of the shoring will be provided by the friction along the back of the walls; below the excavation level, by friction along both sides of the walls. To compute the axial capacity of the shoring, we recommend neglecting the friction in the fill, and using an allowable friction of 250 psf and 1,000 psf on both sides of shoring wall in the Bay Mud and the soil below the Bay Mud, respectively.

9.7 Dewatering

During excavation, the water table should be drawn down to three feet below the bottom of the excavation or to the surface of the Bay Mud where excavations extend to this layer. 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. If a secant pile wall or another type of cutoff wall is installed and extends into the Bay Mud layer sufficiently to cutoff groundwater, seepage through the shoring should be controllable. Water could be controlled inside the excavation using sumps and pumps as needed. The shoring designer should also determine the minimum embedment of the shoring needed to provide groundwater cutoff and prevent base heave of the excavation.

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. Design of the dewatering system should be the responsibility of an experienced dewatering contractor. Dewatering the site could result in subsidence of the surrounding area due to increases in effective stresses in the soil. Therefore, adjacent improvements should be monitored for vertical movement and groundwater levels outside the excavation should be monitored 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.

There is currently a fee imposed by the City and County of San Francisco Department of Public Works for discharge of construction-generated water into the City and County of San Francisco Sewer System.

9.8 Site Preparation

Grading operations should commence after demolition and removal of the existing pavements, foundation slabs, and underground utilities within the development area. Following demolition, all areas to receive improvements should be stripped of vegetation and organic topsoil. The pavement material, including asphalt, may be segregated from organic topsoil and used as compacted fill, provided it meets the fill requirements presented in a subsequent paragraph of this section and is acceptable from an environmental standpoint. The stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the architect; organic topsoil should not be used as compacted fill.

All buildings on site were demolished in 2003; currently, the site is vacant. However, if any foundations are encountered, the following recommendations should be followed.

All existing foundations should be removed to the bottom of the new pile caps, structural slabs, and utilities within the building footprints. All pile caps and footings should be completely removed beneath new shallow foundations, slabs-on-grade, pavements, sidewalks, and landscaped areas. In general, single piles beneath these elements should be removed to a depth of at least four feet below final soil subgrade and pile groups should be removed to a depth of at least eight feet below the final soil subgrade. Single piles should be removed to a depth of at least four feet below new utilities and pile groups should be removed at least eight feet below new utilities, or to the Bay Mud, whichever is shallower. The depth of pile removal may be vary based on site-specific conditions.

For the abandonment of subsurface utilities, we recommend the following:

- Where utility is greater than six inches in diameter, is within the building footprint, and is within the depth of new pile caps, the pipe should be removed and the trench backfilled and compacted with soil.
- Where utility is greater than six inches in diameter, is within the building footprint, and is below the depth of new pilecaps, ends of the line should be capped with concrete to prevent entrance of water. The length of the cap should not be less than five feet and the concrete mix should have minimum shrinkage.
- Where utility is outside of building footprints and is within three feet of soil subgrade, the pipe should be removed and the trench backfilled and compacted with soil.
- Where utility is greater than six inches in diameter, outside of building footprint, and is deeper than three feet below soil subgrade, the ends of the line should be capped with concrete to prevent entrance of water. The length of the cap should not be less than five feet and the concrete mix should have minimum shrinkage.
- All pipes less than six inches in diameter, is outside of building footprints and is deeper than three feet below soil subgrade may be left in place.

For the installation of new subsurface utilities (mains and laterals) and vaults, existing foundations should be removed within the width of the utility trench to a depth of at least two feet below new utilities or to the bottom of the trench excavation, whichever is greater. For new tree wells, existing foundations should be removed to the bottom of the tree well excavation. Below new roadways and sidewalks, existing foundations should be removed to a depth of at least four feet below subgrade. Existing pile foundation locations should be checked for conflicts with future light poles foundations. The geotechnical engineer may vary the depth of pile removal based upon site specific conditions.

Where pile foundations are removed, the excavation and/or voids should be backfilled with engineered fill. At locations where there is not adequate space for compaction equipment, we recommend backfilling the lower portion of the hole with controlled density fill (CDF) or lean concrete. Overexcavations that extend below water should be backfilled with crushed rock to at least six inches above water. The crushed rock should be covered entirely with filter fabric, above which suitable backfill maybe placed and compacted.

9.9 Earthwork

The surface exposed by stripping and/or excavation should be:

- scarified to a minimum depth of six inches
- moisture conditioned to near optimum
- compacted to at least 90 percent relative compaction[20].

If soft areas are encountered, the soft material should be removed and replaced with either lean concrete or engineered fill. Excavations made to remove utilities, old foundations, or other improvements should be filled with lean concrete or properly compacted fill. Where the bottom of an overexcavation is near or below the water table, it should be covered with a reinforcing geotextile or geogrid (Mirafi 500x, for example) overlain by 1/2- to 3/4-inch crushed rock to at least six inches above the water table to provide a more stable base for backfill. A layer of filter fabric, such as Mirafi 140NC, should be placed between the crushed rock and compacted fill to reduce the potential for fines infiltrating into the voids between the crushed rock particles. If excavations extend into Bay Mud, a geogrid should be placed over the base of the excavation to prevent the rock from pushing into the clay and then overlain by at least 12 inches of crushed rock. Once geogrid and a sufficient thickness of rock has been placed to create a stable working surface, fill can be placed and compacted according to our recommendations.

All fill ("engineered fill") should be placed in horizontal layers not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. All fill material should be free of organic debris and rocks or lumps larger than four inches in greatest dimension and have a low expansion potential, defined by a liquid limit (LL) less than 40 and a plasticity index (PI) lower than 12. From a geotechnical standpoint, most on-site soil free of organic matter and rocks or lumps larger than four inches in greatest dimension should be suitable for use as fill or backfill provided it is properly moisture conditioned. Bay Mud is not suitable material for use as fill. Samples of all imported fill should be submitted to the geotechnical engineer for testing at least 72 hours before delivery to the site.

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[20] 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.

Backfill behind retaining and below-grade walls and around grade beams and pile caps may consist of either on-site material or approved imported fill. The backfill should be placed in layers of eight-inch maximum thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. Retaining walls should be braced or hand compaction equipment used where compaction of backfill by heavy equipment could cause unwanted surcharges on walls or foundations (as determined by the structural engineer).

We recommend exterior concrete slabs, pavers, and pavements that are supported on grade be underlain by at least two feet of engineered fill. In cut areas, or where less than two feet of new fill will be placed, existing grade should be overexcavated at least 18 inches below existing or final soil subgrade, whichever is shallower. The exposed excavation surface should then be scarified to a depth of at least six inches, moisture conditioned, and recompacted to at least 90 percent relative compaction. New fill or the soil removed by excavation should be placed in eight-inch-thick loose lifts and compacted to at least 90 percent relative compaction. The final six inches beneath exterior slabs and pavements should be rolled to expose a firm non-yielding surface, compacted to at least 95 percent relative compaction.

9.10 Utilities

Utilities should be designed to accommodate the computed up to 12 inches of differential settlement (static and seismic settlements assuming RIC is performed). Hangers and flexible connections may be used. The hangers should be corrosion resistant. Where utilities are hung, they should be backfilled with pea gravel to allow the ground to settle and reduce the soil loading on the pipes. However, because of the flowable nature of pea gravel, it cannot be relied upon to provide lateral load resistance against pile caps or grade beams; therefore, where passive resistance against an adjacent pile cap or other structural element is being relied upon, all trenches within five feet of pile caps should be backfilled with properly compacted soil and the hanger spacing should account for the soil loading. Flexible connections, which allow for up to 12 inches of differential movement (where utilities enter the building), should be used. Alternatively, if flexible connection cannot tolerate the estimated movements they will need to be replaced periodically.

The existing fill is generally corrosive. Corrosion control measures, such as dielectric coated steel and cathodic protection, should be used to protect utility lines. Alternatively, nonmetallic pipes such as PVC may be used (if approved by the City and County of San Francisco) per the recommendations presented in the corrosion study. For more detail, see the recommendations by JDH Corrosion Consultants in Appendix D. A corrosion consultant should be retained during utility design.

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

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in accordance with the recommendations for engineered fill in Section 9.9.

9.11 Site Drainage

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly above slopes or adjacent to building foundations, roadways, pavements, or slabs. Surface runoff should be directed away from slopes and foundations and collected in lined ditches or drainage swales. The water collected should be directed to a storm drain or paved roadway. Discharge from the roof gutter and downspout systems should be included in the collection system and not allowed to infiltrate the subsurface near the structures or in the vicinity of slopes.

We understand the development will be designed to meet the stormwater retention requirements of the City and County of San Francisco. If project will include the installation of a stormwater infiltration system in order to comply with the San Francisco Public Utility Commission (SFPUC) Urban Watershed Management Program Stormwater Requirements, we recommend infiltration tests be performed in the surficial soils. Infiltration tests are currently not within the scope of work of this investigation.

10.0 ADDITIONAL GEOTECHNICAL SERVICES

During final design we should be retained to consult with the design team as geotechnical questions arise. 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 site preparation, excavation, shoring, compaction of fill and backfill, RIC test section and production, and installation of the building foundations. These observations will allow us to compare the actual with the anticipated soil and bedrock conditions and to check that the contractors' work conforms to the geotechnical aspects of the plans and specifications.

11.0 LIMITATIONS

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

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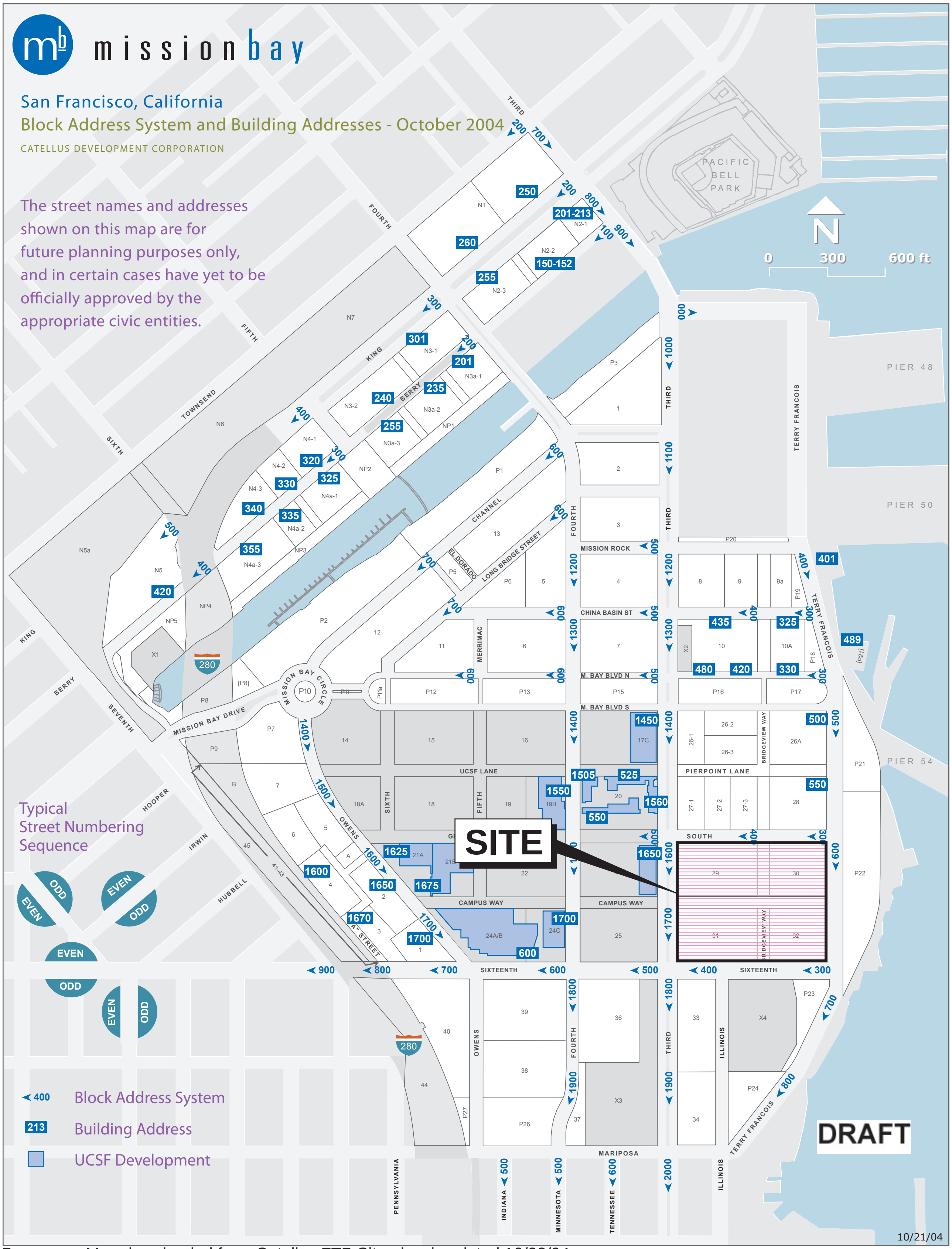
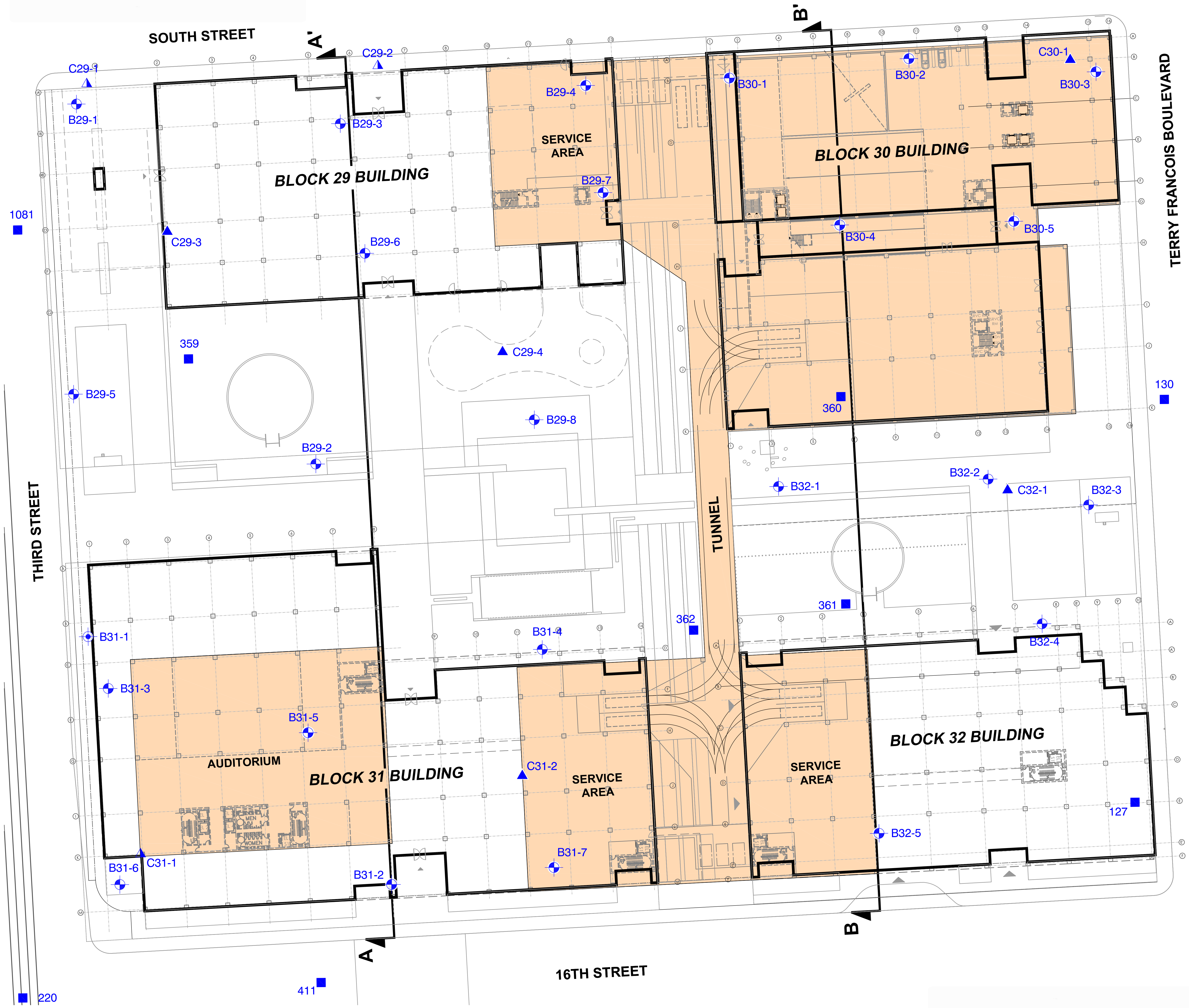
FIGURES



Basemap: Map downloaded from Catellus FTP Site, drawing dated 10/22/04.

BLOCKS 29-32 **MISSION BAY** San Francisco, California	**SITE LOCATION MAP**
Treadwell&Rollo A LANGAN COMPANY	Date 10/10/11 | Project No. 750603902 | Figure 1



EXPLANATION

B29-3 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C29-3 — Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

B31-1 — Approximate location of boring by Treadwell & Rollo for other developer during a previous investigation

C31-1 — Approximate location of cone penetration test by Treadwell & Rollo for other developer during a previous investigation

361 — Approximate locations of boring by others (data base designation)

Below grade areas

A A' — Idealized subsurface profile

Notes:

1. Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.
2. Below grade areas from "111028_site_basement_plan" emailed from YamaMar design on 28 October 2011.

DRAFT

0 60 Feet

Approximate Scale

BLOCKS 29-32
MISSION BAY
San Francisco, California

SITE PLAN

Date 11/22/11	Project No. 750603902	Figure 2

Treadwell&Rollo
A LANGAN COMPANY

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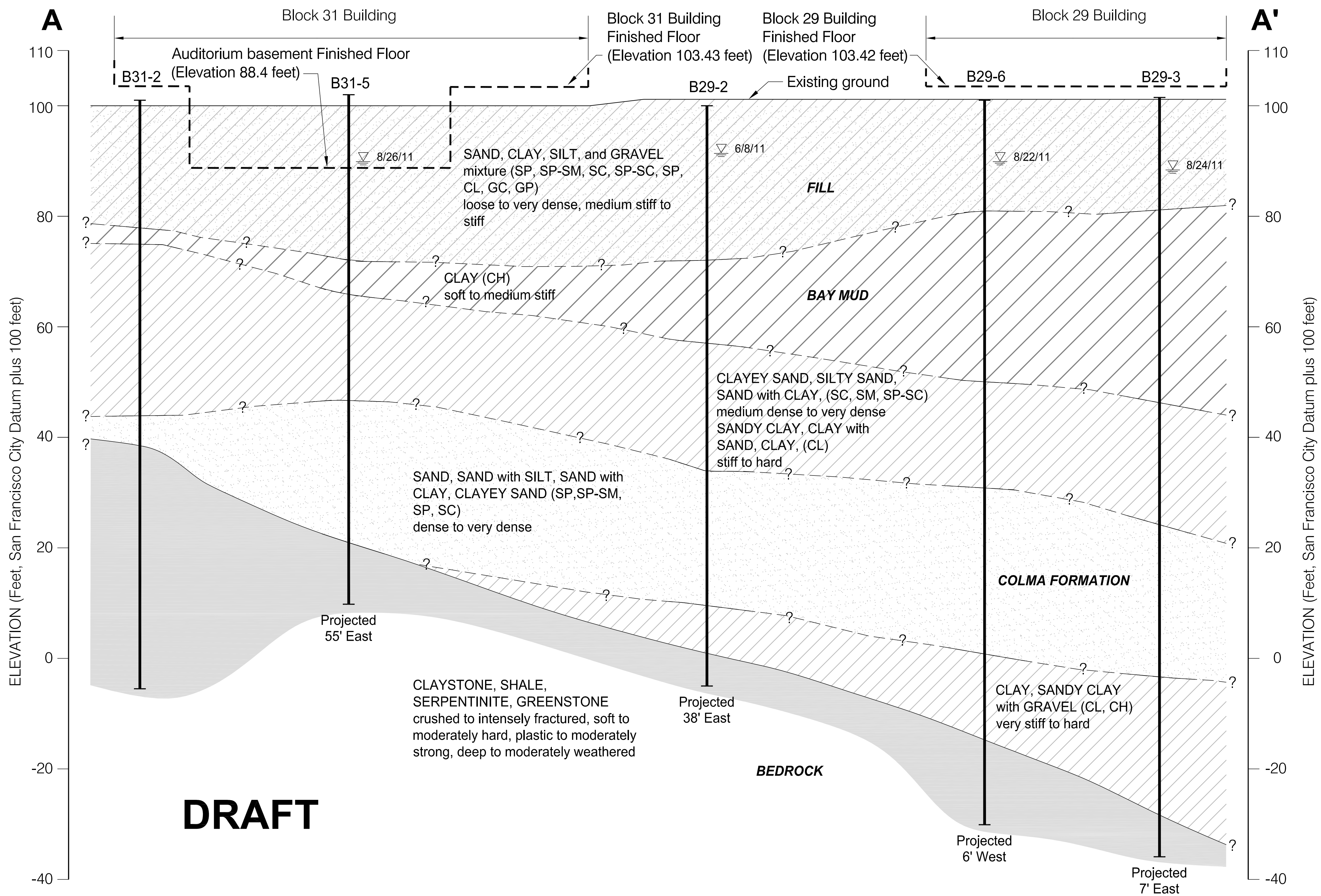


Notes:
1. See Figure 2, Site Plan for location of subsurface profile.
2. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
3. Elevations are based on San Francisco City Datum plus 100 feet. Existing ground surface based on Topographic Survey "X-site-survey.dwg." by Sherwood Deisign Engineers, emailed to Treadwell & Rollo on 10/14/11.
4. Top of borings may not line up with ground surface because borings are projected.
5. Bedrock surface based on Top of Bedrock contour presented on Figure 6.

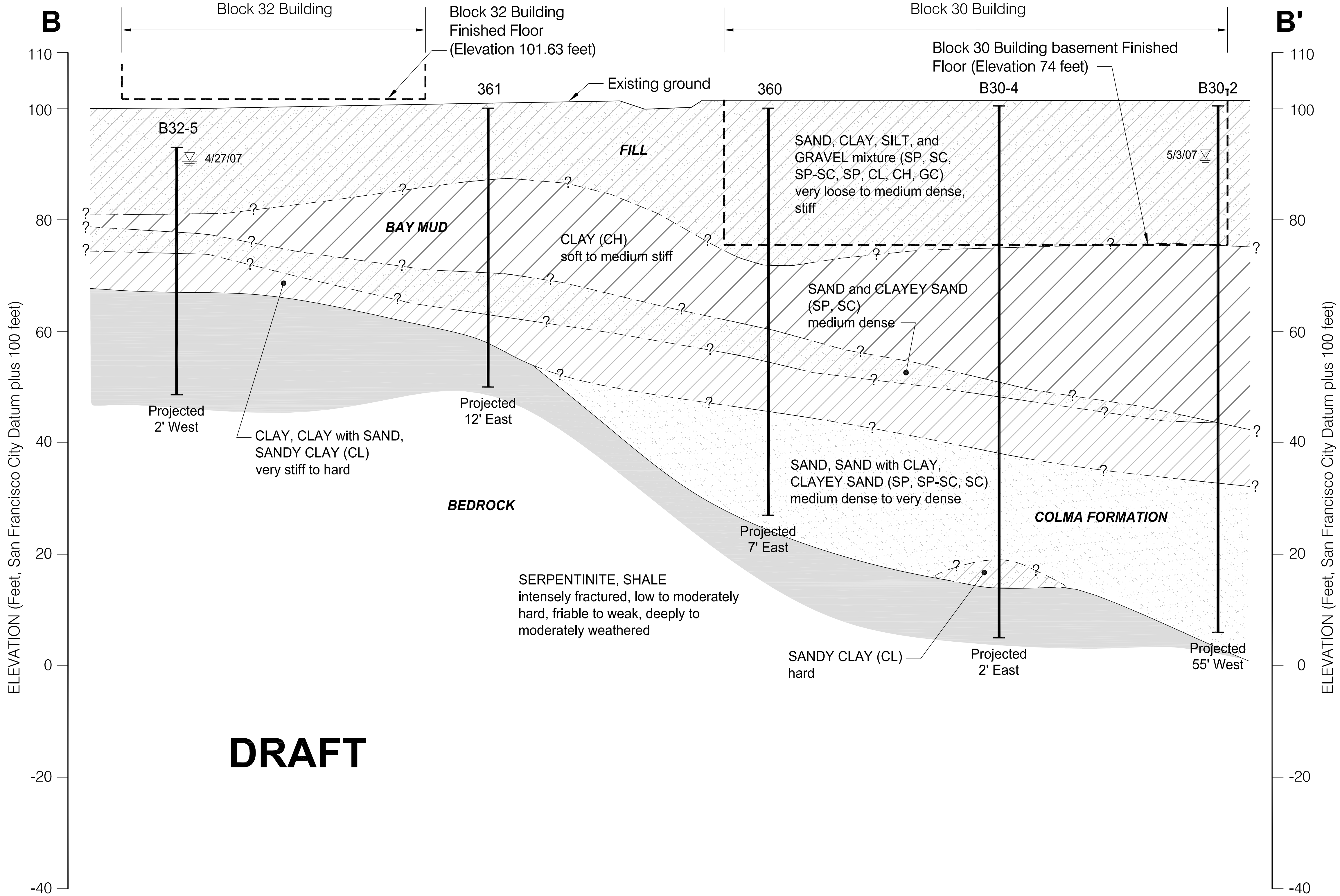
20
0
0 60
SCALE IN FEET

BLOCK 29-32
MISSION BAY
San Francisco, California

IDEALIZED SUBSURFACE PROFILE
A-A'

Date 12/20/11	Project No. 750603902	Figure 3

Treadwell&Rollo
A LANGAN COMPANY



Notes:
1. See Figure 2, Site Plan for location of subsurface profile.
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.
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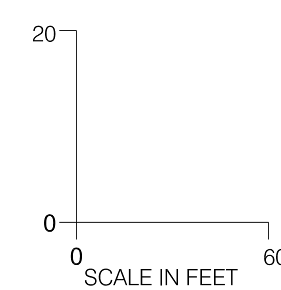
4. Top of borings may not line up with ground surface because borings are projected.
5. Bedrock surface based on Top of Bedrock contour presented on Figure 6.



BLOCK 29-32
MISSION BAY
San Francisco, California

IDEALIZED SUBSURFACE PROFILE
B-B'

Date 12/20/11	Project No. 750603902	Figure 4

Treadwell&Rollo
A LANGAN COMPANY

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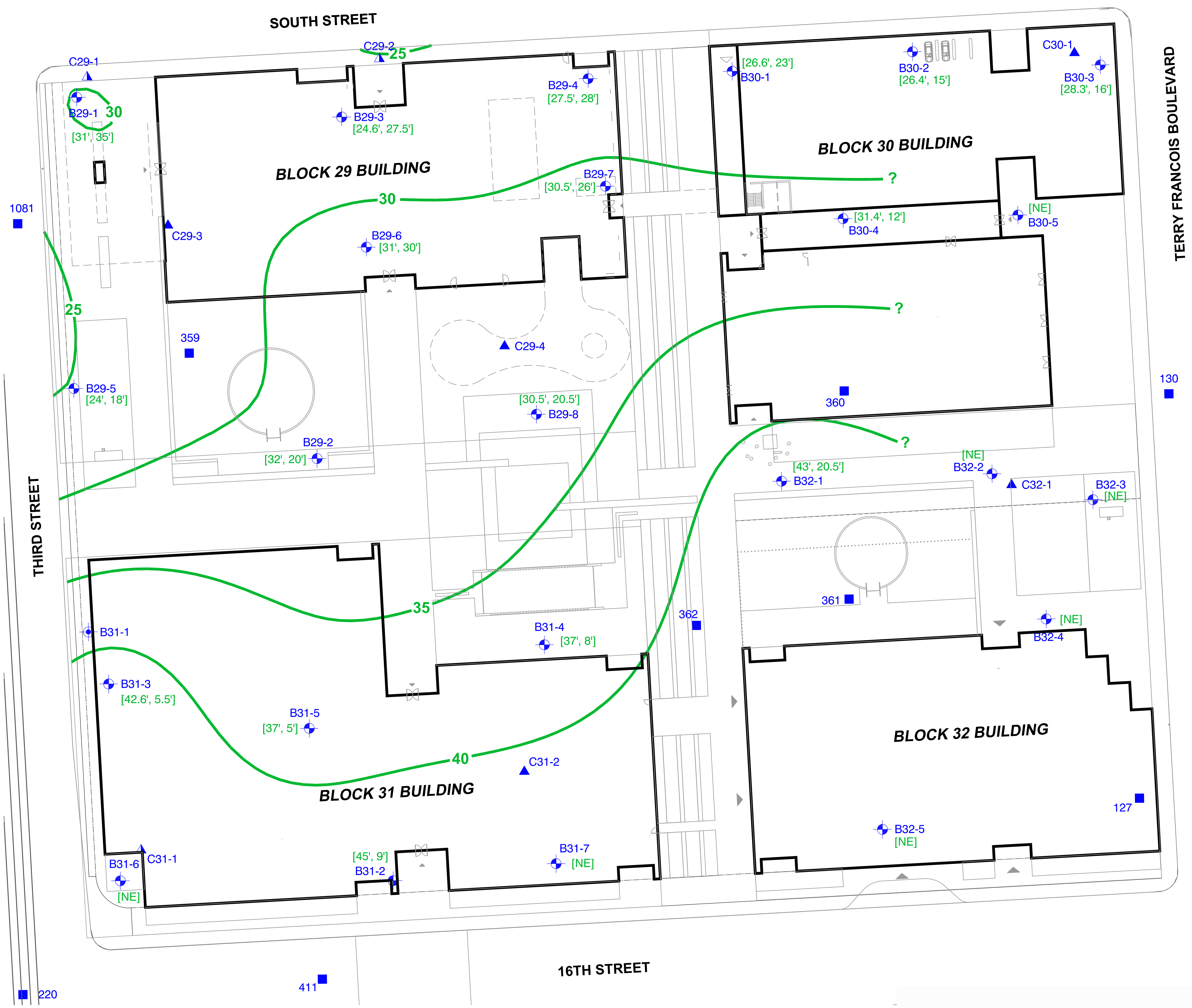

EXPLANATION

B29-3 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C29-3 — Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

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C31-1 — Approximate location of cone penetration test by Treadwell & Rollo for other developer during previous investigation

361 — Approximate locations of boring by others (data base designation)

Top of very dense sand contour (feet, SFCD+100) (see Note 1) Based on interpretation between borings

[24.6', 27.5'] — Approximate top of very dense sand layer elevation (feet, SFCD+100 feet) (see Notes 1 and 2) Estimated thickness of very dense sand (feet) (see Notes 1 and 2)

[NE] — Not Encountered

Notes:
1. Contours and thickness are for the very dense portion of the Colma Formation Sand (where SPT N-Value is greater than 50 blows per foot).
2. For CPT and borings that did not extend through the Colma Formation or did not record SPT-N values on the boring logs, the estimated top of very dense sand elevation and thickness is left blank.

DRAFT

0 60 Feet
Approximate Scale

BLOCKS 29-32
MISSION BAY
San Francisco, California

TOP OF VERY DENSE SAND CONTOURS

Date 11/23/11 | Project No. 750603902 | Figure 5

Treadwell&Rollo
A LANGAN COMPANY

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

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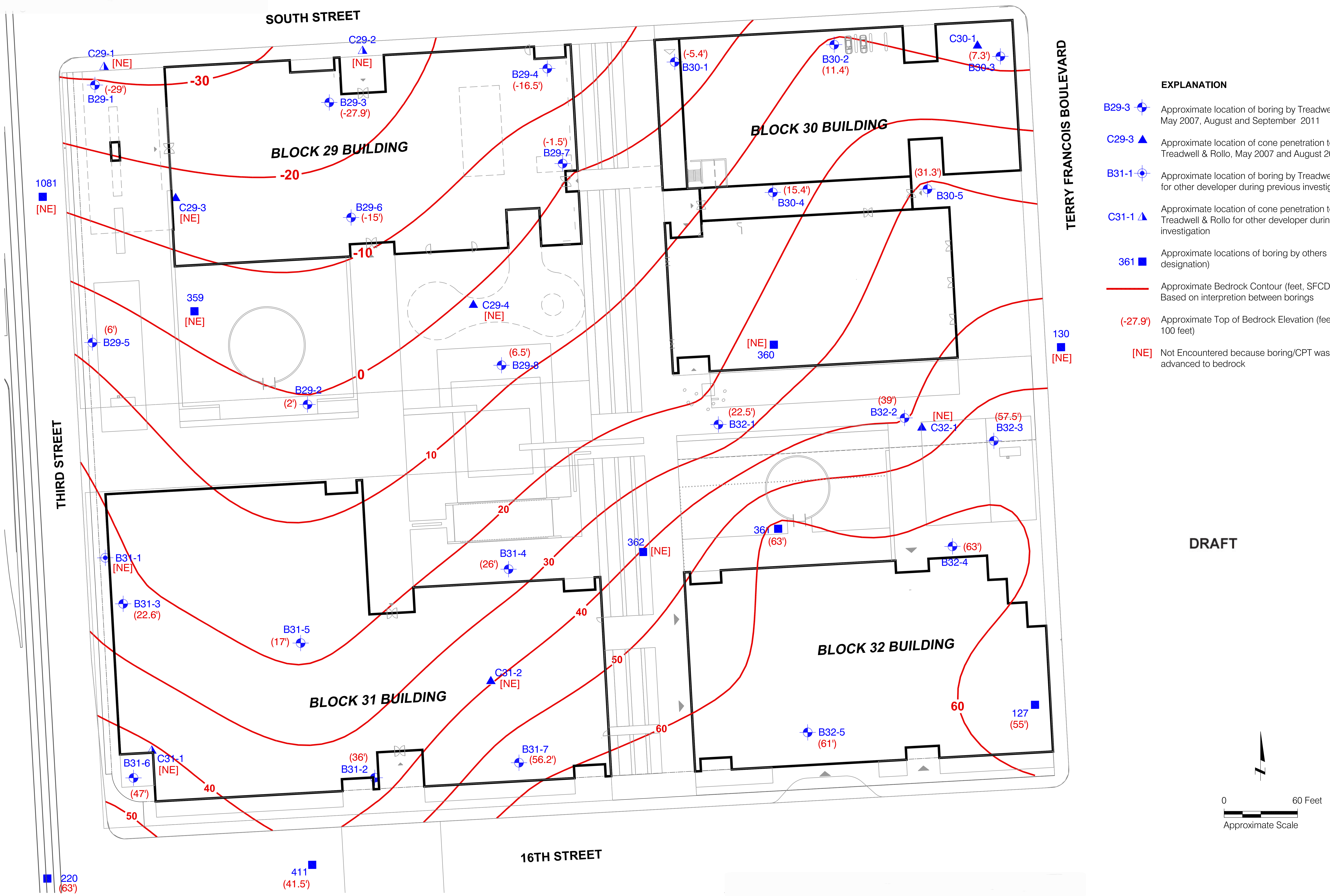

EXPLANATION

- B29-3 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011
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- C31-1 — Approximate location of cone penetration test by Treadwell & Rollo for other developer during previous investigation
- 361 — Approximate locations of boring by others (data base designation)
- Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings
- (-27.9') — Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet)
- [NE] — Not Encountered because boring/CPT was not advanced to bedrock

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

BLOCKS 29-32
MISSION BAY
San Francisco, California

TOP OF BEDROCK CONTOURS

Date 11/21/11	Project No. 750603902	Figure 6

Treadwell&Rollo
A LANGAN COMPANY

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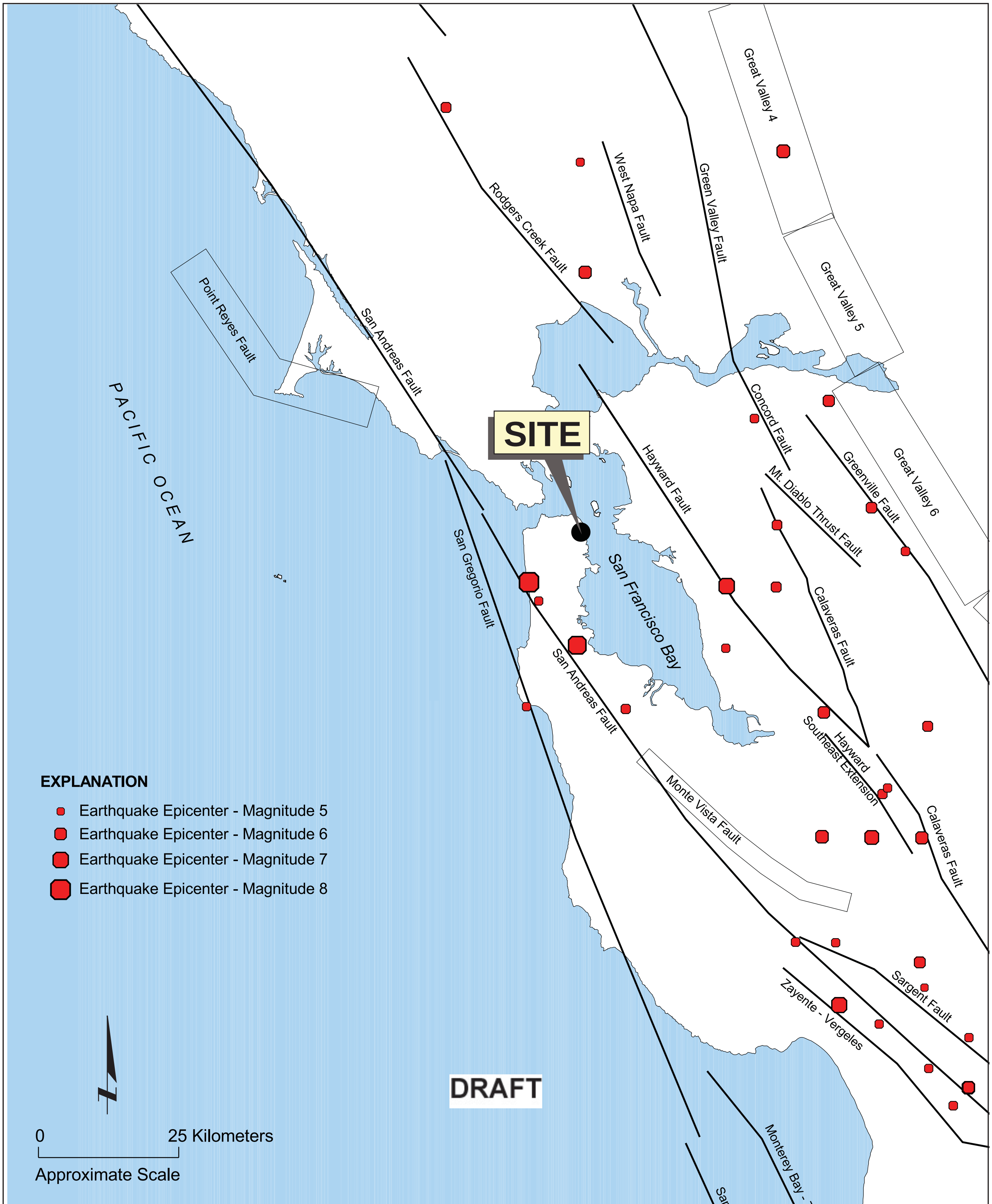

NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

BLOCKS 29-32
MISSION BAY
San Francisco, California

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

Date 11/10/11 | Project No. 750603902 | 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.

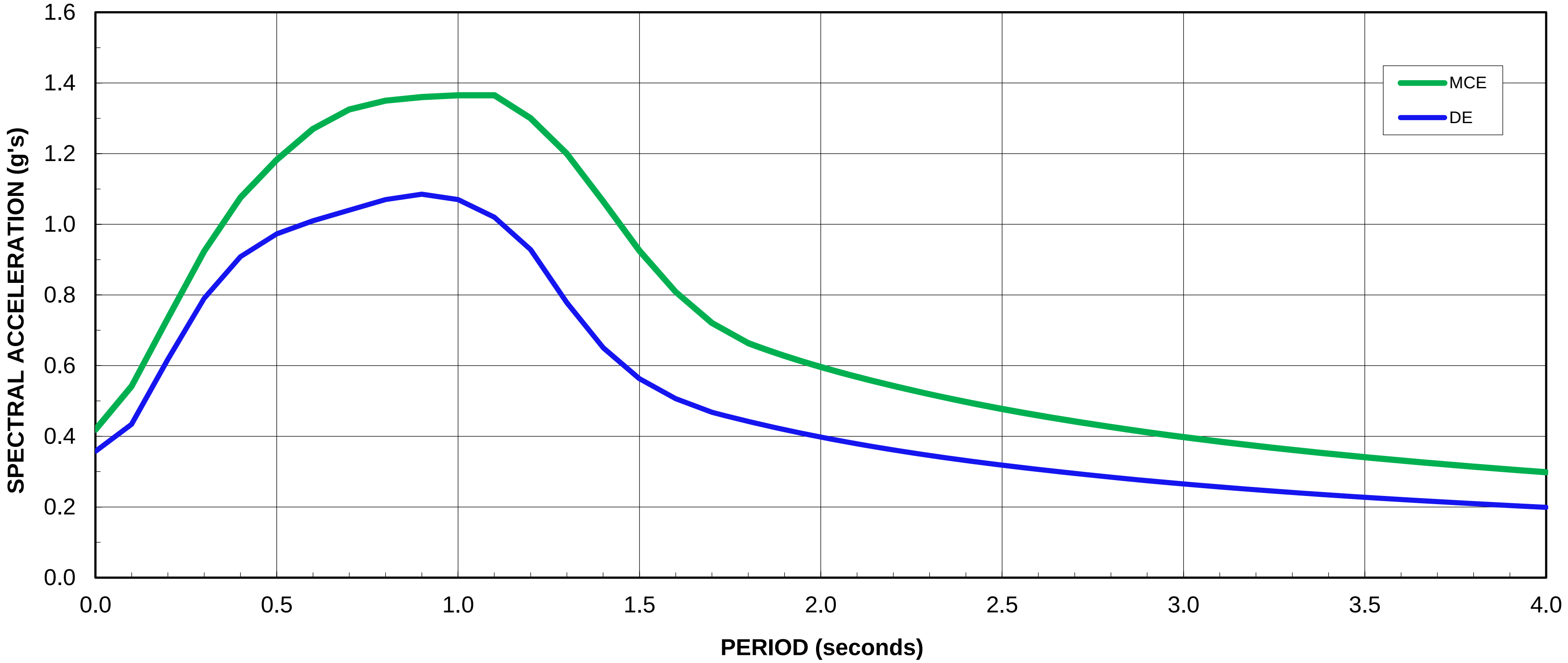
X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water 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.

DRAFT

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.

BLOCKS 29-32 **MISSION BAY** San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**
Treadwell&Rollo A LANGAN COMPANY	Date 11/10/11 | Project No. 750603902 | Figure 8



DRAFT

Damping Ratio = 5%

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29 THROUGH 32
MISSION BAY
San Francisco, California

RECOMMENDED SPECTRA
BLOCK 29

Date 11/23/11	Project No. 750603902	Figure 9

Treadwell&Rollo
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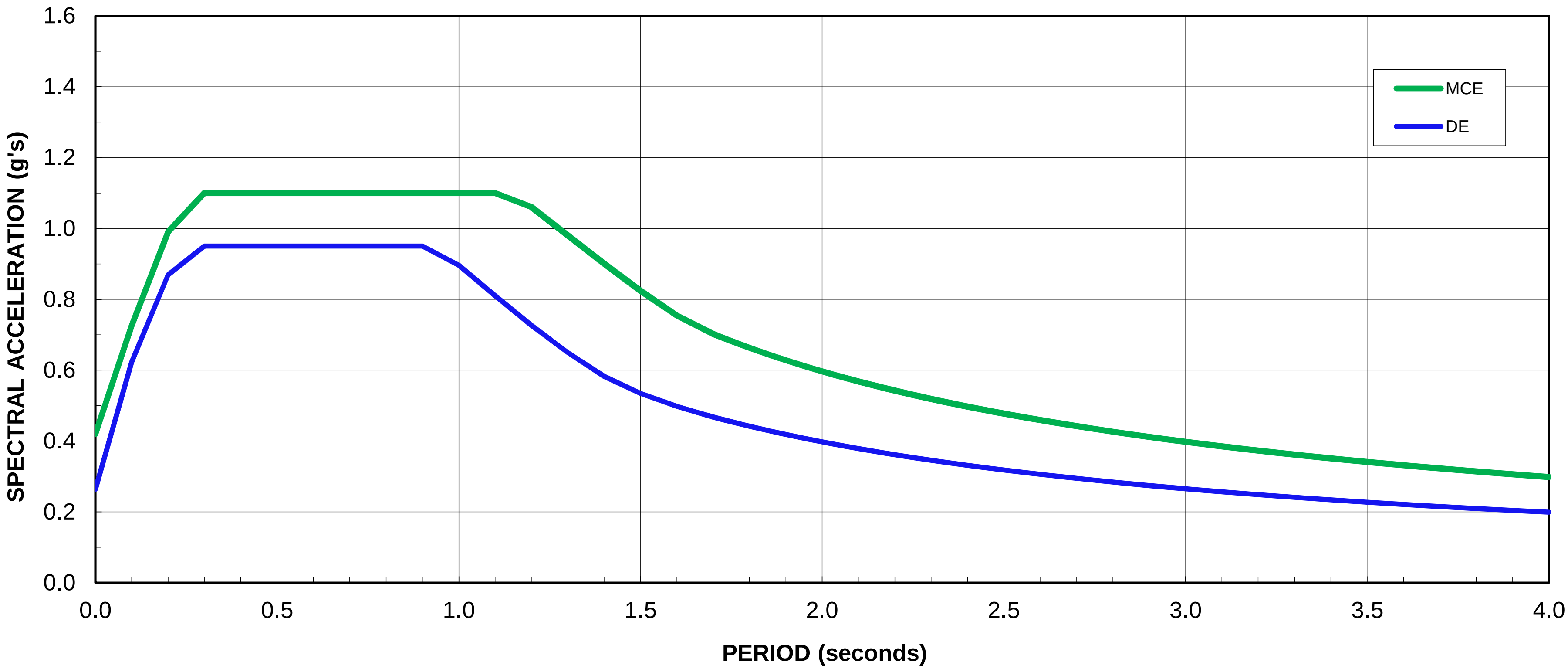

DRAFT

Damping Ratio = 5%

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29 THROUGH 32 MISSION BAY San Francisco, California		
RECOMMENDED SPECTRA BLOCK 30		
Date 12/13/11	Project No. 750603902	Figure 10

Treadwell&Rollo
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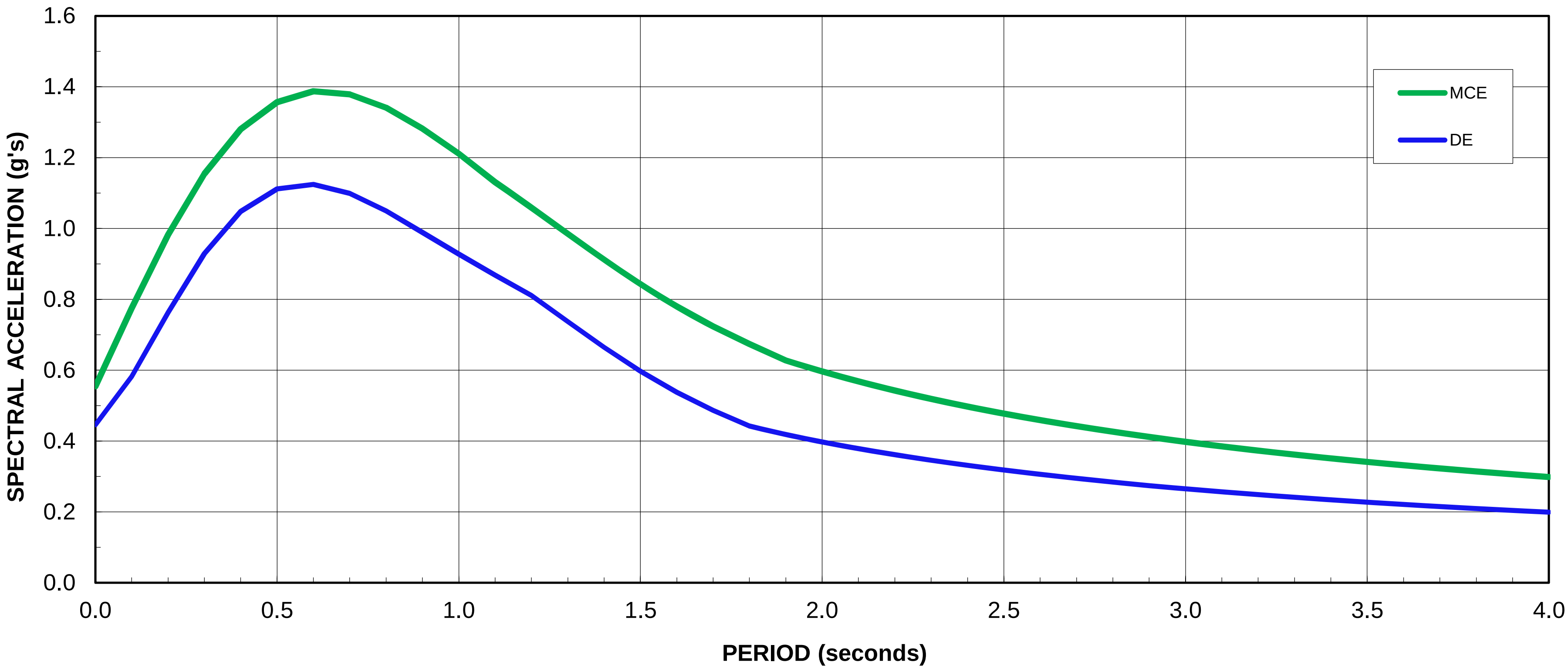


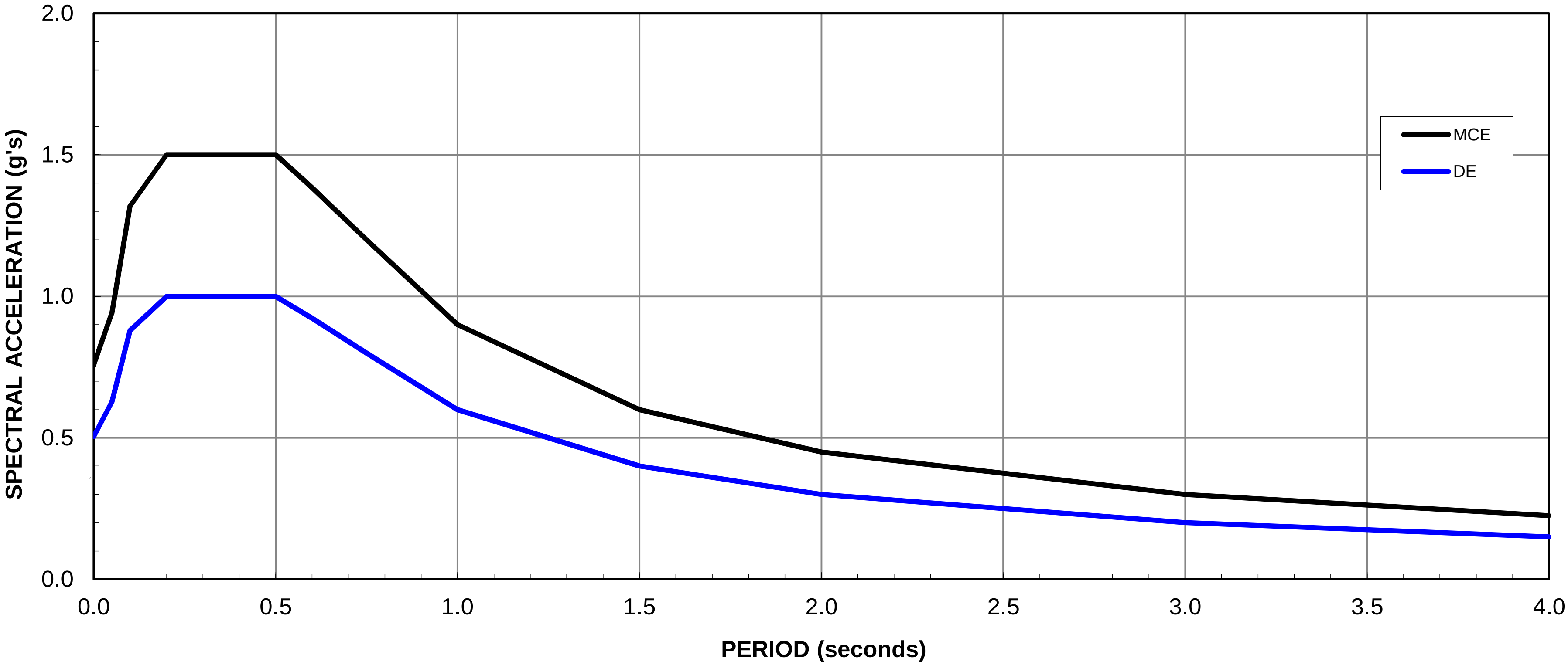
DRAFT

Damping Ratio = 5%

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29 THROUGH 32 **MISSION BAY** San Francisco, California		
RECOMMENDED SPECTRA **BLOCK 31**		
Date 12/13/11	Project No. 750603902	Figure 11

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

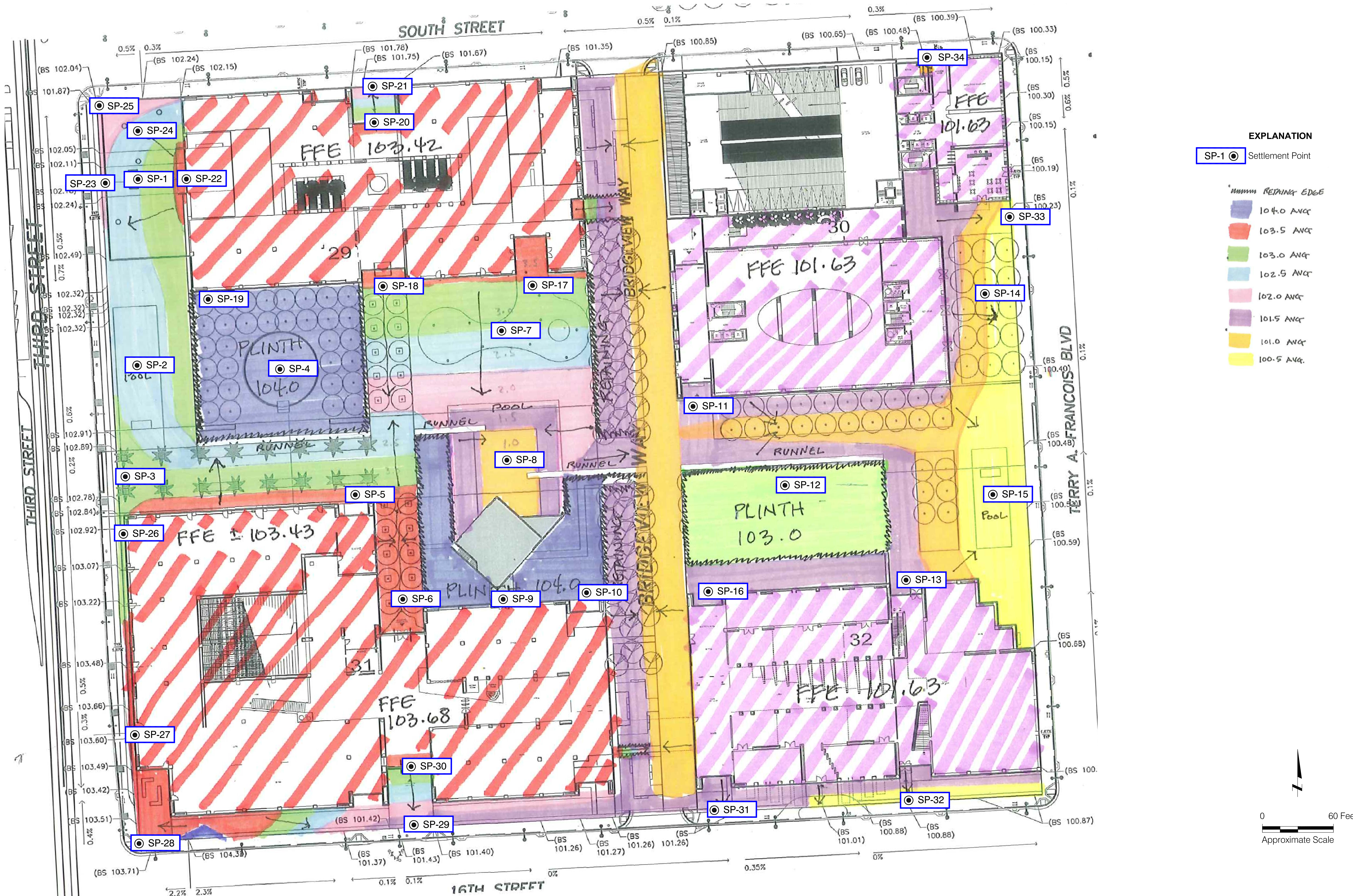
Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29-32
MISSION BAY
San Francisco, California

RECOMMENDED SPECTRA
BLOCK 32

Date 1122/11	Project No. 750603902	Figure 12

Treadwell&Rollo
A LANGAN COMPANY



Basemap from schematic titled "Proposed Preliminary Elevations Study", dated 10-17-11 from Tom Leader Studio.

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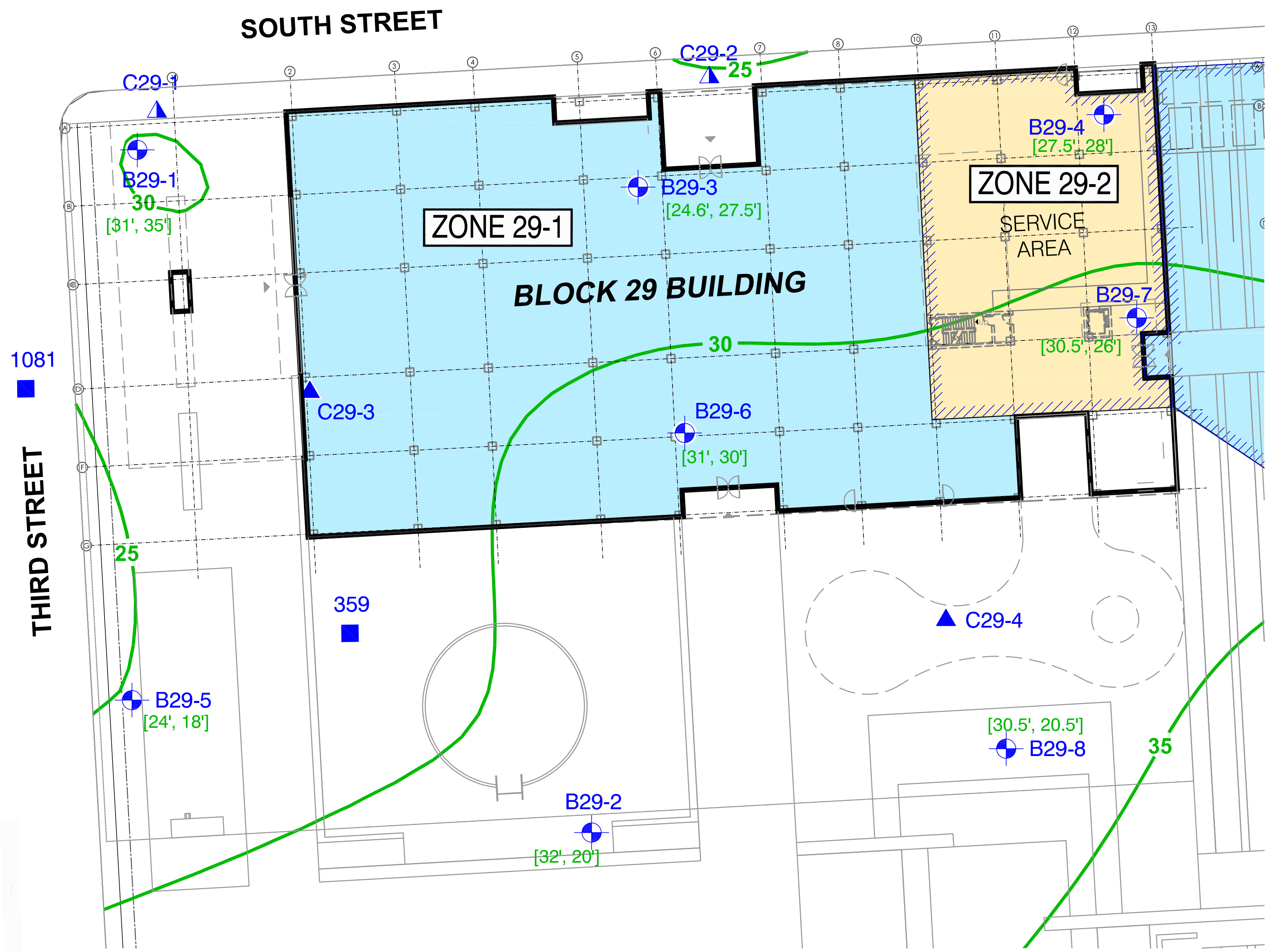
BLOCKS 29-32
MISSION BAY
San Francisco, California

SETTLEMENT POINT PLAN

Date 11/21/11	Project No. 750603902	Figure 13

Treadwell&Rollo
A LANGAN COMPANY



EXPLANATION

B29-1 Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C29-4 Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

359 Approximate locations of boring by others (data base designation)

Top of very dense sand contour (feet, SFCD+100) (see Note 1) Based on interpretation between borings

[24.6', 27.5'] Approximate top of very dense sand layer elevation (feet, SFCD+100 feet) (see Notes 1 and 2) Estimated thickness of very dense sand (feet) (see Notes 1 and 2)

Below-Grade Areas

ZONE 29-1 Pile Capacity Zone

DRAFT

Notes:

1. Contours and thickness are for the very dense portion of the Colma Formation Sand (where SPT N-Value is greater than 50 blows per foot).
2. For CPT and borings that did not extend through the Colma Formation or did not record SPT-N values on the boring logs, the estimated top of very dense sand elevation and thickness is left blank.

Note: See Table 8 for corresponding Pile Capacities.

0 60 Feet

Approximate Scale

BLOCKS 29-32
MISSION BAY
San Francisco, California

BLOCK 29
PILE CAPACITY ZONES

Date 12/02/11	Project No. 750603902	Figure 14

Treadwell&Rollo
A LANGAN COMPANY

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

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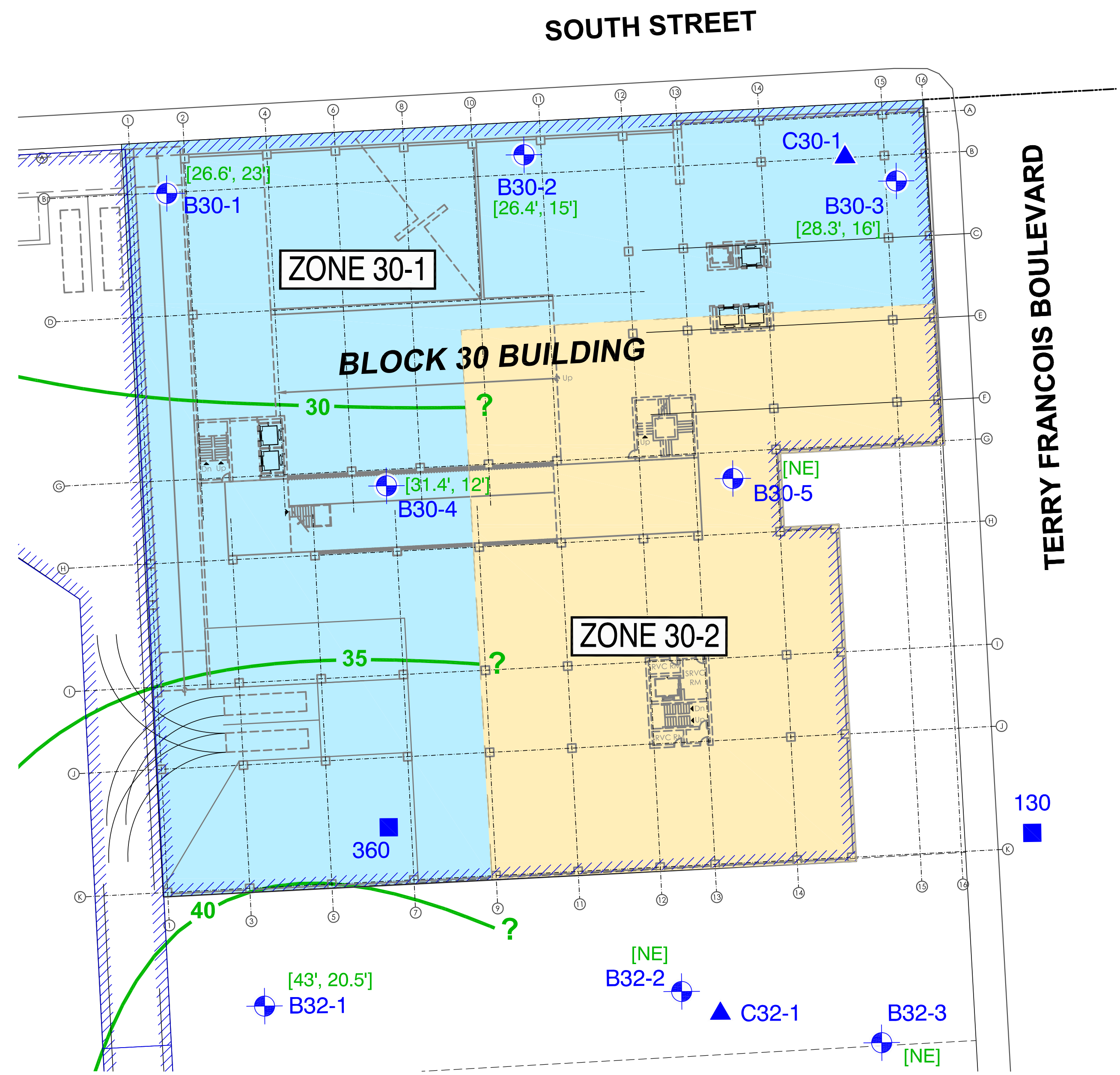

DRAFT

EXPLANATION

B30-1 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C30-1 — Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

360 — Approximate locations of boring by others (data base designation)

Top of very dense sand contour (feet, SFCD+100) (see Note 1) Based on interpretation between borings

[26.6', 23'] — Approximate top of very dense sand layer elevation (feet, SFCD+100 feet) (see Notes 1 and 2) Estimated thickness of very dense sand (feet) (see Notes 1 and 2)

[NE] — Not Encountered (see Note 3)

Below-Grade Areas

ZONE 30-1 — Pile Capacity Zone

Notes:

1. Contours and thickness are for the very dense portion of the Colma Formation sand (where SPT N-Value is greater than 50 blows per foot).
2. For CPT and borings that did not extend through the Colma Formation or did not record SPT-N values on the boring logs, the estimated top of very dense sand elevation and thickness is left blank.
3. In Zone 30-2, the very dense Colma Formation sand was not encountered or there is insufficient data where explored. We estimate piles in the zone will be driven to refusal in bedrock.

Note: See Table 10 for corresponding Pile Capacities.

0 60 Feet

Approximate Scale

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

BLOCKS 29-32
MISSION BAY
San Francisco, California

BLOCK 30
PILE CAPACITY ZONES WITH
TOP OF VERY DENSE SAND CONTOURS

Date 12/20/11	Project No. 750603902	Figure 15

Treadwell&Rollo
A LANGAN COMPANY

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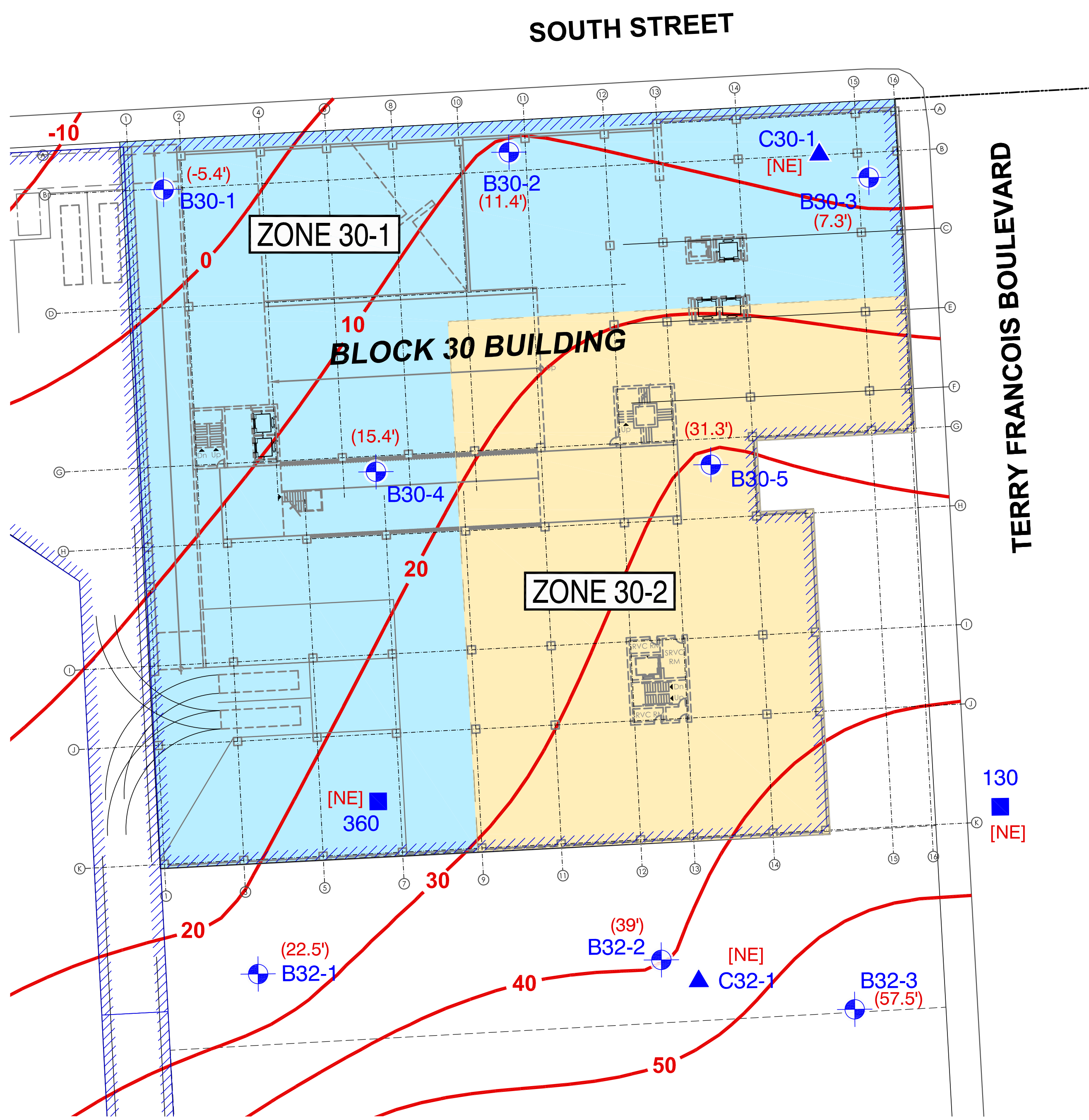

EXPLANATION

B30-1	Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011
C30-1	Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011
360	Approximate locations of boring by others (data base designation)
	Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings
(-5.4')	Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet)
[NE]	Not Encountered because boring/CPT was not advanced to bedrock
	Below-Grade Areas
ZONE 30-2	Pile Capacity Zone

DRAFT

0 60 Feet

Approximate Scale

Note: See Table 10 for corresponding Pile Loads.

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

BLOCKS 29-32
MISSION BAY
San Francisco, California

BLOCK 30
PILE CAPACITY ZONES WITH TOP OF BEDROCK CONTOURS

Date 12/20/11	Project No. 750603902	Figure 16

Treadwell&Rollo
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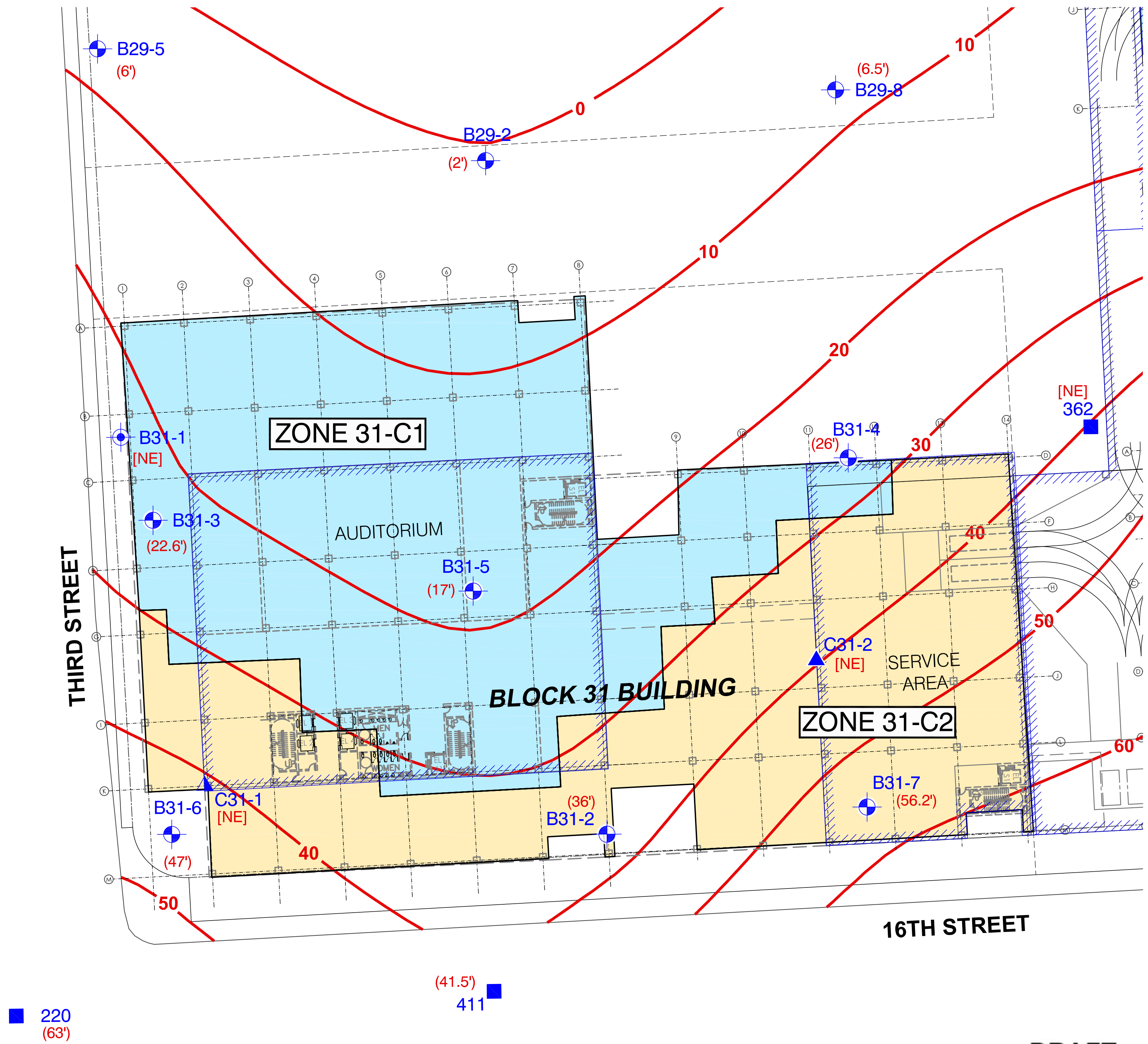

EXPLANATION

B31-3 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C31-3 — Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

B31-1 — Approximate location of boring by Treadwell & Rollo for other developer during previous investigation

361 — Approximate locations of boring by others (data base designation)

Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings

(-27.9') Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet)

[NE] Not Encountered because boring/CPT was not advanced to bedrock

Below-Grade Areas

ZONE 31-C1 Compression Pile Capacity Zone

Note: See Table 11 for corresponding Compression Pile Capacities.

DRAFT

BLOCKS 29-32 MISSION BAY San Francisco, California		
BLOCK 31 COMPRESSION PILE CAPACITY ZONES		
Date 12/20/11	Project No. 750603902	Figure 17

Treadwell&Rollo A LANGAN COMPANY

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

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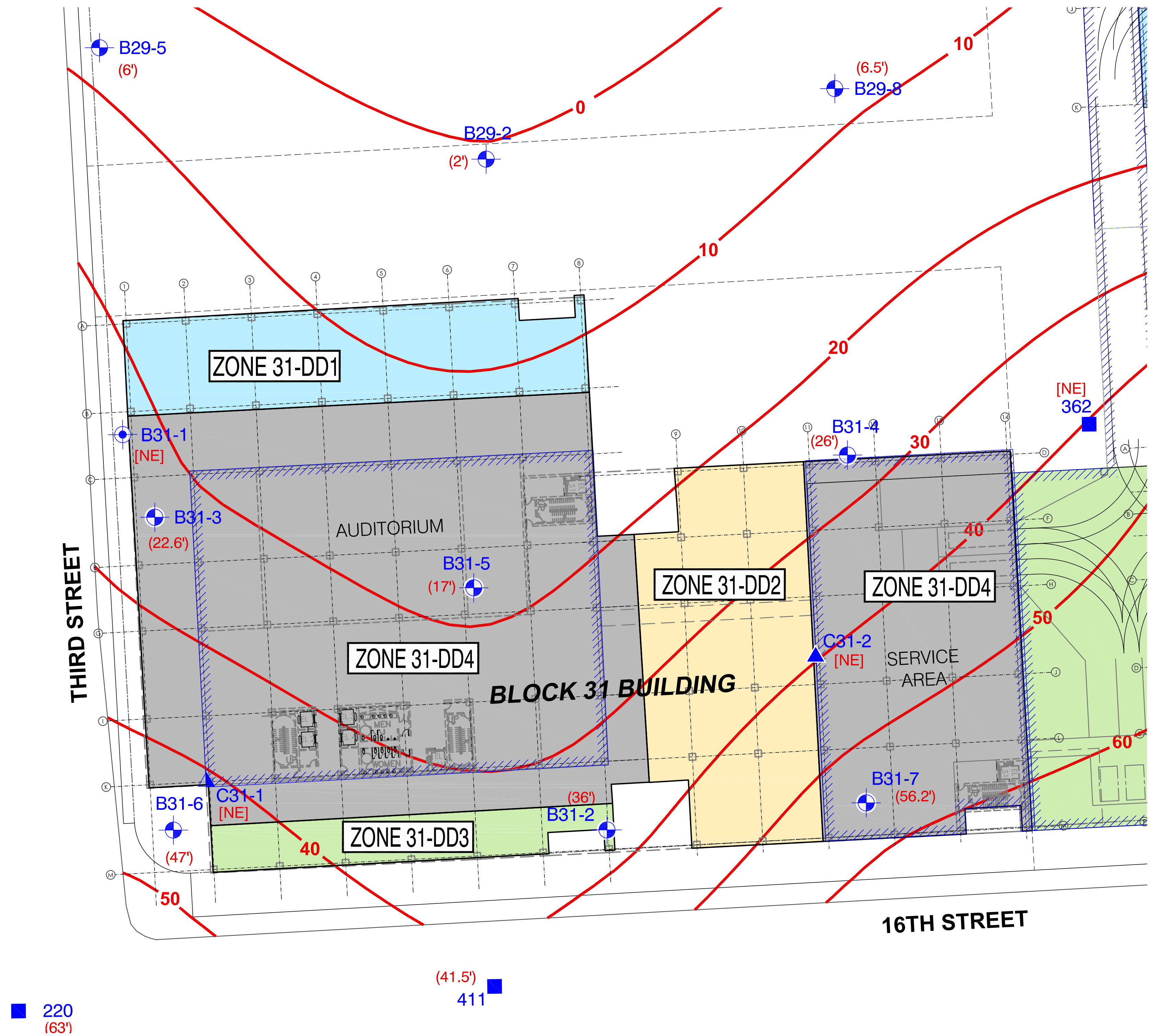

EXPLANATION

B31-3 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C31-3 — Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

B31-1 — Approximate location of boring by Treadwell & Rollo for other developer during previous investigation

361 — Approximate locations of boring by others (data base designation)

Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings

(-27.9') — Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet)

[NE] — Not Encountered because boring/CPT was not advanced to bedrock

Below-Grade Areas

ZONE 31-DD1 — Downdrag Zone

DRAFT

0 60 Feet

Approximate Scale

Note: See Table 12 for corresponding Downdrag Loads.

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

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BLOCKS 29-32
MISSION BAY
San Francisco, California

BLOCK 31
DOWNDRAG ZONES

Date 12/20/11	Project No. 750603902	Figure 18

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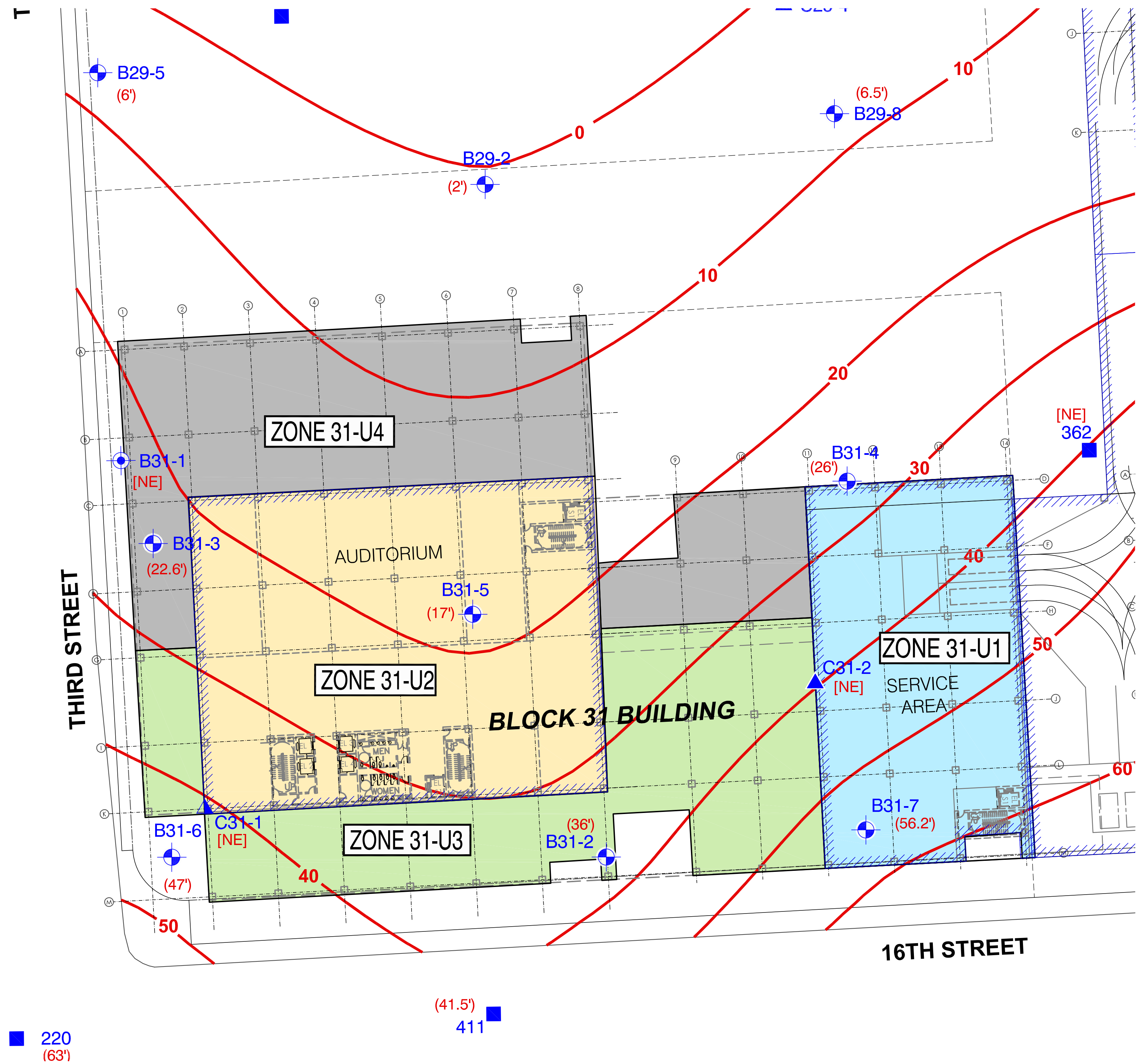


EXPLANATION

B31-3	Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011
C31-3	Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011
B31-1	Approximate location of boring by Treadwell & Rollo for other developer during previous investigation
361	Approximate locations of boring by others (data base designation)
	Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings
(-27.9')	Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet)
[NE]	Not Encountered because boring/CPT was not advanced to bedrock
	Below-Grade Areas
ZONE 31-U1	Uplift Capacity Zone

DRAFT

0 60 Feet
Approximate Scale

Note: See Table 13 for corresponding Uplift Pile Capacities.

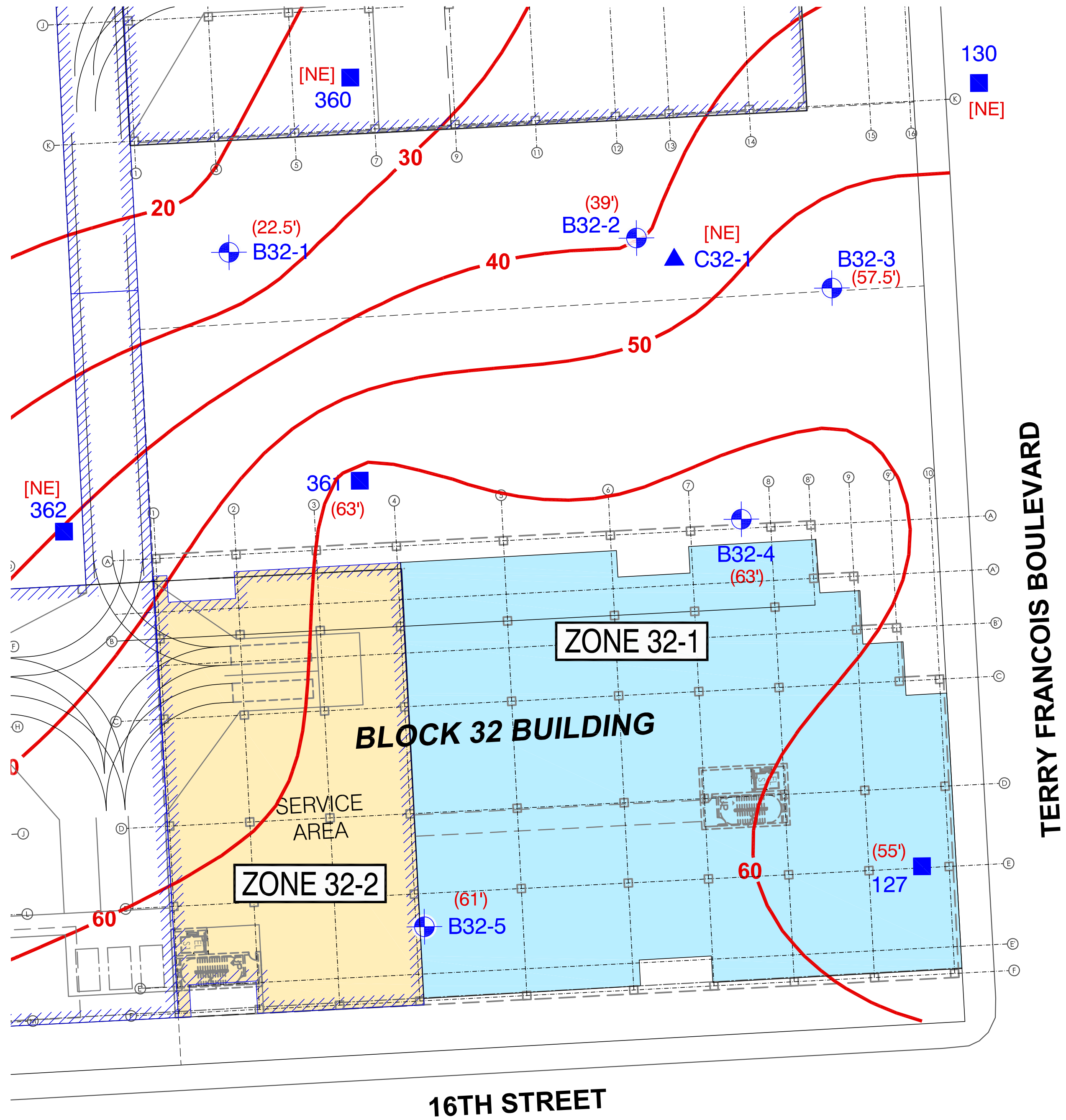
Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

BLOCKS 29-32
MISSION BAY
San Francisco, California

BLOCK 31
UPLIFT PILE CAPACITY ZONES

Date 12/20/11	Project No. 750603902	Figure 19

Treadwell&Rollo
A LANGAN COMPANY



EXPLANATION

B32-3	Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011
C31-3	Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011
361	Approximate locations of boring by others (data base designation)
	Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings
(-27.9')	Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet)
[NE]	Not Encountered because boring/CPT was not advanced to bedrock
	Below-Grade Areas
ZONE 32-1	Pile Capacity Zone

DRAFT

0 60 Feet

Approximate Scale

Note: See Table 14 for corresponding Pile Capacities.

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

BLOCKS 29-32
MISSION BAY
San Francisco, California

BLOCK 32
PILE CAPACITY ZONES

Date 12/20/11	Project No. 750603902	Figure 20

Treadwell&Rollo
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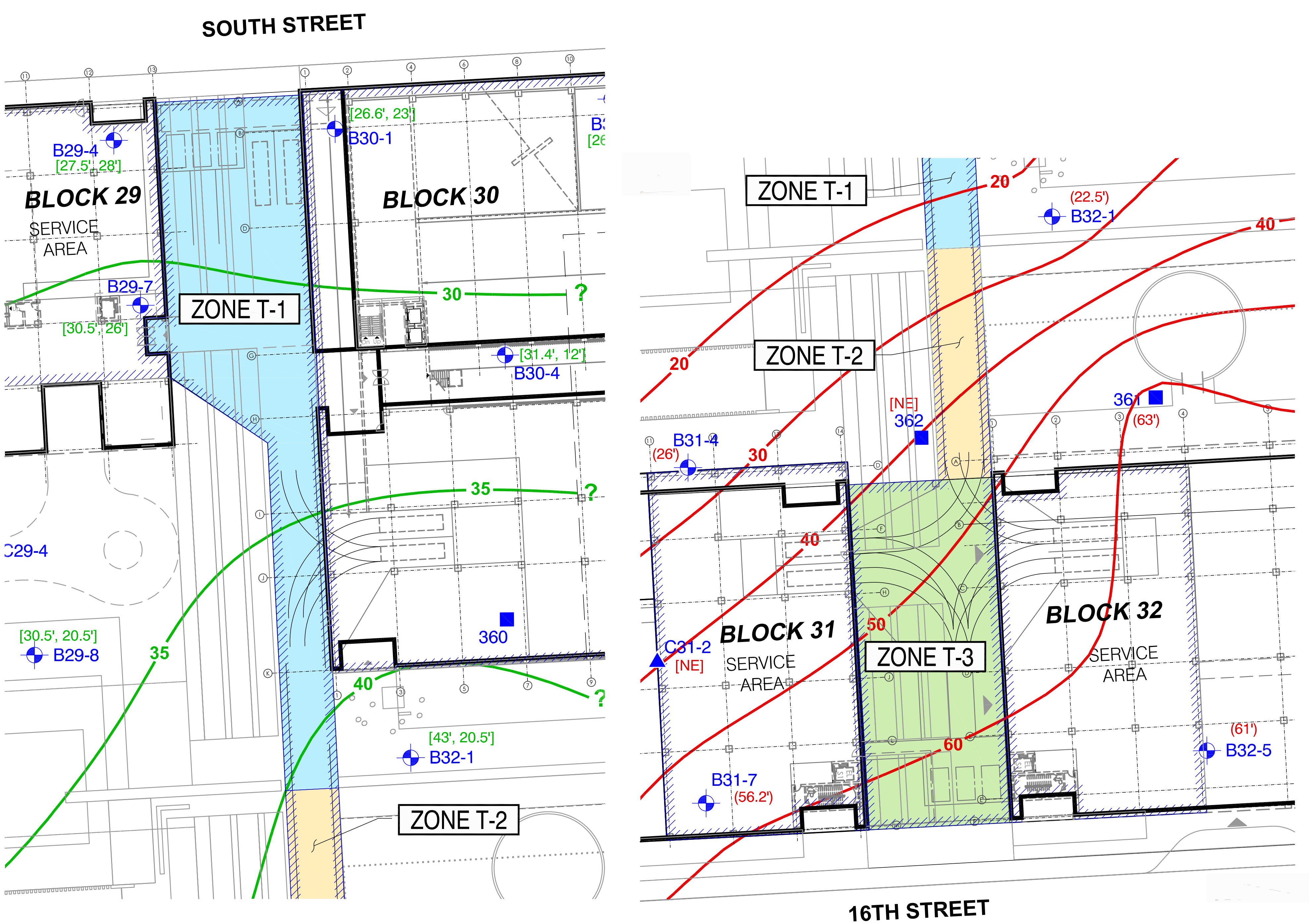

A. TOP OF VERY DENSE SAND CONTOUR

B. TOP OF BEDROCK CONTOUR

EXPLANATION

B30-1 — Approximate location of boring by Treadwell & Rollo, May 2007, August and September 2011

C30-1 — Approximate location of cone penetration test by Treadwell & Rollo, May 2007 and August 2011

360 — Approximate locations of boring by others (data base designation)

Below-Grade Areas

ZONE T-1 — Pile Capacity Zone

Approximate Bedrock Contour (feet, SFCD + 100 feet) Based on interpretion between borings

(-5.4') — Approximate Top of Bedrock Elevation (feet, SFCD + 100 feet) Based on interpretation between borings

[NE] — Not Encountered because boring/CPT was not advanced to bedrock

Top of very dense sand contour (feet, SFCD+100);

[24.6', 27.5'] — Approximate top of very dense sand layer elevation (feet, SFCD+100 feet) (see Note 1)
Estimated thickness of very dense sand (feet) (see Note 1)

Notes:
1. The contour and thickness for the very dense sand is for the portion of the Colma Formation with SPT N-Value greater than 50 blows per foot.
2. See Table 15 for corresponding pile capacities. We estimate piles within Zone T-1 will bear in the very dense sand and piles in Zone T-2 and T-3 will bear in bedrock.

DRAFT

0 60 Feet
Approximate Scale

BLOCKS 29-32
MISSION BAY
San Francisco, California

SERVICE TUNNEL
PILE CAPACITY ZONES

Date 12/20/11	Project No. 750603902	Figure 21

Treadwell&Rollo
A LANGAN COMPANY

Reference: Base map from "Schematic Design, Level 1 Base Map, A1-01" provided by Flad Architects, dated 30 August 2011.

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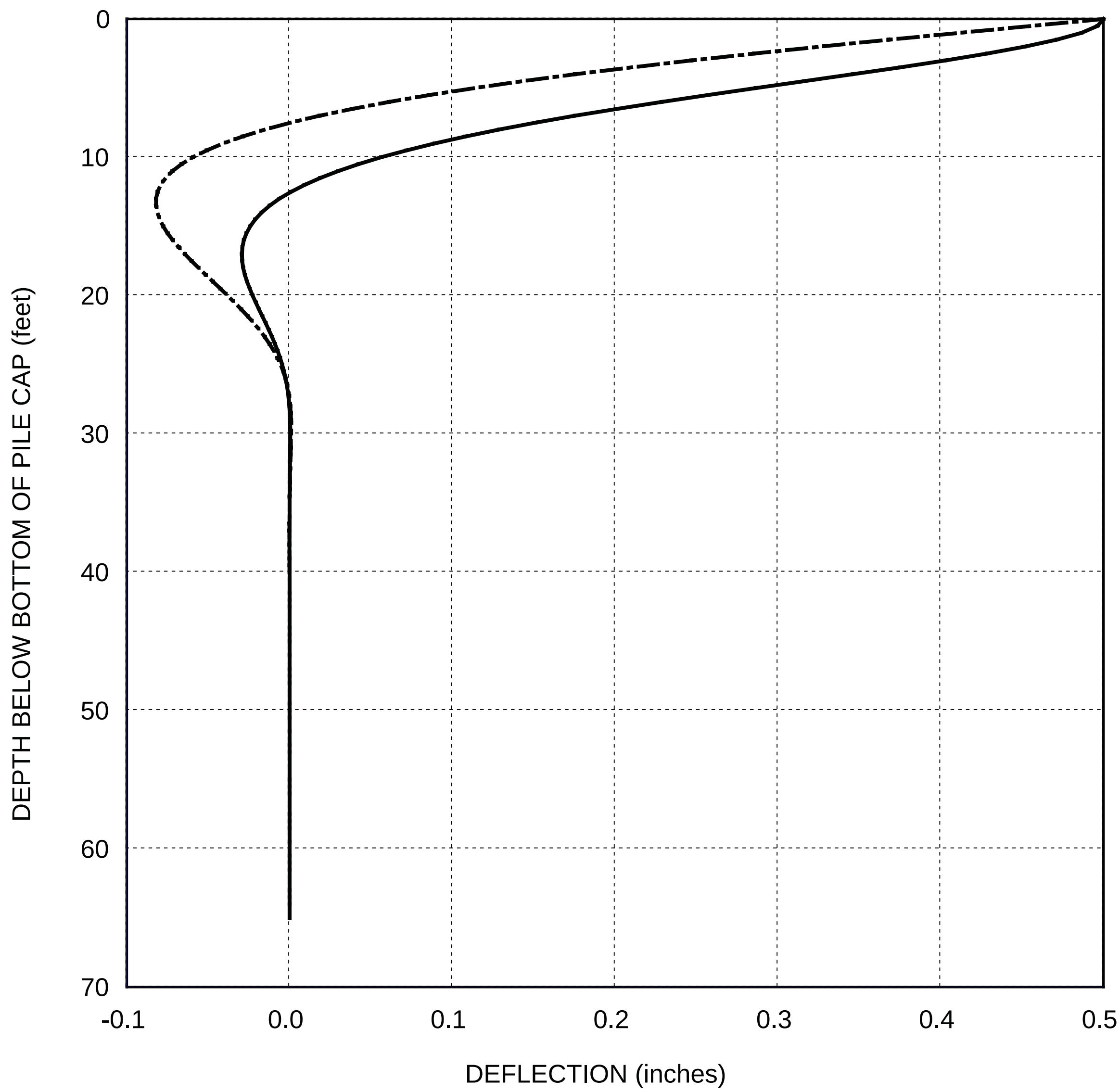

Symbol	Lateral Load	Pile Head Connection
———	62 Kips	Fixed
—·—·—·—	25 Kips	Free

DRAFT

Notes:

1. The profiles shown are for a single 14-inch square prestressed precast (PSPC) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 350 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 100.5 feet, and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 94 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
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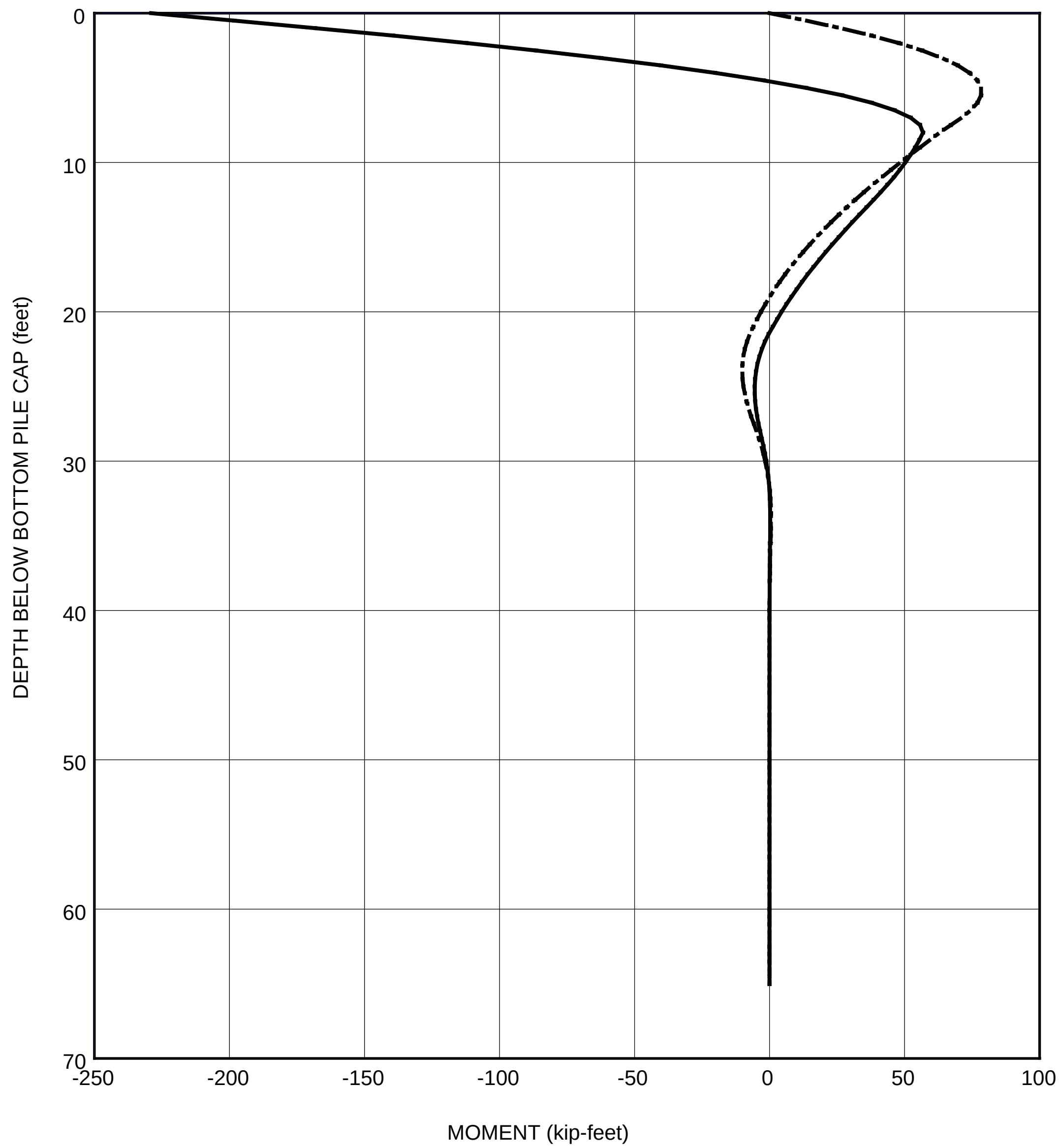
DEFLECTION PROFILE
14-INCH SQUARE PSPC CONCRETE PILE
BLOCK 29 - AT GRADE
LIQUEFACTION PARTIALLY MITIGATED

Date 12/12/11	Project No. 750603902	Figure 22



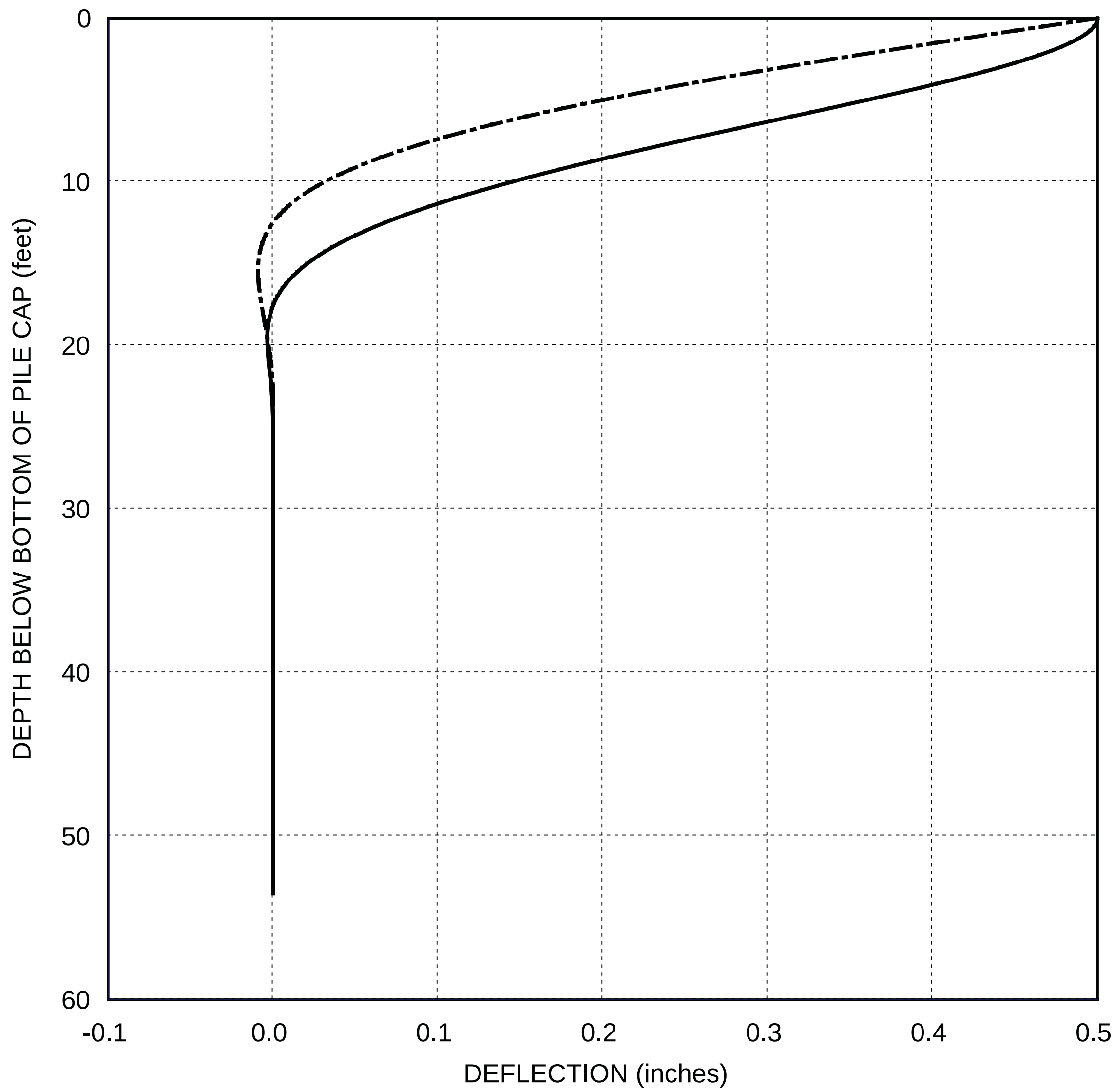
Symbol	Lateral Load	Pile Head Connection
(solid line)	62 Kips	Fixed
(dash-dot line)	25 Kips	Free

DRAFT

Notes:

1. The profiles shown are for a single 14-inch square prestressed precast (PSPC) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 350 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 100.5 feet, and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 94 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32 MISSION BAY San Francisco, California	MOMENT PROFILE 14-INCH SQUARE PSPC CONCRETE PILE BLOCK 29 - AT GRADE LIQUEFACTION PARTIALLY MITIGATED
Treadwell&Rollo A LANGAN COMPANY	Date 11/29/11 | Project No. 750603902 | Figure 23



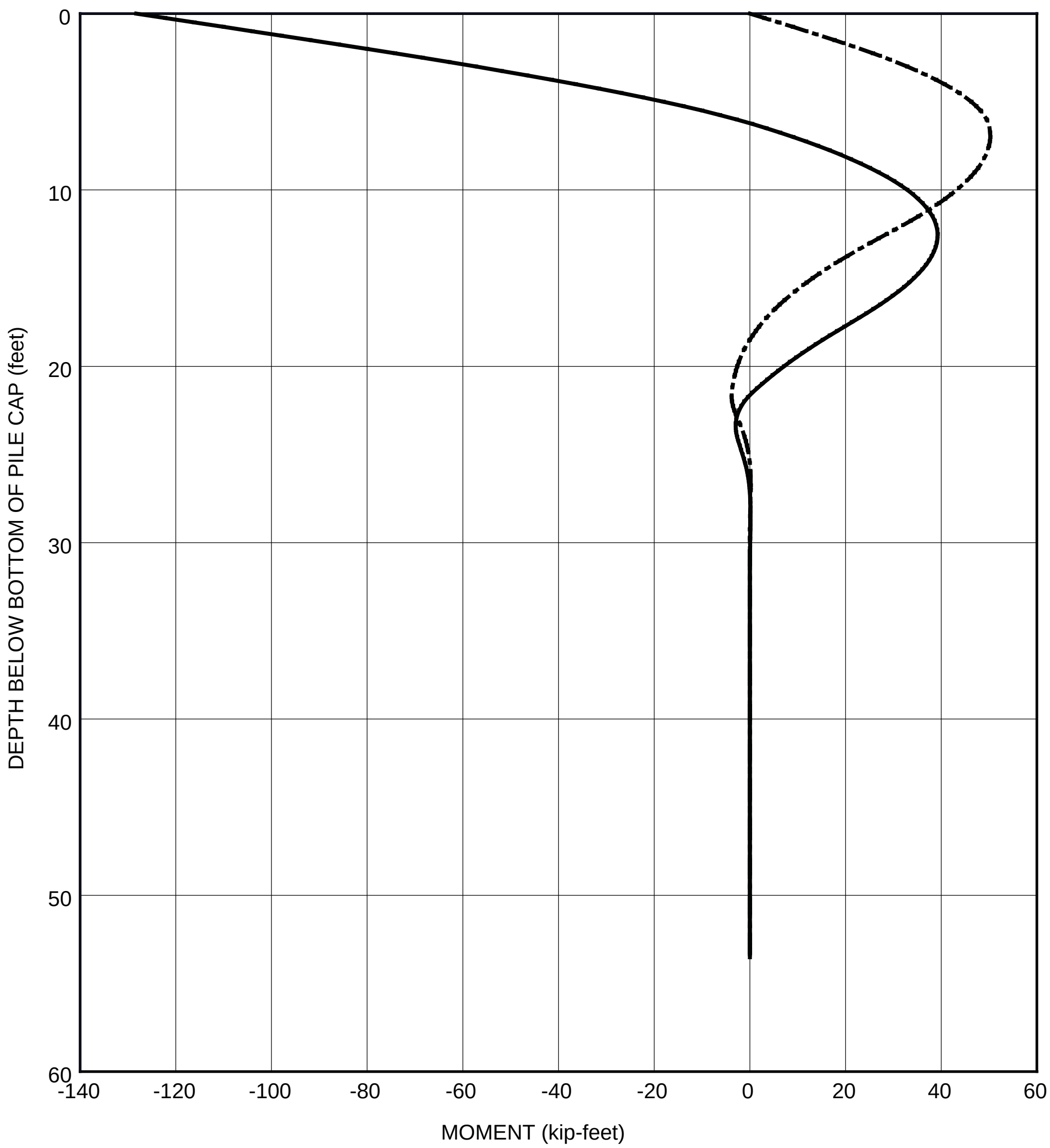
Symbol	Lateral Load	Pile Head Connection
———————	24 Kips	Fixed
—·—·—·—	10 Kips	Free

DRAFT

Notes:

1. The profiles shown are for a single 14-inch square prestressed precast (PCPS) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 400 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 79 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32 **MISSION BAY** San Francisco, California	**DEFLECTION PROFILE** **14-INCH SQUARE PSPC CONCRETE PILE** **BLOCK 29 - SERVICE AREA**
Treadwell&Rollo A LANGAN COMPANY	Date 11/29/11 · Project No. 750603902 · Figure 24

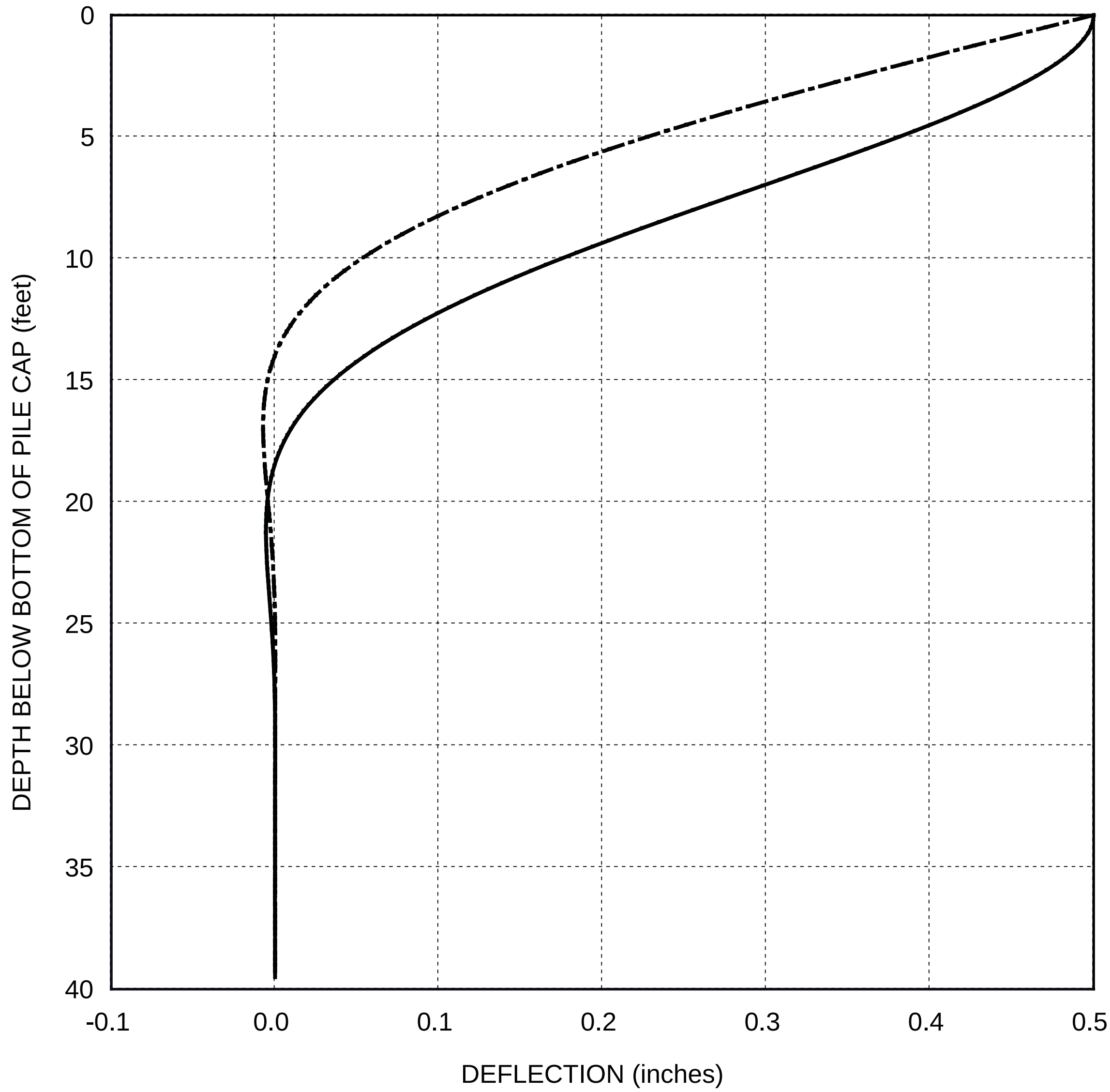


Symbol	Lateral Load	Pile Head Connection
———— (solid line)	24 Kips	Fixed
—·—·—·— (dash-dot line)	10 Kips	Free

DRAFT

Notes:
1. The profiles shown are for a single 14-inch square prestressed precast (PCPS) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 400 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 79 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32 MISSION BAY San Francisco, California	MOMENT PROFILE 14-INCH SQUARE PSPC CONCRETE PILE BLOCK 29 - SERVICE AREA
Treadwell&Rollo A LANGAN COMPANY	Date 12/14/11 · Project No. 750603902 · Figure 25



Symbol	Lateral Load	Pile Head Connection
———	16 Kips	Fixed
—·—·—	6.5 Kips	Free

DRAFT

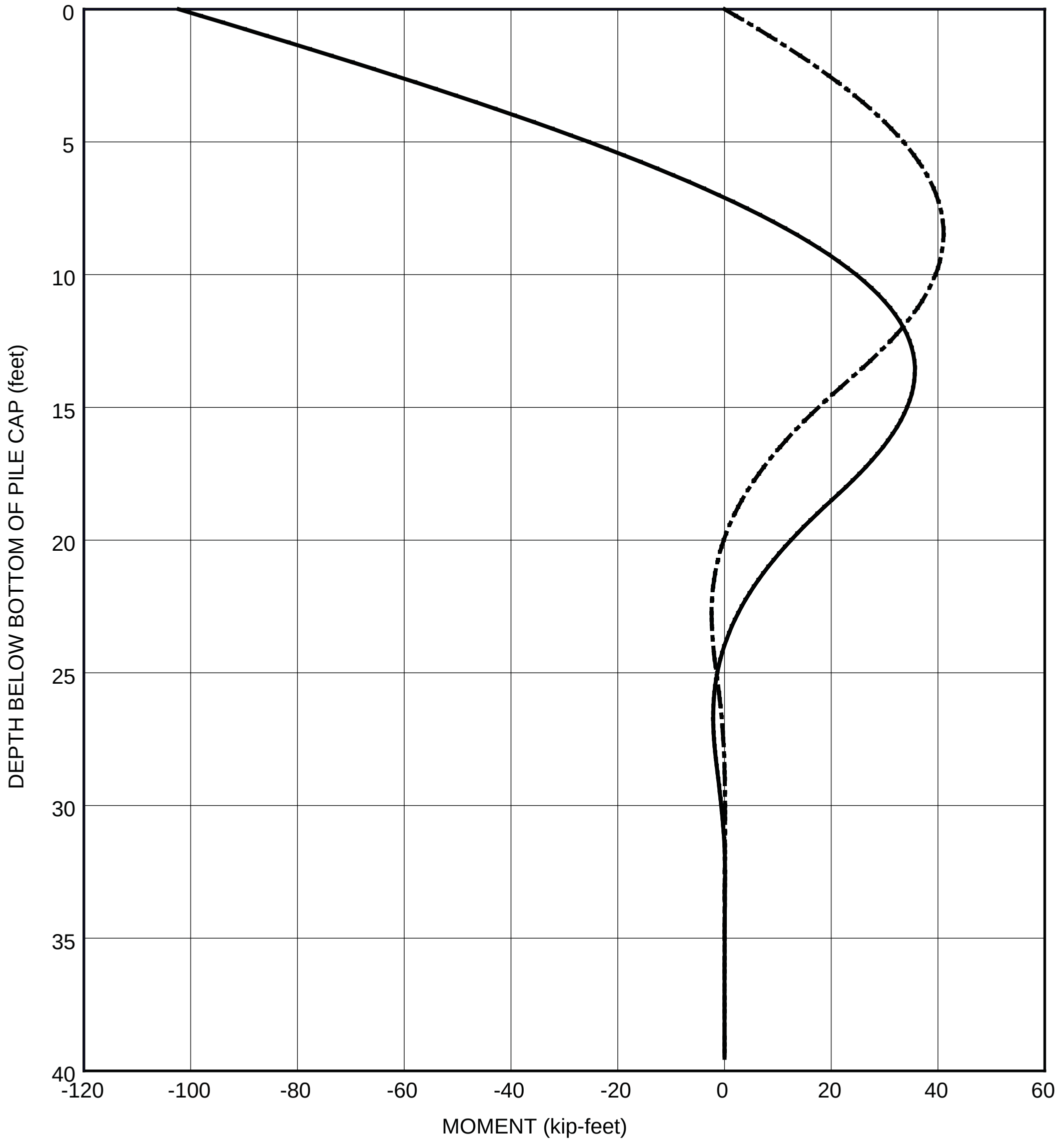
Notes:
1. The profiles shown are for a single 14-inch square prestresed precast concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 375 kips.
2. To account for group effects, the lateral load capacty of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 74.5 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DEFLECTION PROFILE
14-INCH SQUARE PCPS CONCRETE PILE
BLOCK 30 - BASEMENT

Date 11/29/11 | Project No. 750603902 | Figure 26



Symbol	Lateral Load	Pile Head Connection
————	16 Kips	Fixed
—·—·—·—	6.5 Kips	Free

DRAFT

Notes:

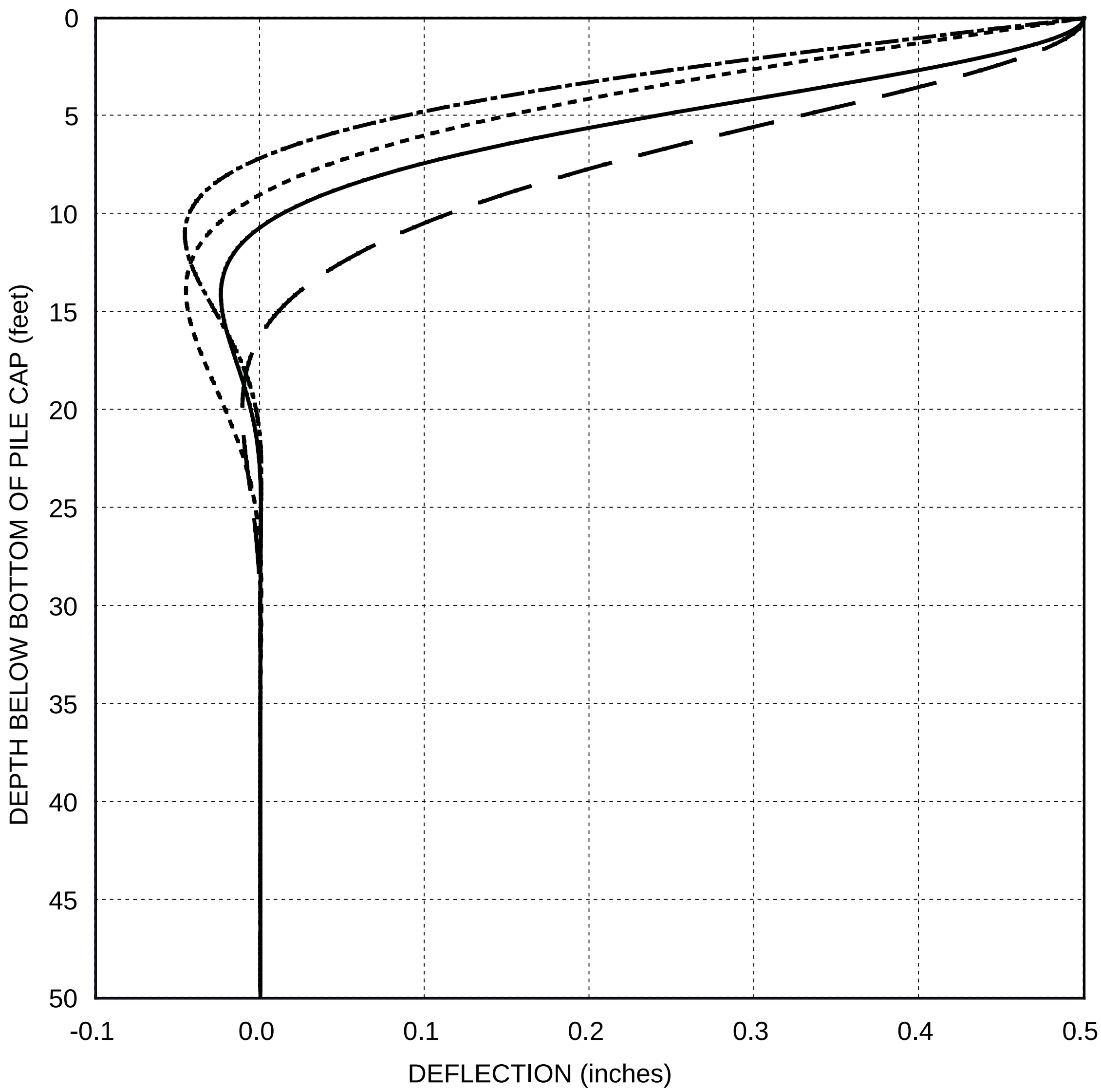
1. The profiles shown are for a single 14-inch square prestressed precast concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 375 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 74.5 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MOMENT PROFILE
14-INCH SQUARE PCPS CONCRETE PILE
BLOCK 30 - BASEMENT

Date 11/29/11	Project No. 750603902	Figure 27



Symbol	Lateral Load (kips)	Pile Head Connection	Loading Direction
————————	51	Fixed	Weak
—·—·—·—	21	Free	Weak
—— ——	70	Fixed	Strong
------------	31	Free	Strong

DRAFT

Notes:

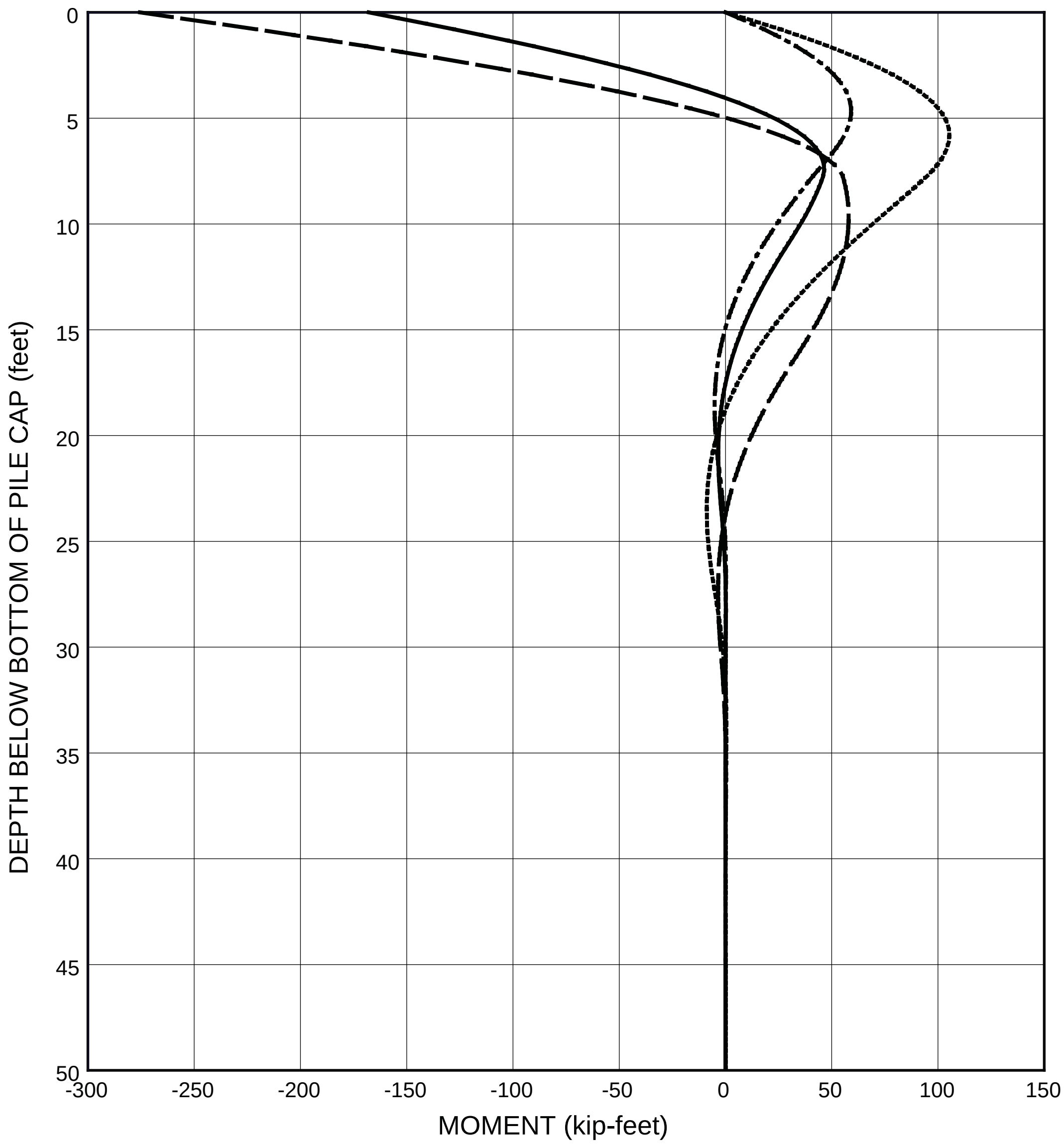
1. The profiles shown are for a single HP14x73 steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 390 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 101.5 feet and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 94.5 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DEFLECTION PROFILE
HP 14X73 STEEL PILE
BLOCK 31 - AT GRADE
LIQUEFACTION PARTIALLY MITIGATED

Date 12/13/11 | Project No. 750603902 | Figure 28



Symbol	Lateral Load (kips)	Pile Head Connection	Loading Direction
———	51	Fixed	Weak
—·—·—	21	Free	Weak
— —	70	Fixed	Strong
------	31	Free	Strong

DRAFT

Notes:

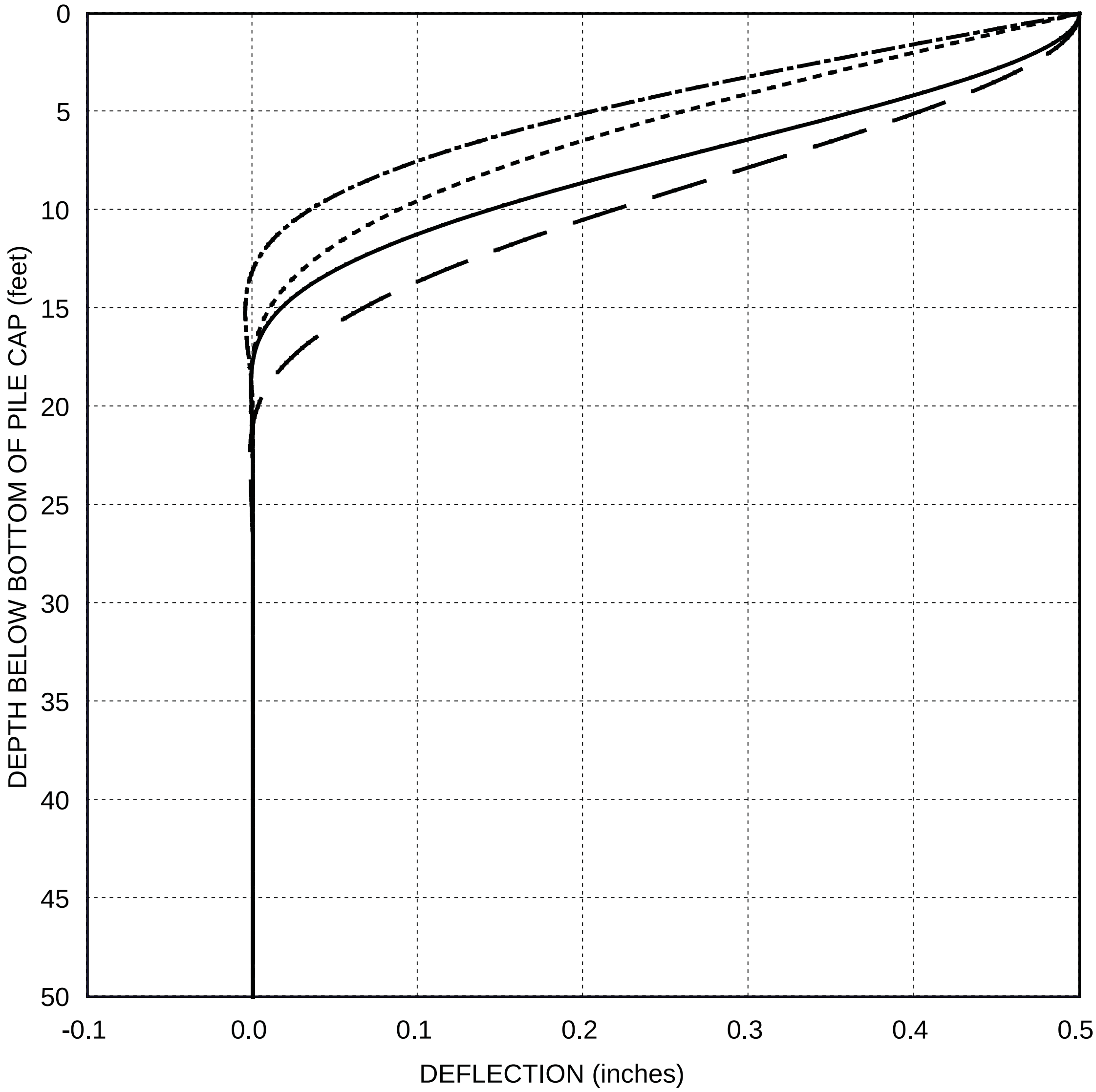
1. The profiles shown are for a single HP14x73 steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 390 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 101.5 feet and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 94.5 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MOMENT PROFILE
HP 14X73 STEEL PILE
BLOCK 31 - AT GRADE
LIQUEFACTION PARTIALLY MITIGATED

Date 12/13/11	Project No. 750603902	Figure 29



Symbol	Lateral Load (kips)	Pile Head Connection	Loading Direction
———— (solid)	11.5	Fixed	Weak
—·—·—·— (dash-dot)	4.5	Free	Weak
—— —— (long dash)	17.5	Fixed	Strong
------------ (short dash)	7	Free	Strong

DRAFT

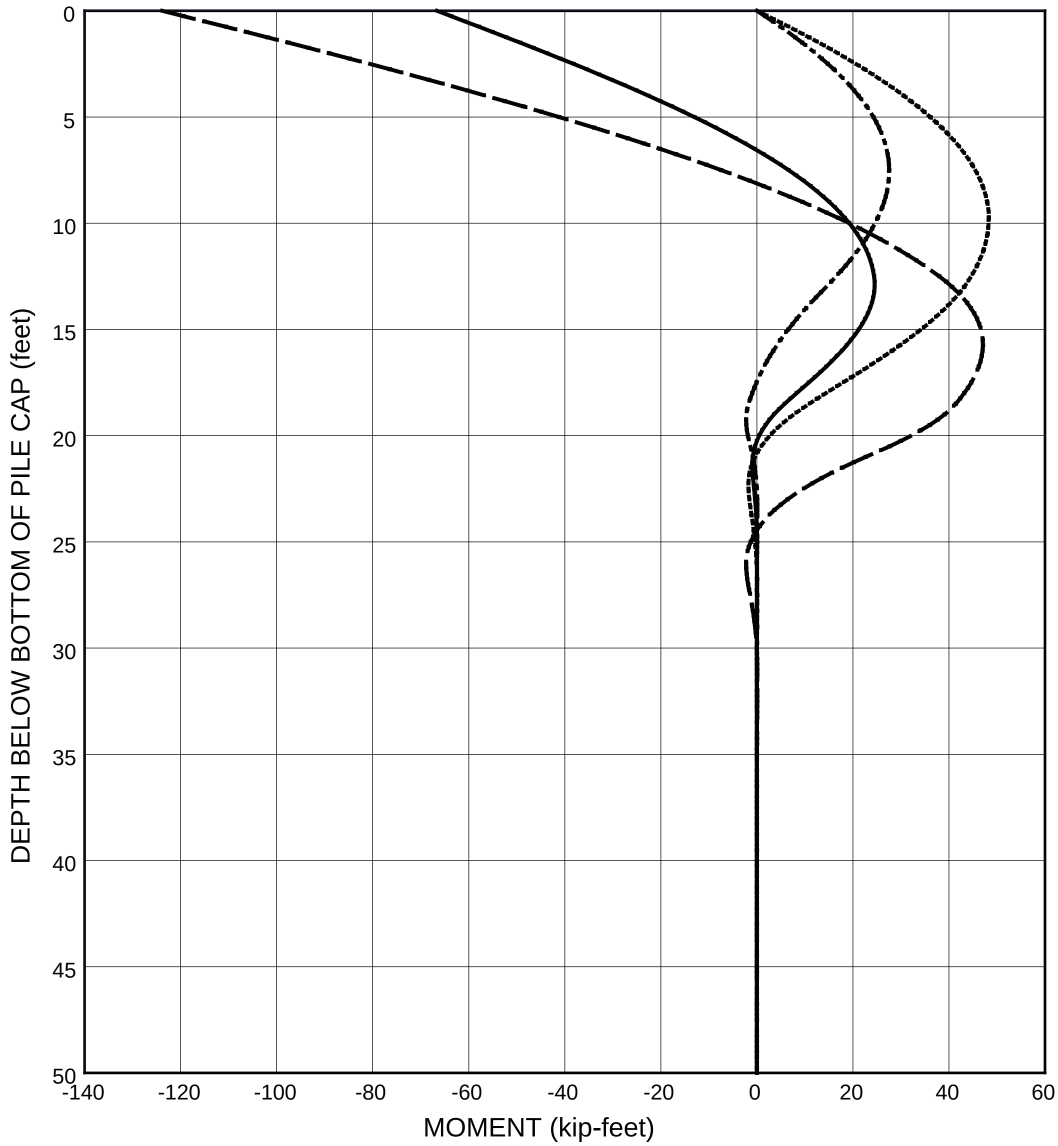
Notes:
1. The profiles shown are for a single HP14x73 steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 400 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 101.5 feet and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 86.5 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DEFLECTION PROFILE
HP 14X73 STEEL PILE
BLOCK 31 - AUDITORIUM
LIQUEFACTION PARTIALLY MITIGATED

Date 12/13/11	Project No. 750603902	Figure 30



Symbol	Lateral Load (kips)	Pile Head Connection	Loading Direction
———————	11.5	Fixed	Weak
—·—·—·—	4.5	Free	Weak
—— ——	17.5	Fixed	Strong
-----------	7	Free	Strong

DRAFT

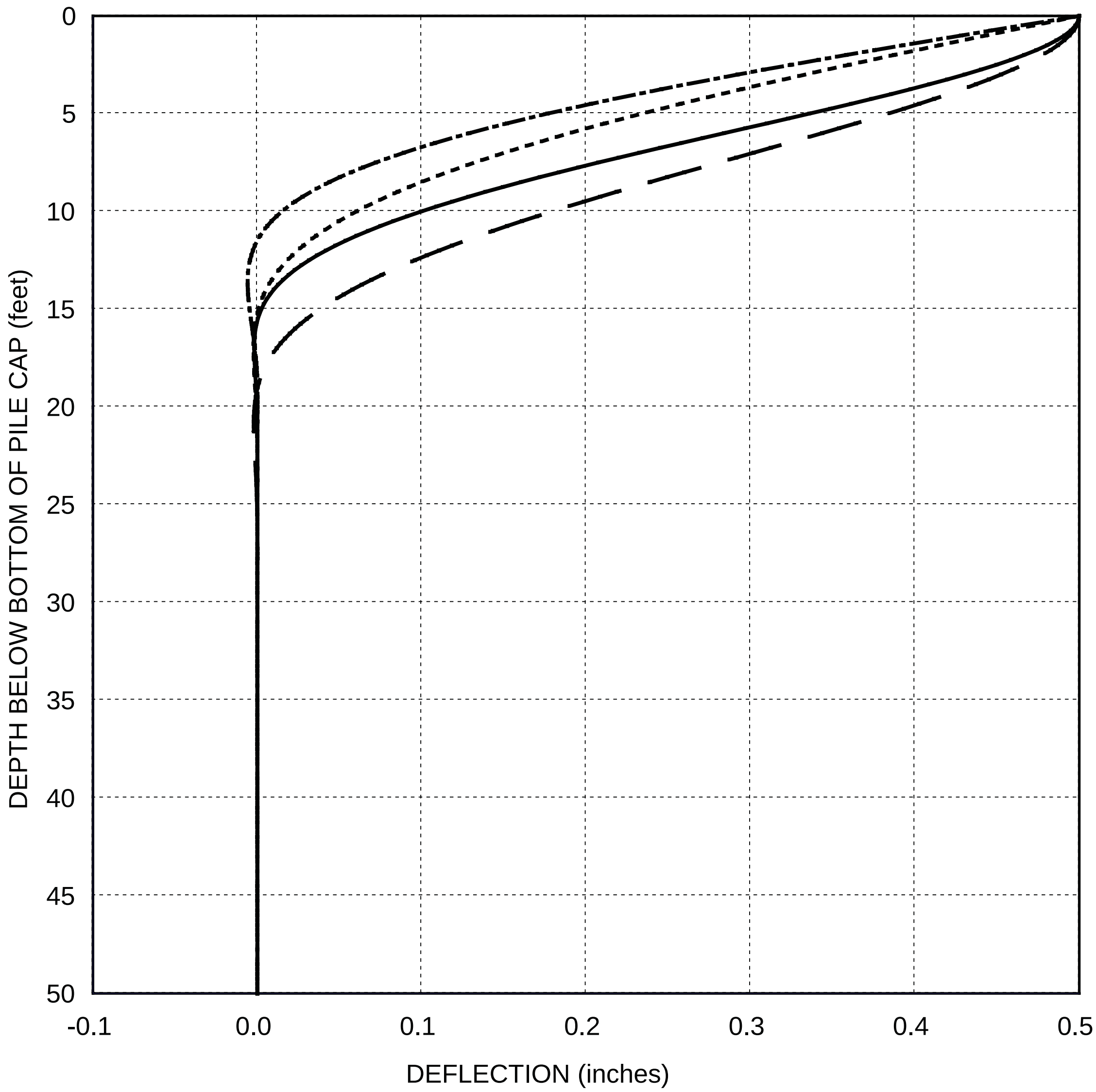
Notes:
1. The profiles shown are for a single HP14x73 steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 400 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 101.5 feet and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 86.5 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MOMENT PROFILE
HP 14X73 STEEL PILE
BLOCK 31 - AUDITORIUM
LIQUEFACTION PARTIALLY MITIGATED

Date 11/29/11	Project No. 750603902	Figure 31



Symbol	Lateral Load (kips)	Pile Head Connection	Loading Direction
	16.5	Fixed	Weak
	6.5	Free	Weak
	24.5	Fixed	Strong
	10.5	Free	Strong

DRAFT

Notes:

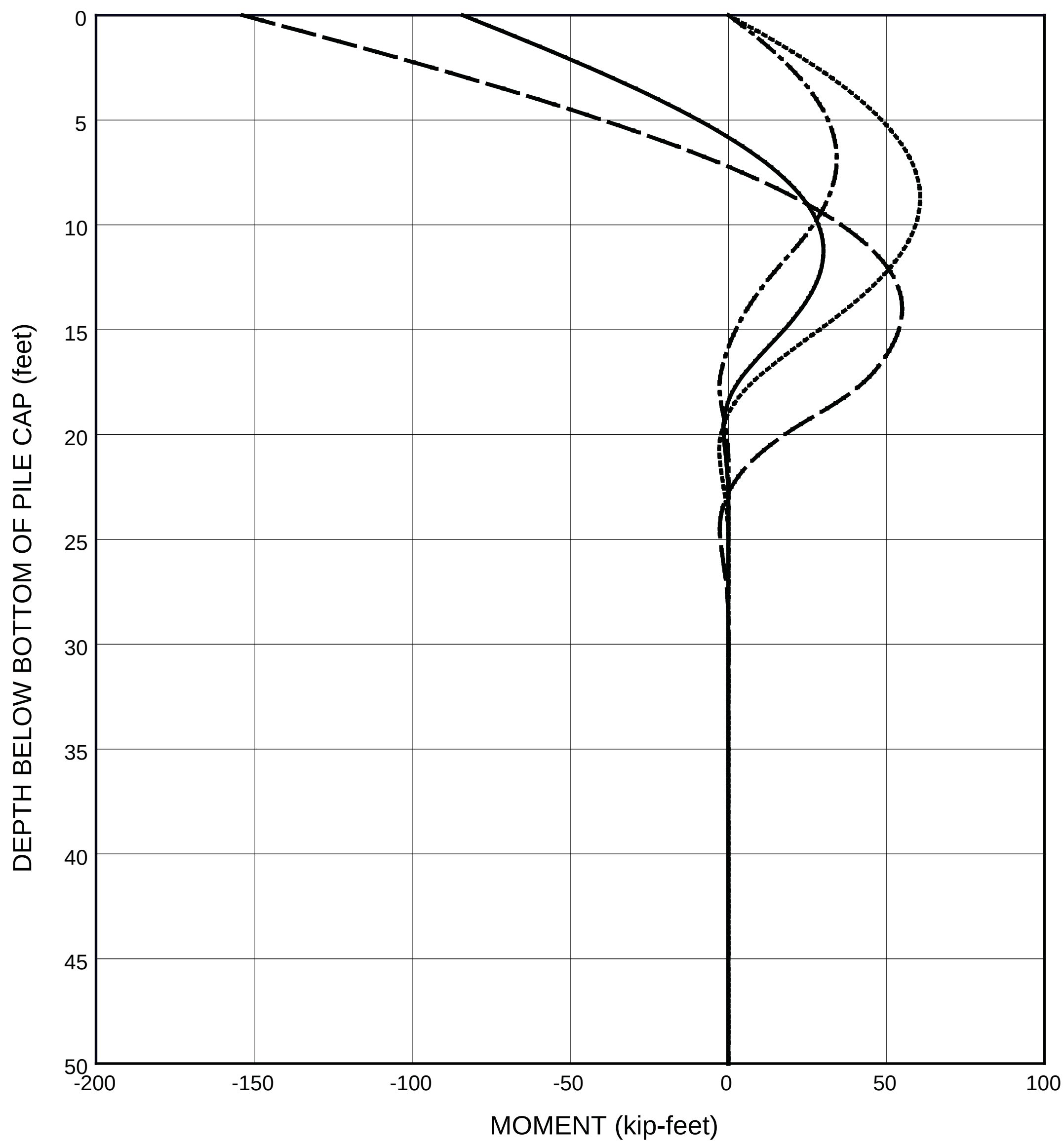
1. The profiles shown are for a single HP14x73 steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 400 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 79 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DEFLECTION PROFILE
HP 14X73 STEEL PILE
BLOCK 31 - SERVICE AREA

Date 12/13/11	Project No. 750603902	Figure 32



Symbol	Lateral Load (kips)	Pile Head Connection	Loading Direction
——— (solid)	16.5	Fixed	Weak
—·—·— (dash-dot)	6.5	Free	Weak
—— —— (long dash)	24.5	Fixed	Strong
- - - - (short dash)	10.5	Free	Strong

DRAFT

Notes:

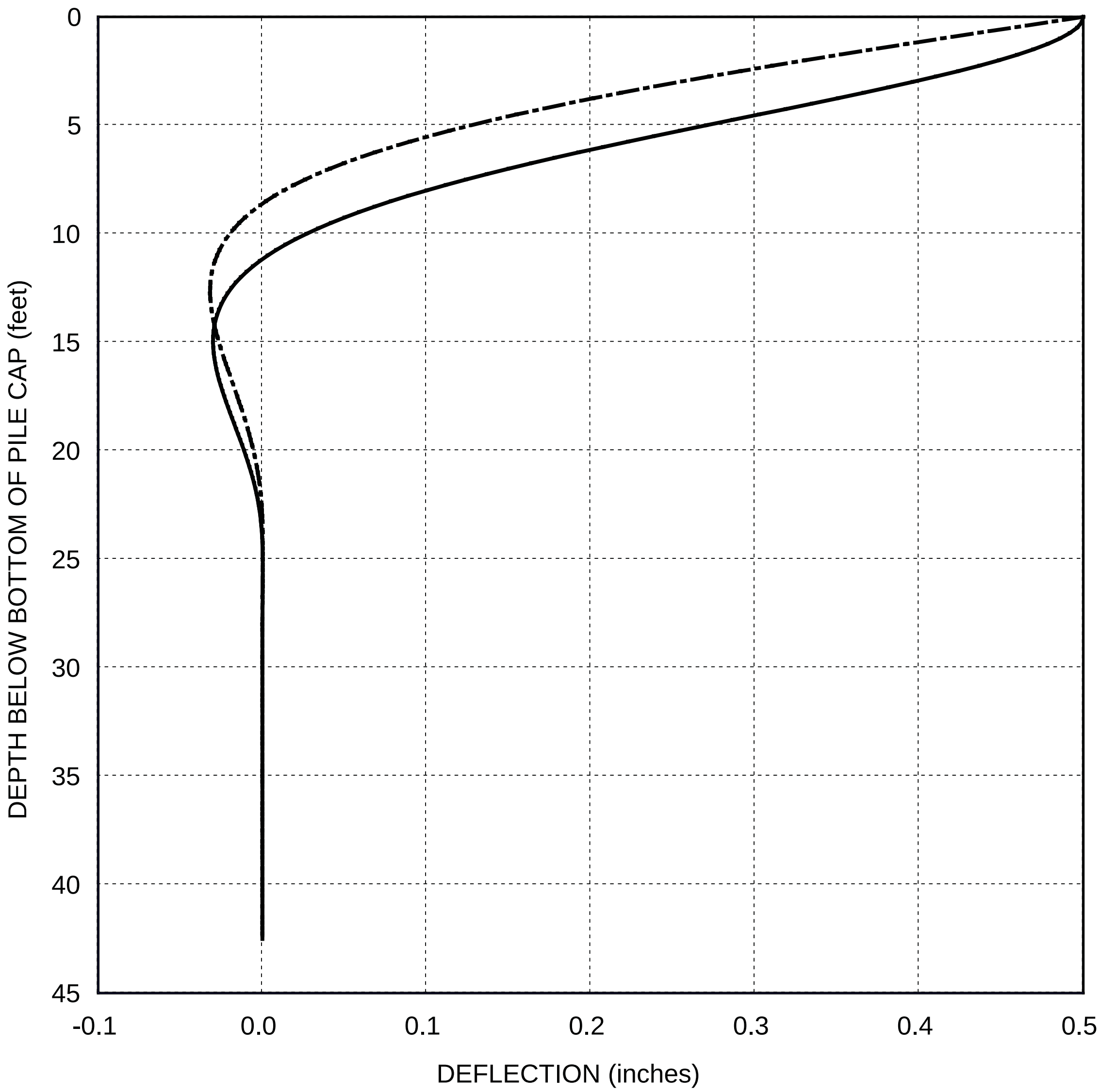
1. The profiles shown are for a single HP14x73 steel pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 400 kips. Pile section chosen in final design will need to include corrosion allowance, as discussed in Appendix D.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 79 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MOMENT PROFILE
HP 14X73 STEEL PILE
BLOCK 31 - SERVICE AREA

Date 12/14/11	Project No. 750603902	Figure 33



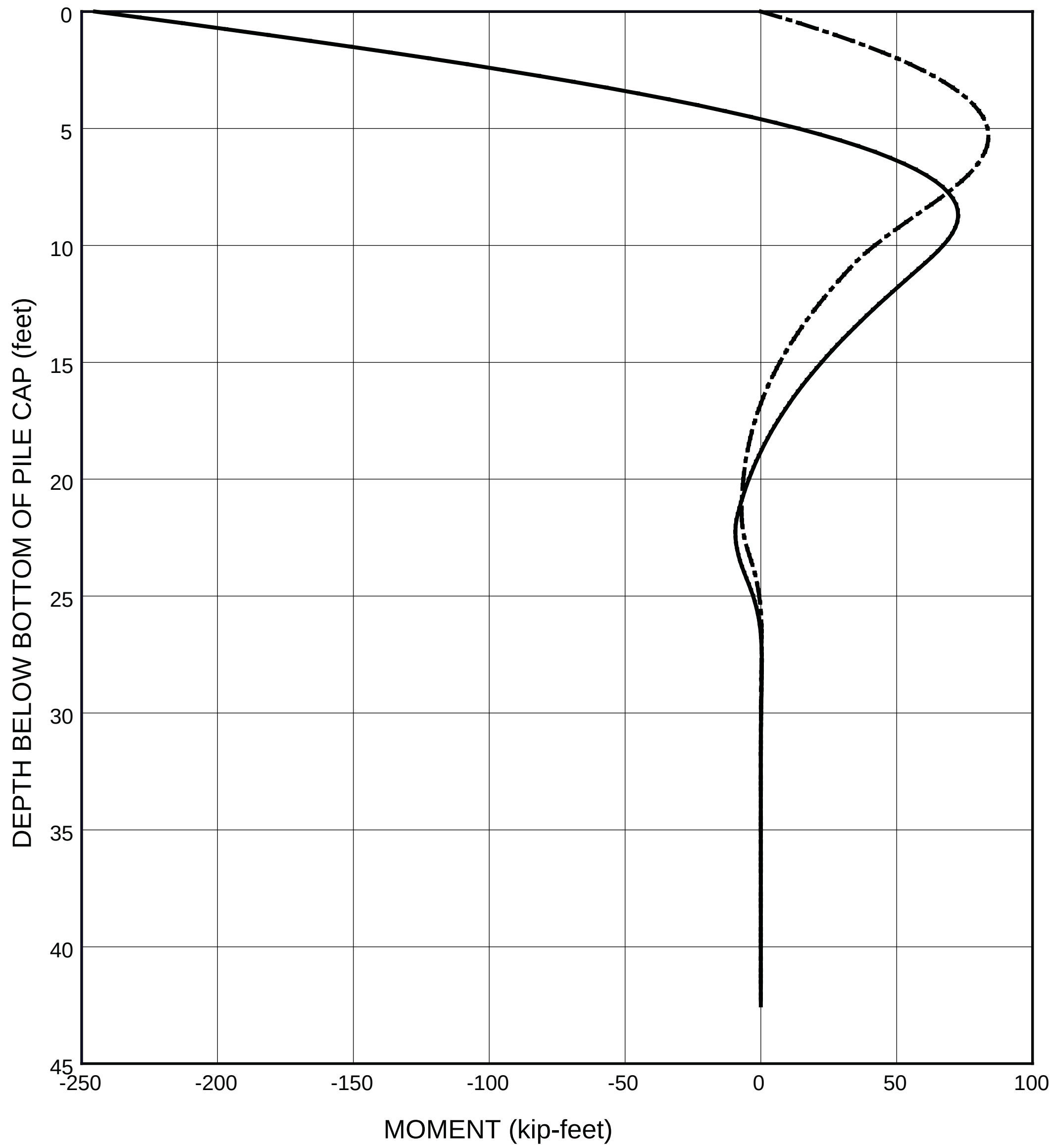
Symbol	Lateral Load	Pile Head Connection
solid line	65.5 Kips	Fixed
dash-dot line	26.5 Kips	Free

DRAFT

Notes:

1. The profiles shown are for a single 14-inch square prestressed precast (PSPC) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 375 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 100.5 feet, and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 92.6 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32 **MISSION BAY** San Francisco, California	**DEFLECTION PROFILE** **14-INCH SQUARE PSPC CONCRETE PILE** **BLOCK 32 - AT GRADE** **LIQUEFACTION PARTIALLY MITIGATED**
Treadwell&Rollo A LANGAN COMPANY	Date 12/13/11 | Project No. 750603902 | Figure 34



Symbol	Lateral Load	Pile Head Connection
—————— (solid)	65.5 Kips	Fixed
—·—·—·— (dash-dot)	26.5 Kips	Free

DRAFT

Notes:

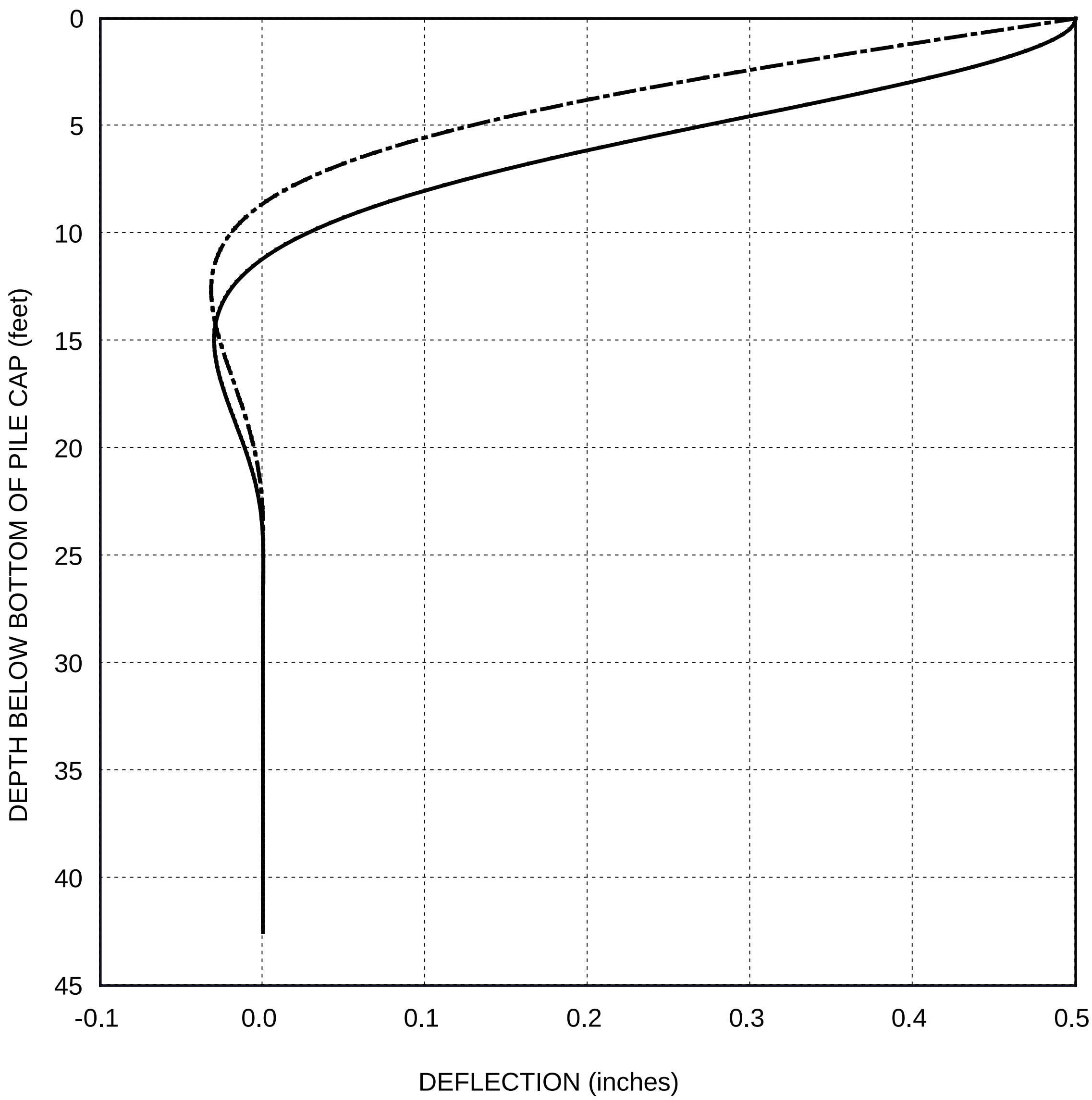
1. The profiles shown are for a single 14-inch square prestressed precast (PSPC) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 375 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes existing ground surface is at approximately Elevation 100.5 feet, and the site has been improved to mitigate against liquefaction to 15 feet below existing ground elevation.
5. Assumes top of pile is at approximately Elevation 92.6 feet and the lateral load is applied at the top of the pile.
6. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MOMENT PROFILE
14-INCH SQUARE PSPC CONCRETE PILE
BLOCK 32 - AT GRADE
LIQUEFACTION PARTIALLY MITIGATED

Date 12/13/11	Project No. 750603902	Figure 35



Symbol	Lateral Load	Pile Head Connection
——— (solid)	24 Kips	Fixed
—·—·—·— (dash-dot)	9 Kips	Free

DRAFT

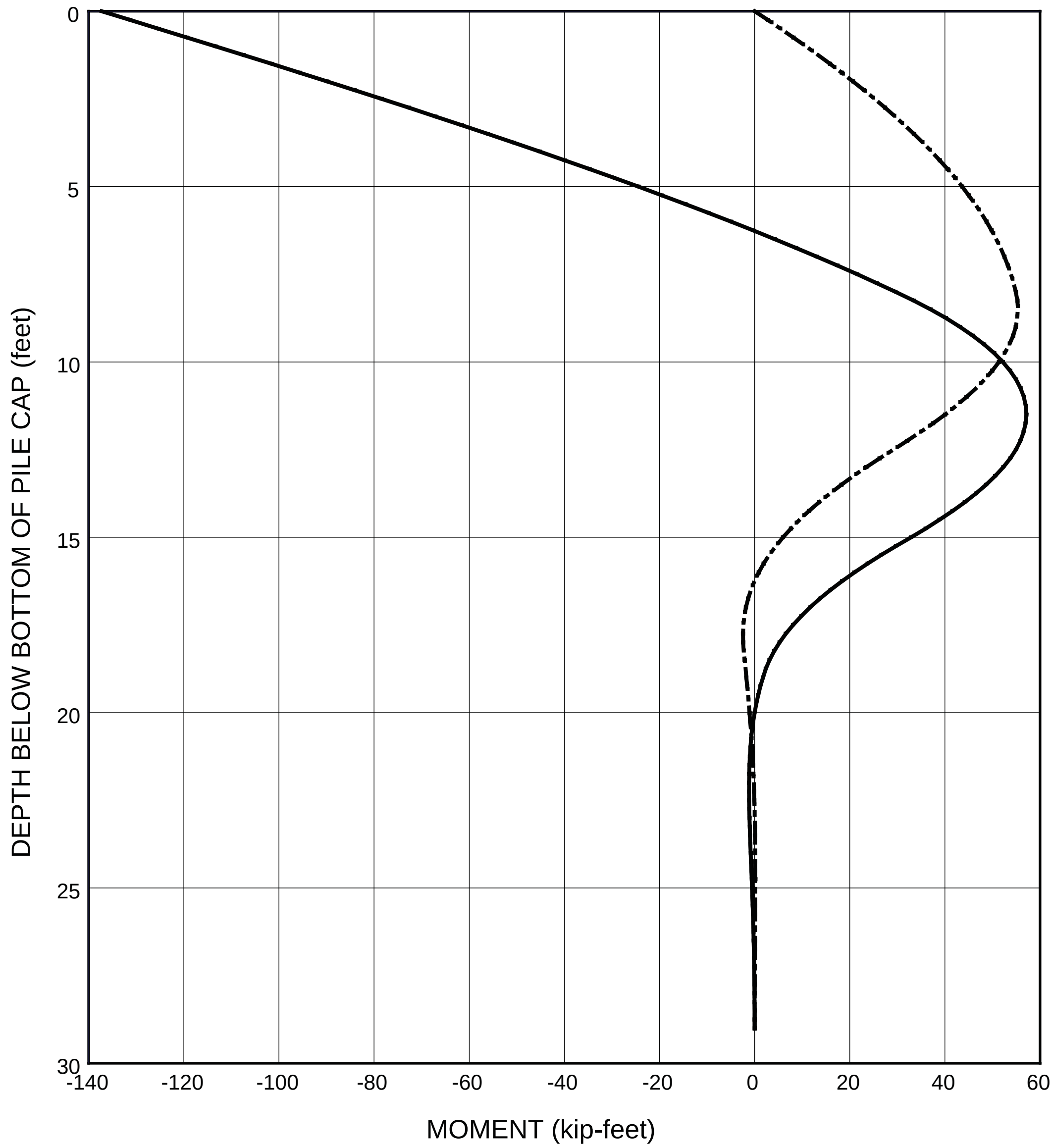
Notes:
1. The profiles shown are for a single 14-inch square prestressed precast (PCPS) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 375 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 79 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DEFLECTION PROFILE
14-INCH SQUARE PSPC CONCRETE PILE
BLOCK 32 - SERVICE AREA

Date 12/13/11	Project No. 750603902	Figure 36



Symbol	Lateral Load	Pile Head Connection
——— (solid line)	24 Kips	Fixed
—·—·— (dash-dot line)	9 Kips	Free

DRAFT

Notes:

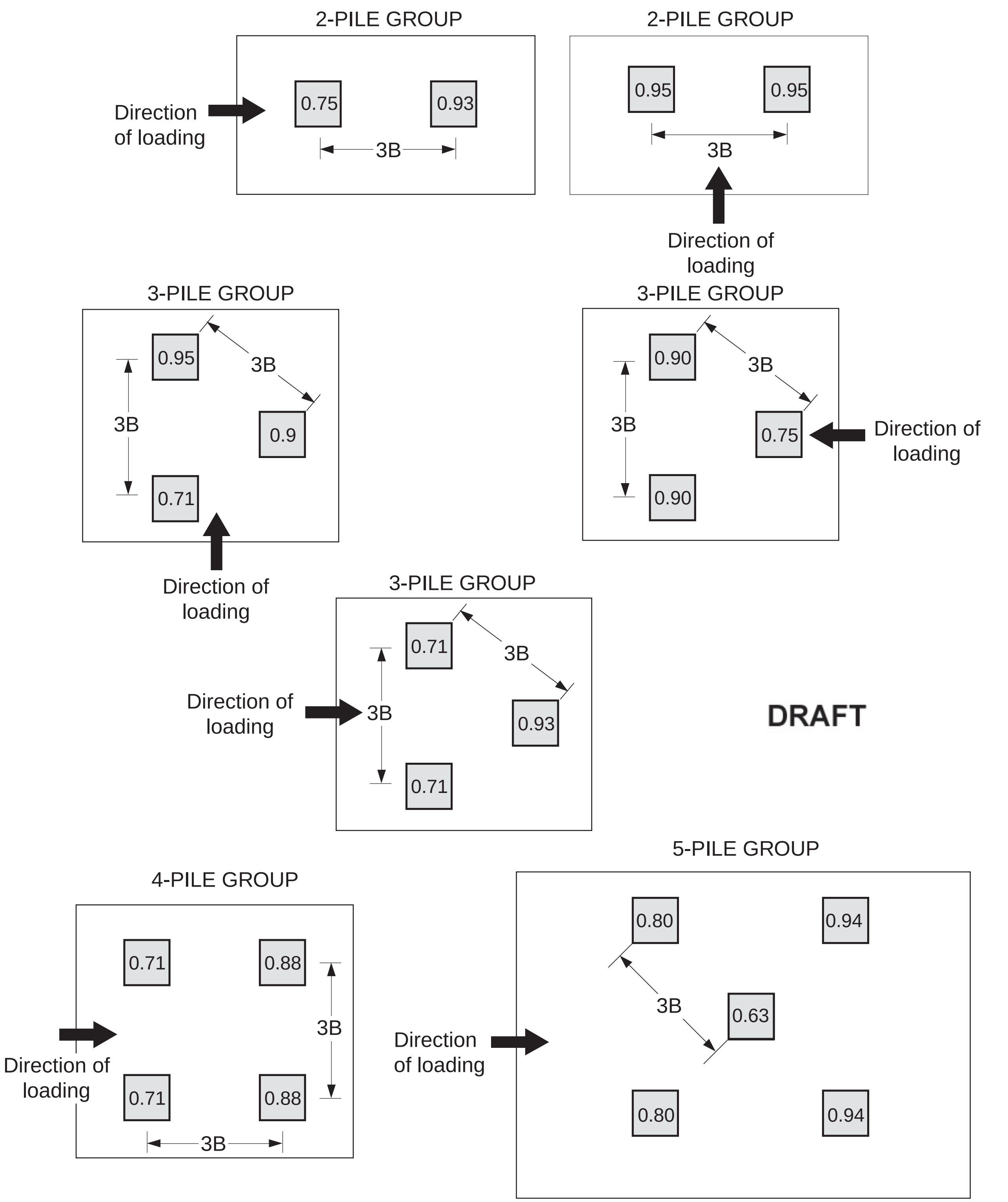
1. The profiles shown are for a single 14-inch square prestressed precast (PCPS) concrete pile with a maximum pile head deflection of 0.5 inch and an axial compressive load of 375 kips.
2. To account for group effects, the lateral load capacity of the pile group should be multiplied by the factor shown in Figure 38a and 38b, 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. Assumes top of pile is at approximately Elevation 79 feet and the lateral load is applied at the top of the pile.
5. Passive resistance of pile caps has not been included.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MOMENT PROFILE
14-INCH SQUARE PSPC CONCRETE PILE
BLOCK 32 - SERVICE AREA

Date 12/13/11	Project No. 750603902	Figure 37



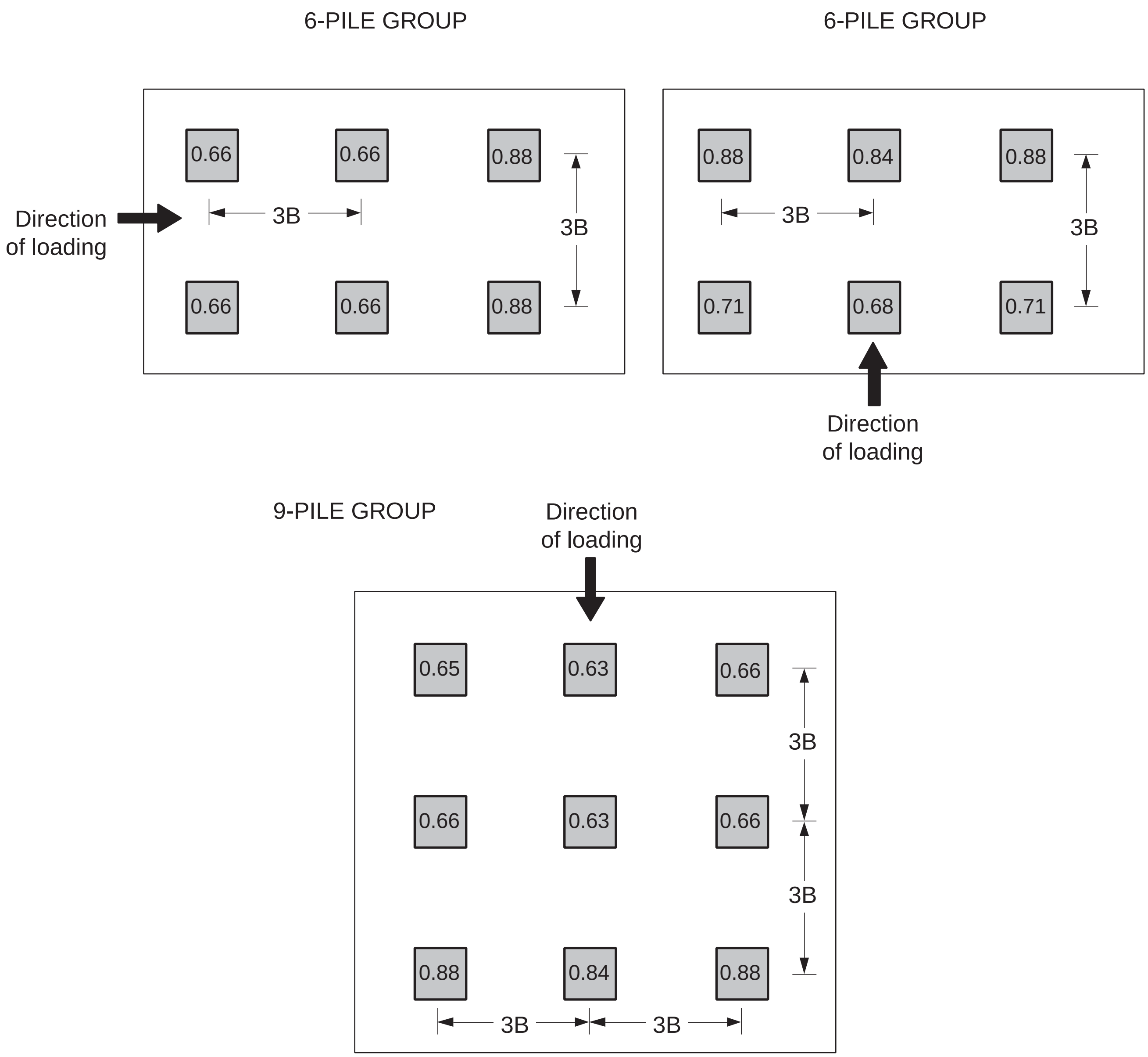
Note: To account for group effects, the lateral load capacity of the pile group should be multiplied by the average of the factors shown; however, the moment profiles (Figures 23, 25, 27, 29, 31, 33, 35 and 37) should be used to check individual piles in a group should be for the unfactored load.

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

LATERAL GROUP REDUCTION FACTORS

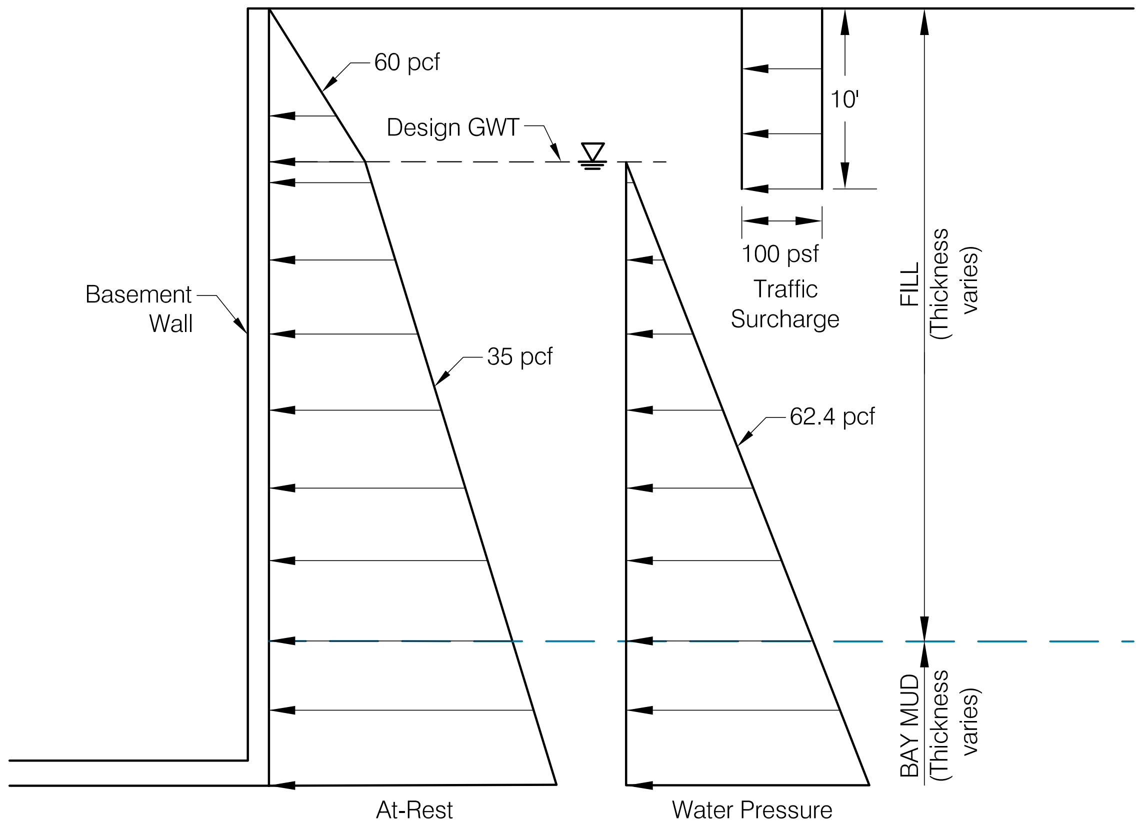
Date 12/20/11	Project No. 750603902	Figure 38a



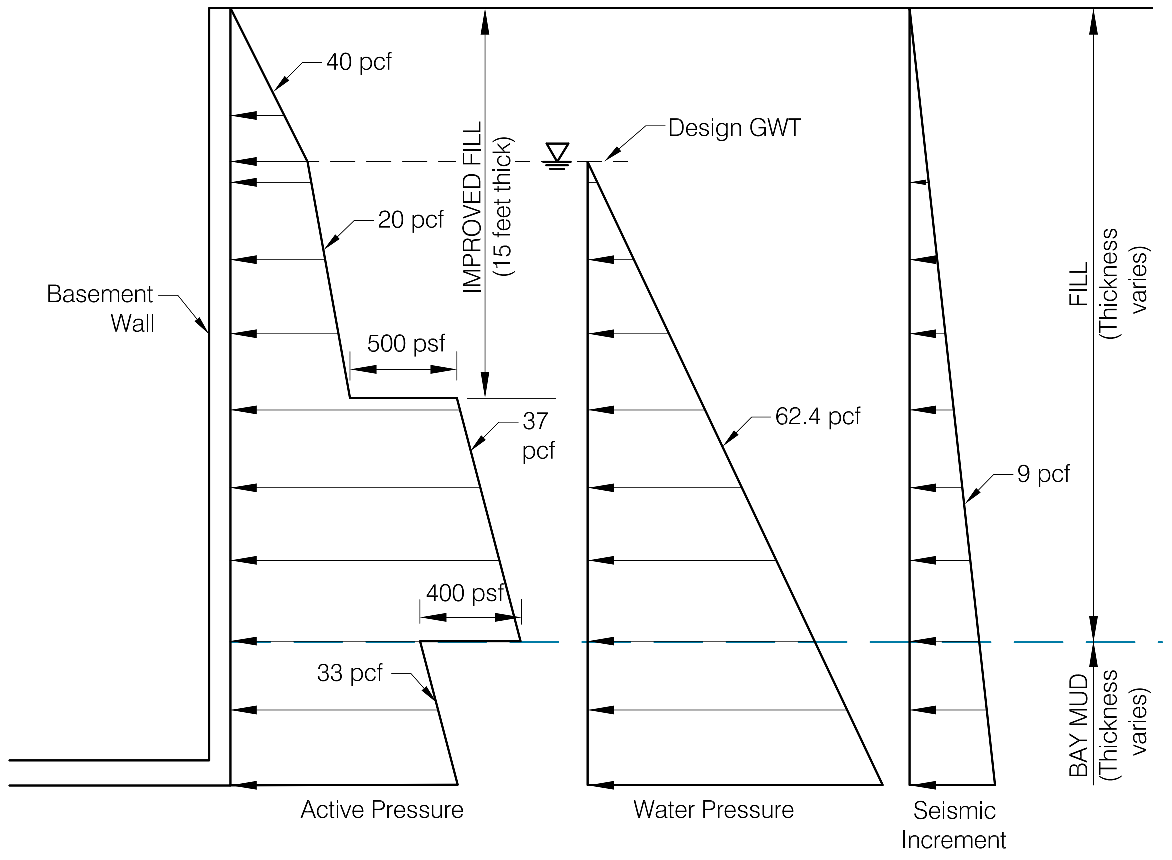
DRAFT

Note: To account for group effects, the lateral load capacity of the pile group should be multiplied by the average of the factors shown; however, the moment profiles (Figures 23, 25, 27, 29, 31, 33, 35 and 37) should be used to check individual piles in a group should be for the unfactored load.

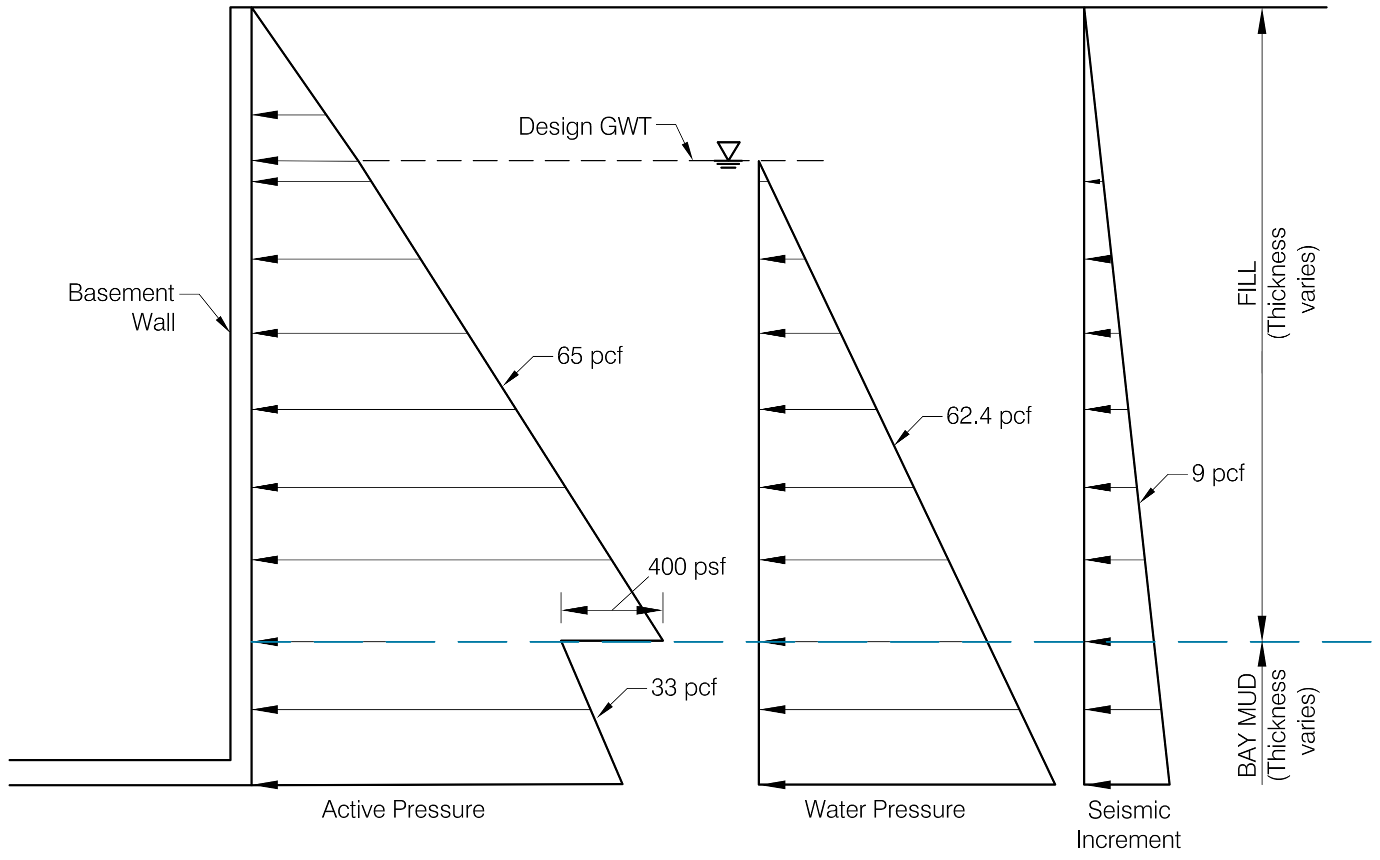
BLOCKS 29-32 **MISSION BAY** San Francisco, California	**LATERAL GROUP REDUCTION FACTORS**
Treadwell&Rollo A LANGAN COMPANY	Date 12/20/11 · Project No. 750603902 · Figure 38b



A. STATIC CONDITION (AT-REST)



B. SEISMIC CONDITION WITH IMPROVED FILL (UPPER 15 FEET)



C. SEISMIC CONDITION WITH UNIMPROVED FILL

Notes:
1. The more critical condition of either at-rest pressure for static conditions or total pressure for seismic conditions should be checked.
2. Where traffic is expected within 10 feet of the walls, a surcharge of 100 psf should be added to the top 10 feet of wall.
3. Design groundwater table (GWT) is Elevation 95 feet.
4. Improved fill assumes the upper 15 feet of fill has been improved to mitigate liquefaction and lateral spreading.
5. The recommended pressures do not include surcharges from adjacent buildings. Surcharge pressure from adjacent buildings should be added to the above shoring pressures.
6. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
7. Thickness of fill and Bay Mud varies throughout site. See boring logs for details.

DRAFT

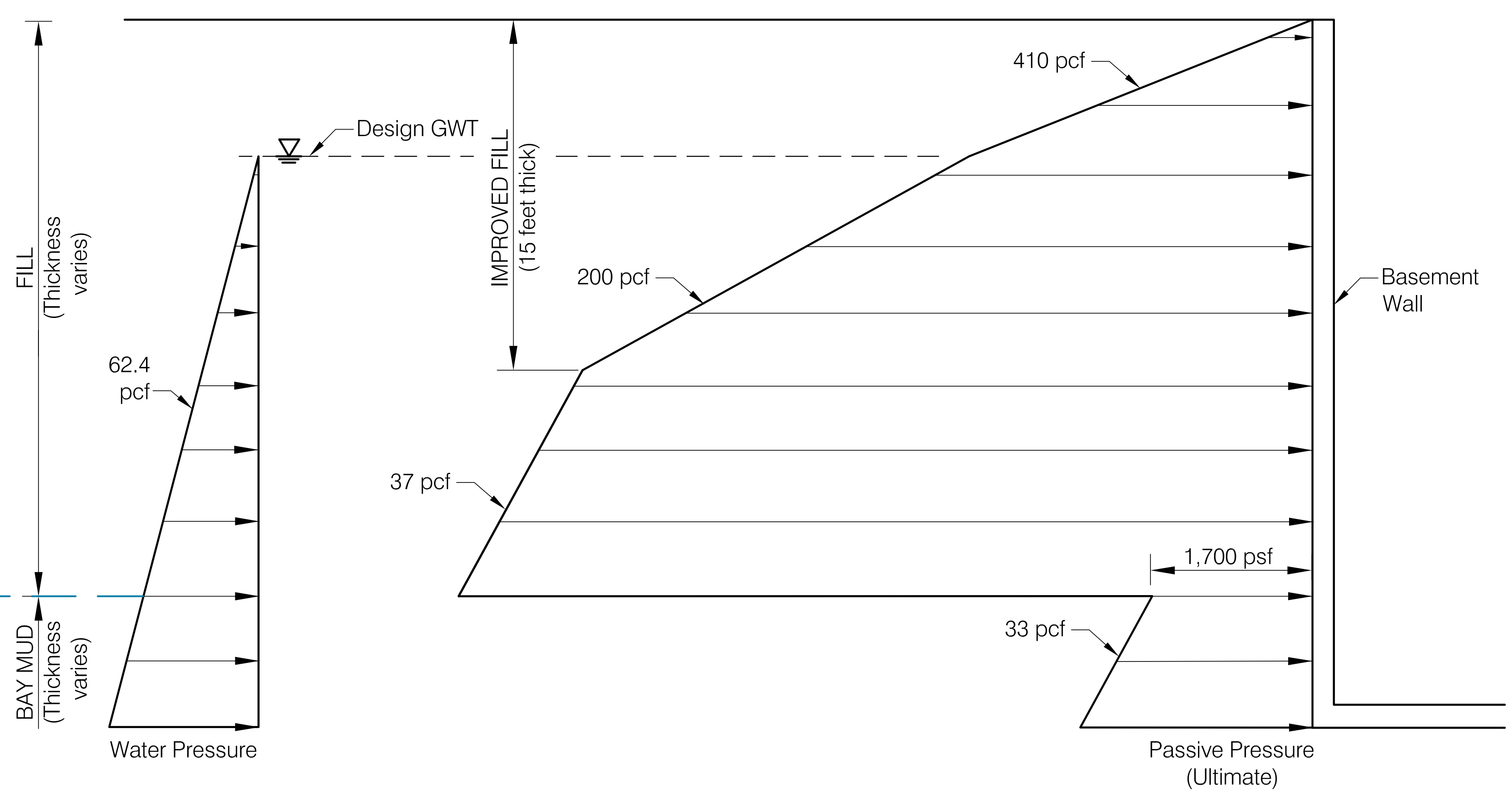
NOT TO SCALE

BLOCK 29-32
MISSION BAY
San Francisco, California

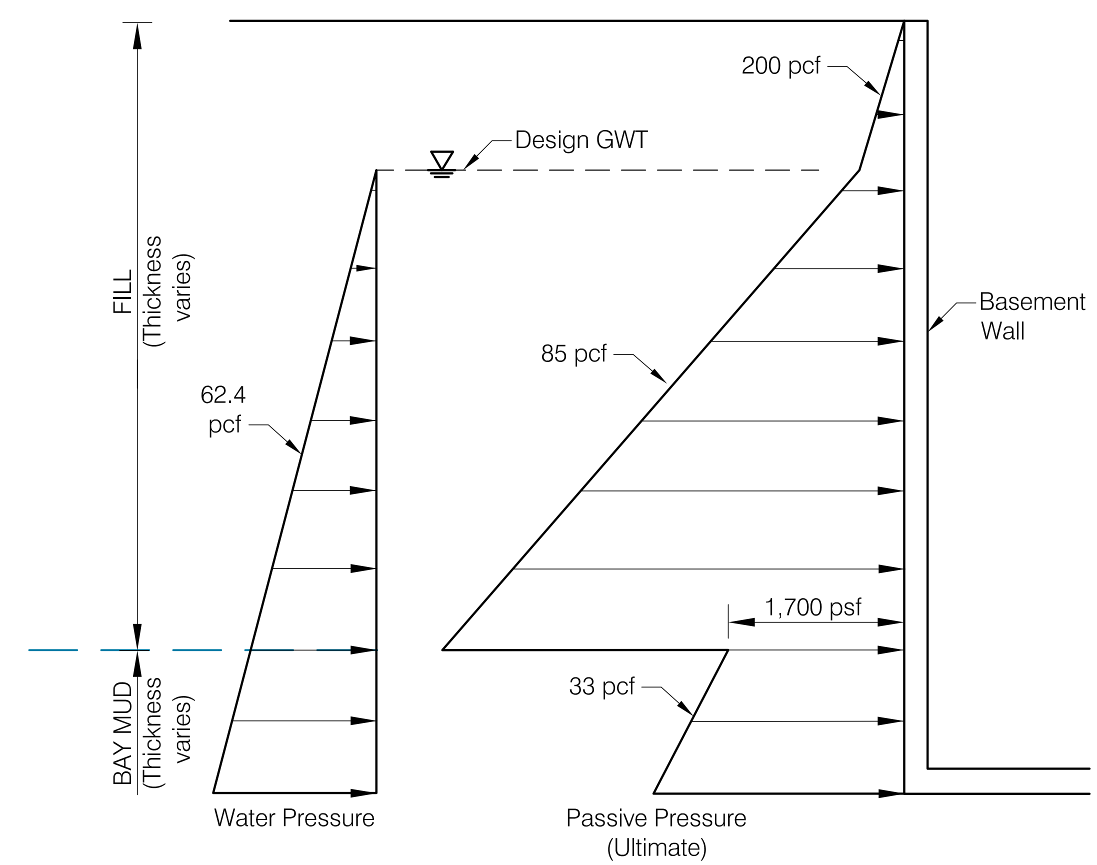
EARTH PRESSURES FOR BASEMENT WALLS

Date 12/16/11 | Project No. 750603902 | Figure 39

Treadwell&Rollo
A LANGAN COMPANY



A. PASSIVE PRESSURE (ULTIMATE) UNDER SEISMIC CONDITION WITH IMPROVED FILL (UPPER 15 FEET)



B. PASSIVE PRESSURE (ULTIMATE) UNDER SEISMIC CONDITION WITH LIQUEFIABLE FILL

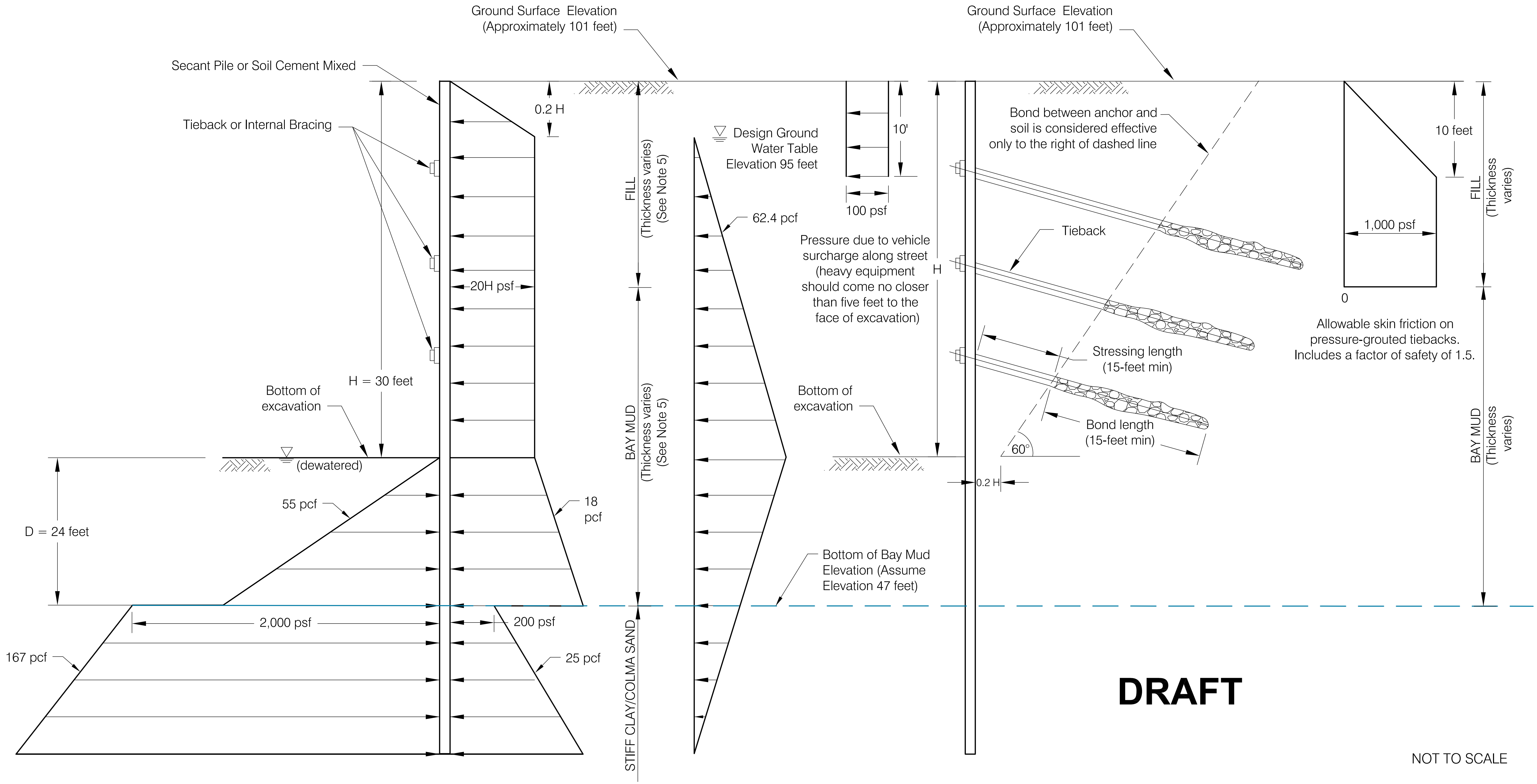
DRAFT

NOT TO SCALE

Notes:
1. Passive pressure presented are ultimate. For active/at-rest pressure, see Figure 39.
2. Design ground water table (GWT) is at Elevation 95 feet.
3. Pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
4. Thickness of fill and Bay Mud varies throughtout site. See boring logs for details.
5. Improved fill assumed the upper 15 feet of fill has been improved to mitigate liquefaction and lateral spreading.

BLOCK 29-32 **MISSION BAY** San Francisco, California		
PASSIVE RESISTANCE **FOR BASEMENT WALLS**		
Date 12/20/11	Project No. 750603902	Figure 40

Treadwell&Rollo
A LANGAN COMPANY



Notes:
1. Passive pressures includes a factor of safety of approximately 1.5.
2. Surcharge pressure due to construction, if any, should be added to the above shoring pressure.
3. The recommended pressures do not include surcharges from adjacent buildings.
   Surcharge pressure from adjacent buildings should be added to the above shoring pressures.
4. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.
5. Thickness of fill and Bay Mud varies throughout site. See boring logs for details.

BLOCK 29-32
MISSION BAY
San Francisco, California

DESIGN PARAMETERS FOR SECANT PILE OR SOIL CEMENT MIXED WALL WITH TIEBACKS OR INTERNAL BRACING

Date 12/19/11	Project No. 750603902	Figure 41

Treadwell&Rollo
A LANGAN COMPANY

\\Langan.com\data\OA\data9\750603902\Cadd Data - 750603902\2D-DesignFiles_JUDY\Geotech\Design Parameters Secant Pile.dwg 12/20/11

APPENDIX A

Boring Logs

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-1 PAGE 1 OF 5

Boring location: See Site Plan, Figure 2	Logged by: T. Shu
Date started: 6/7/11 — Date finished: 6/8/11	
Drilling method: Rotary Wash	
Hammer weight/drop: 140 lbs./30 inches — Hammer type: Automatic	LABORATORY TEST DATA
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)	

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 101 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	GRAB.			SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, medium dense, moist, fine to coarse sand and gravel, trace brick fragments	FILL						
2–5	S&H	30 25 23	34	SC	CLAYEY SAND with GRAVEL (SC) olive-brown, dense, moist						9.4	127
	SPT	11 10 16	31									
5–8	SPT	6 8 6	17	SP-SC	SAND with CLAY and GRAVEL (SP-SC) olive-brown, medium dense, moist, with brick fragments fines are non-plastic					14.2	8.3	
8–11	SPT	3 4 4	10	SP	SAND (SP) brown, loose, moist, trace silt ▽ (06/07/11, 9:10 a.m.)							
	SPT	1 3 3	7		grades gray							
11–20	GRAB.			SC	CLAYEY SAND (SC) blue-gray, medium dense, wet, trace fine gravel							
	S&H	6 8 8	11		grades with coarse gravel fines are non-plastic					12.1	19.8	
20–30	D&M		200 psi	CH	CLAY (CH) gray, soft, wet, trace shell fragments Consolidation Test, see Figure C-1	BAY MUD	TV PP		1,000 1,250		49.4	72

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Project No.: 750603902 | Figure: A-1a

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	D&M			150 psi	CH	CLAY (CH) (continued) Consolidation Test, see Figure C-2	TV PP		1,000 600		71.6	56
41	D&M			180 psi		Consolidation Test, see Figure C-3	TV PP		900 500		73.2	56
51	D&M			150 psi		Consolidation Test, see Figure C-4	TV PP		1,040 550		65.1	61
58–59						coarse sand observed in cuttings						

BAY MUD (31–60 ft)

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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
60	D&M			150 psi	CL	SANDY CLAY (CL) brown and olive-brown, very stiff, wet	PP		2,500		24.9	101
65–66	S&H		8 18 28	32	CL	CLAY (CL) yellow-brown and olive-brown, hard, wet, with silt	PP		4,000			
70–71	SPT		10 17 26	52	SC	CLAYEY SAND (SC) red-brown, very dense, wet, fine-grained						
75–76	S&H		34 50/5"	35/5"	SP-SM	SAND with SILT (SP-SM) red-brown, very dense, wet, fine-grained, trace clay						
80	S&H		50/6"	35/6"	SP-SM							
81	SPT		26 50/6"	60/6"	SP-SM							
85–86	S&H		30 50/6"	35/6"	SP-SM	brown, with some clay				8.9	22.9	107
90												

COLMA FORMATION (70 to 90 feet)

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Project No.: 750603902

Figure: A-1c

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-1 PAGE 4 OF 5

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
90–91	SPT	37 45 50/4"	114/ 10"	SP-SM	SAND with SILT (SP-SM) (continued)						
95–96	SPT	23 39 50	107	SP-SM	yellow-brown, with trace red speckling				9.9	22.3	
100–101	SPT	26 50/6"	60/6"	SP-SM	olive-brown						
105–106	S&H	6 9 15	17	CH	CLAY (CH) gray, very stiff, wet, dark brown sandy silt veins, trace brown organics	PP		2,250		59.0	69
115–116	S&H	8 17 25	29	CH	grades blue-gray, trace rock and gravel fragments						

COLMA FORMATION (90–105 ft)

OLD BAY CLAY (105–120 ft)

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Project No.: 750603902

Figure: A-1d

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
						CLAY (CH) (continued)	OLD BAY CLAY						
121													
122													
123					CH								
124													
125													
126						CLAY with SAND (CL) olive-brown, very stiff, wet, coarse sand							
127	S&H		14 23 31	38									
128					CL								
129													
130	S&H		50/4"	35/4"		SERPENTINITE olive-green and gray, intensely fractured, low hardness, weak, deep weathering	BEDROCK						
131													
132													
133						SHALE olive-green to gray, crushed to intensely fractured, soft, plastic, deep weathering							
134													
135	SPT		28 50/4"	60/4"									
136													
137													
138													
139													
140													
141													
142													
143													
144													
145													
146													
147													
148													
149													
150													

Boring terminated at a depth of 135.8 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10 feet during drilling.
PP = pocket penetrometer; TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902

Figure: A-1e

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-2 PAGE 1 OF 4

Boring location: See Site Plan, Figure 2		Logged by: T. Shu
Date started: 6/8/11	Date finished: 6/9/11	
Drilling method: Rotary Wash		
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Automatic	
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 100.6 feet[2]		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					6-inches Asphalt Concrete							
1–2	GRAB.			SM	SILTY SAND with GRAVEL (SM) brown to olive-brown, medium dense, slightly moist, fine and coarse sand and gravel, trace brick fragments	FILL						
2–3	SPT	2 8 10	22									
5–6	SPT	1 2 2	5		grades loose fines: LL = 24, PL = 19, PI = 5, see Figure C-38 trace pieces of sheet metal					23.6	17.4	
7–8	SPT	0 0 50/5"	60/5"		grades clayey, serpentinite fragments in shoe ▽ (06/08/11, 3:30 PM)							
9–10	GRAB.				driller reports boulders							
13–15	SPT	4 3 4	8	GP	GRAVEL with SAND (GP) gray-green, loose, wet, fine-to coarse gravel and gravel size serpentinite							
20–21	SPT	3 7 6	16	GP	GRAVEL with SAND (GP) black and blue-gray, medium dense, wet, with cobbles							
25–26	SPT	6 2 3	6		grades with clay							
28–30	DIST			CH	CLAY (CH) gray, soft, wet, trace white shell fragments and fine sand	BAY MUD						

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Project No.: 750603902	Figure: A-2a

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
31	D&M		80 psi	CH	CLAY (CH) (continued) trace red-brown peat Consolidation Test, see Figure C-5	TV		1,000		63.0	62
38–39	D&M		200 psi	CH	Consolidation Test, see Figure C-6	TV TxUU	 2,700	1,200 1,290		64.1 66.0	60 59
43	D&M		100 psi	CH	BAY MUD (30–43 ft)						
43–44	S&H	12 32 43	53	SM	SILTY SAND (SM) blue-gray to olive-gray, very dense, wet						
45–46	SPT	16 23 36	71	SM							
49–51	SPT	6 11 10	25	SC	CLAYEY SAND (SC) gray-brown, medium dense, wet, some dark yellowish-brown clasts of silty sand up to 1/8-inch in diameter						
53–56	S&H	6 12 18	21	CL	CLAY with SAND (CL) yellow-brown with gray mottling, very stiff, moist, low to medium plasticity	PP TxUU	 3,700	3,100 3,160		 21.4	 108

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Project No.: 750603902
Figure: A-2b

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
						CLAY with SAND (CL) (continued)						
61					CL							
62						SANDY CLAY (CL) dark yellow-brown						
63												
64					CL							
65			13									
66	S&H		27 41	47		SAND with CLAY (SP-SC) red-brown, dense, moist						
67												
68					SP-SC							
69												
70			60			SAND with SILT (SP-SM) red-brown to brown, very dense, moist, fine-grained sand, trace clay						
71	SPT		30 40	84								
72												
73												
74					SP-SM							
75			15			grades with dark brown clasts of sand						
76	SPT		25 38	76						8.2	24.7	
77												
78						CLAYEY SAND (SC) yellow-brown, very dense, moist						
79												
80			18		SC							
81	S&H		43 50/3"	65/8"		SAND with SILT (SP-SM) yellow-brown to reddish-brown, very dense, wet, fine-grained sand						
82												
83												
84												
85			23									
86	SPT		36 48	101	SP-SM					8.1	23.4	
87												
88												
89						grades clayey, trace coarse sand						
90												

COLMA FORMATION

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PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-2 PAGE 4 OF 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
91	S&H		14 25 29	38	CL	CLAY (CL) blue-gray, hard, wet, with silt and trace gravel	OLD BAY CLAY	PP		2,750			
92–99													
						grades yellowish-brown with coarse gravel							
100	S&H		50/ 5.5"	35/ 5.5"		SERPENTINITE gray and olive-green, intensely fractured, low hardness, weak, deep weathering	BEDROCK						
101–104													
105	SPT		50/1"	60/1"									
106–120													

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Boring terminated at a depth of 105.1 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 8.5 feet during drilling.
PP = pocket penetrometer; TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Figure: A-2d

PROJECT:	**BLOCKS 29-32** **MISSION BAY** San Francisco, California	**Log of Boring B29-3**
		PAGE 1 OF 5

Boring location:	See Site Plan, Figure 2		
Date started:	8/23/11	Date finished:	8/24/11
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)		

Logged by: M. McKee

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 101.5 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
0–2.5				SM	SILTY SAND (SM) brown and gray, loose, dry, trace plastic debris, concrete and sandstone fragments	FILL						
3–3.5	S&H	15 26 35	43	SM	grades to dense	FILL						
3.5–6	SPT	15 18 16	41	SM	SILTY SAND with GRAVEL (SM) brown to dark brown, dense, moist, trace brick fragments	FILL						
6–8	DIST			CL	SANDY CLAY with GRAVEL (CL) dark gray and olive, stiff, moist, trace serpentinite and concrete fragments	FILL						
8–13	S&H	9 50/6"	35/6"	SC	CLAYEY SAND with GRAVEL (SC) brown and blue-gray, very dense, moist, serpentinite fragments (8/24/11; 8:10 AM) grades with brick sandstone and olive serpentinite fragments	FILL						
13–20	S&H	3 4 5	6	CH	SANDY CLAY with GRAVEL (CH) olive gray, medium stiff, wet	FILL						
20–30	D&M	100 psi			CLAY (CH) gray, soft, wet, trace white shells	BAY MUD	TV		660			

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Project No.:	Figure:
750603902	A-3a

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
30–54					CH	CLAY (CH) (continued) — BAY MUD						
35–36	D&M		140 psi				TV		640			
45–46	D&M		150 psi				TV		760			
54–55					CL	SANDY CLAY (CL) blue-gray, very stiff, hard, wet						
55–56	S&H		12 36 38	52	SP-SC	SAND with CLAY (SP-SC) gray to brown, very dense, wet						
56–60					SP-SC	SANDY CLAY (CL) gray-brown, very stiff, wet						

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Project No.: 750603902

Figure: A-3b

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-3 PAGE 3 OF 5

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
61	SPT		4 7 10	20	CL	CLAY (CL) olive, very stiff, wet, trace fine sand	TV		2,400			
65–75					SC	CLAYEY SAND (SC) red-brown, dense, wet, fine grained						
71	S&H		17 28 38	46								
76	SPT		22 32 38	84	SP-SM	SAND with SILT (SP-SM) brown, very dense, wet, fine grained, trace clay						
81	SPT		25 35 30	78		some clay						
86	SPT		11 30 43	88		grades olive-brown trace red-brown sand seams, trace clay						

COLMA FORMATION

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Project No.: 750603902

Figure: A-3c

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

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
						SAND with SILT (SP-SM) (continued)						
91												
92	S&H		30 50/6"	35/6"								
93–97					SP-SM	COLMA FORMATION						
98	SPT		30 50/6"	60/6"		red-brown, grades less silt						
99–103												
103					CH	CLAY (CH) blue-gray, very stiff, wet, trace organics						
104–105	S&H		11 15 25	28		OLD BAY CLAY	TV		2,250			
106–110												
110					CH	SANDY CLAY with GRAVEL (CH) blue-gray, hard, wet, with trace chert fragments and carbonate deposits						
111–114												
115–116	S&H		15 25 32	40			TV		4,300			
117–120												

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Project No.: 750603902

Figure: A-3d

PROJECT: BLOCKS 29-32 MISSION BAY San Francisco, California	**Log of Boring B29-3** PAGE 5 OF 5

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 with GRAVEL (CH) (continued)						
121	S&H		15									
122			25 30	38		with chert fragments up to 2.5-inch diameter	TV		4,500			
123												
124					CH							
125												
126												
127												
128						GREENSTONE gray to dark gray with iron-oxide staining, crushed to intensely fractured, low hardness, friable to weak, deep weathering (BEDROCK)						
129												
130	S&H SPT		50/2" 60/3"	35/2" 72/3"								
131												
132												
133												
134												
135	SPT		40 50/4"	60/4"		no iron-oxide staining						
136												
137												
138												
139												
140												
141												
142												
143												
144												
145												
146												
147												
148												
149												
150												

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Boring terminated at a depth of 135.9 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 11.5 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902 | Figure: A-3e

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-4

Boring location:	See Site Plan, Figure 2	Logged by:	M. McKee
Date started:	8/22/11	Date finished:	8/23/11
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST), Dames & Moore (D&M)		

Ground Surface Elevation: 100.6 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					2.5-inch Asphalt Concrete (AC)	FILL						
1					11.5-inch Aggregate Base (AB)							
1–3	S&H (2)	13 50/3"	35/3"	SM	SILTY SAND (SM) brown, medium dense, dry to moist, trace brick fragments							
3–5.5				SP-SM	SAND with SILT and GRAVEL (SP-SM) brown and gray, medium dense, (blow count influenced by gravel), moist, with gravel size serpentinite fragments and brick black sand and brick fragments at 5 to 5.5 feet							
5.5–12	S&H (6) SPT (7.5)	4 10 8 3 4 12	13 19	SC/CL	CLAYEY SAND (SC)/ SANDY CLAY (CL) yellow brown mottled gray-brown, medium dense/ stiff to very stiff, moist grades dark olive/green, with abundant serpentinite fragments, trace shale fragments ▽ (08/22/11, 10:00 AM) grades with angular fragments of black graywacke sandstone and serpentinite							
12–15				CH	CLAY (CH) gray, medium stiff, wet, with fine sand and trace rock fragments							
15–30	D&M (17.5)	90 psi		CH	CLAY (CH) gray, soft, wet grades with some shells (28)	BAY MUD	TV TxUU	 1,550	660 680		 54.7	 67

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Project No.: 750603902

Figure: A-4a

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-4

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY (CH) (continued)	BAY MUD						
31													
32	D&M		140 psi					TV		740			
								TxUU	2,075	800		45.8	71
33													
34													
35													
36													
37													
38													
39													
40													
41													
42					CH								
43													
44													
45													
46	D&M		150 psi					TV		770			
								TxUU	2,600	740		47.1	72
47													
48						shells at 48 to 50 (discontinuous)							
49													
50													
51													
52													
53						grades with increase fine sand content							
54													
55						CLAYEY SAND (SC) brown to blue-gray, very dense, wet, fine grained							
56	S&H		21 30 43	51	SC								
57	SPT		20 27 16	52		CLAY with SAND (CL) light olive-brown, hard, wet							
58					CL								
59													
60													

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Project No.:	Figure:
750603902	A-4b

Log of Boring B29-4

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
61–65	S&H	63–64.5	12 22 38	42	CL	CLAY with SAND (CL) (continued) grades brown-gray grades olive brown mottled red-brown, with increased sand content	TxUU	3,750	4,280		16.4	115
66–72	S&H	69–70.5	15 26 30	39	SC	CLAYEY SAND (SC) yellow-brown, dense, wet, trace reddish-brown speckling						
73–79	SPT	75–76.5	22 36 37	88	SP	SAND (SP) brown, very dense, wet						
80–90	SPT	81–82.5	22 30 40	84	SP-SM	SAND with SILT (SP-SM) brown, very dense, wet				8.2	23.6	
	S&H	87–88	32 50/6"	35/6"								

COLMA FORMATION (66–90 ft)

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Project No.: 750603902
Figure: A-4c

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
91–92					SP-SM	SAND with SILT (SP-SM) (continued)	COLMA FORMATION						
93–94	SPT		25 50/6"	60/6"		grades olive-brown							
95–98													
99–100	SPT		22 50/6"	60/6"		grades with up to 1/4-inch thick yellow-brown and black clayey sand seams							
101–104					CH	CLAY (CH) gray, very stiff, wet, with trace organics and medium sand	OLD BAY CLAY						
105–106	S&H		7 14 21	25									
107–108													
109–110	ST		200 to 160 psi										
111–114													
115–116					CH	CLAY with SAND (CH) gray-brown, very stiff, wet, trace gravel (rock fragments)							
117						SERPENTINITE dark olive-green and gray, crushed to intensely fractured, low hardness, friable to weak, deep weathering, scattered zones of oxidation	BEDROCK						
118–119	SPT		35 50/5"	60/5"									
120													

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Project No.: 750603902

Figure: A-4d

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B29-4

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SERPENTINITE (continued) — BEDROCK						
121												
122	SPT		35 50/5"	60/5"								
123												
124												
125												
126												
127												
128												
129												
130												
131												
132												
133												
134												
135												
136												
137												
138												
139												
140												
141												
142												
143												
144												
145												
146												
147												
148												
149												
150												

DRAFT

Boring terminated at a depth of 122.9 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 9.4 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902
Figure: A-4e

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-5 PAGE 1 OF 4

Boring location:	See Site Plan, Figure 2
Date started:	8/24/11
Date finished:	8/25/11
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by:	T. Shu

LABORATORY TEST DATA

Ground Surface Elevation: 102.2 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
0–1	GRAB.			SP	SAND with GRAVEL (SP) olive brown to brown, medium dense, dry to moist, fine to coarse sand and gravel, trace brick fragments, trace silt and clay (FILL)						
2–3	S&H	16, 15, 15	21		trace serpentinite fragments						
3.5–5	SPT	8, 12, 21	40		very dense, grades with clay						
9					grades green-gray						
10–11.5	SPT	3, 2, 2	5	SP-SC	▽ (08/24/11, 8:55 AM) SAND with CLAY and GRAVEL (SP-SC) olive-gray, loose, wet, medium to coarse sand and gravel, serpentinite fragments						
15–16.5	S&H	5, 5, 7	8	GP-GC	GRAVEL with CLAY (GP-GC) blue-green to green-gray, loose, wet, coarse gravel size weathered serpentinite fragments					11.5	126
16.5–18	SPT	3, 2, 5	8								
20–21.5	S&H	5, 1, 0	1	CH	CLAY (CH) gray, soft, wet, trace shell fragments (BAY MUD)						
22–23.5	D&M	100 psi			Consolidation Test, see Figure C-7	TV		600		57.5	65
27–28.5	D&M	190 psi			grades with trace fine brown sand	TV		760			

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Project No.: 750603902

Figure: A-5a

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

Log of Boring B29-5

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
30–34				CH	CLAY (CH) (continued)						
35	D&M	250 psi		CH	Consolidation Test, see Figure C-8	TV		780		80.0	51
39				CH	BAY MUD (30–49 ft)						
42	D&M	200 psi		CH	Consolidation Test, see Figure C-9	TV		600		74.3	55
49	D&M	200 psi		CH							
49.5–52.5	SPT	10 15 17	38	SP-SC	SAND with CLAY (SP-SC) olive gray, dense, wet, fine grained						
52.5–55.5				SP	SAND (SP) olive brown, dense, wet, fine grained, trace clay						
55.5–60	SPT	6 9 10	23	CL	CLAY (CL) light olive brown, very stiff, wet, trace fine sand and silt						

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Project No.: 750603902

Figure: A-5b

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

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
61	S&H		18 25 29	38	SP-SC	SAND with CLAY (SP-SC) brown, dense, wet, fine grained, with trace clay					16.3	121.1
62	SPT		10 15 25	48		grades with less clay						
63												
64												
65–66	SPT		14 18 19	44								
67–69												
70–71	SPT		26 29 40	83		very dense				10.4	23.3	
72–74												
75–76	SPT		16 22 17	47	SC	CLAYEY SAND (SC) olive brown, dense, wet						
77–79					SP-SM	SAND with SILT (SP-SM) olive brown, very dense, wet						
80–81	S&H		19 41 50/5"	64/ 11"								
82–84												
85–86	SPT		27 34 40	89								
87–90												

COLMA FORMATION

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PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-5 PAGE 4 OF 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
90–95	SPT		27 50/6"	60/6"	SP-SM	SAND with SILT (SP-SM) (continued)	COLMA FORMATION						
95–100	SPT		7 12 15	32	CL	CLAY (CL) light olive gray, hard, wet	OLD BAY CLAY						
100–105.3	S&H		50/6"	35/6"		SHEARED SHALE olive-brown and gray, intensely fractured, low hardness, friable to weak, deep weathering clay interbeds, oxidized fracture surfaces, quartz veins	BEDROCK						
105	SPT		50/3"	60/3"									

Depth scale: 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120

DRAFT

Boring terminated at a depth of 105.3 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10.5 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902 | Figure: A-5d

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-6

Boring location: See Site Plan, Figure 2
Date started: 8/22/11 Date finished: 8/23/11
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)

Logged by: T. Shu

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 101 feet[2]		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					4-inch Asphalt Concrete (AC)	FILL						
1				SM	SILTY SAND with GRAVEL (SM) gray-brown, moist, fine to coarse sand							
2	GRAB.				6-inch thick Concrete Slab							
3	S&H	6 22 23	32	SP	SAND with GRAVEL (SP) brown to yellow-brown, dense, moist, fine to medium grained, trace asphalt and brick							
5–6	S&H	9 14 7	15	SP	SAND with GRAVEL (SP) gray-green, medium dense, moist, fine to coarse grained, with angular gravel-size serpentinite fragments, trace clay							
7	SPT	2 7 10	20	SC	CLAYEY SAND (SC) gray-brown, medium dense, moist, medium grained, trace silt and gravel (08/22/11, 10:45 AM)					38.2		
10–11	S&H	3 5 4	6	SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray-brown, loose, wet, fine to coarse grained, serpentinite fragments, angular gravel fines: LL = 41, PL = 20, PI = 21, see Figure C-38					11.1	17.4	105
15–16	S&H	2 4 8	8	SC	CLAYEY SAND (SC) olive-gray, loose, wet, fine grained, trace serpentinite and gravel fragments							
19–20	D&M	50 psi		CH	CLAY (CH) gray, soft, wet, trace shell fragments	BAY MUD						
24–25	D&M	100 psi					TV		800			
27–28	D&M	90 psi					TV		400			
30												

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Project No.: 750603902 Figure: A-6a

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
						CLAY (CH) (continued)						
31												
32												
33												
34												
35												
36	D&M		100 psi				TV		600			
37												
38												
39												
40					CH	BAY MUD						
41												
42												
43												
44												
45												
46	D&M		100 psi				TV		600			
47												
48												
49												
50						SAND with CLAY (SP-SC) yellow-brown, dense, wet, fine grained						
51												
52												
53												
54												
55					SP-SC							
56	S&H		19 25 28	37								
57						grades with increase clay						
58												
59												
60												

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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
61	S&H		5 8 10	13	CL	CLAY (CL) olive mottled yellow brown, stiff, wet, trace fine sand						
68						grades with less clay, increase sand content						
69-70	SPT		15 23 28	61	SP-SM	SAND with SILT (SP-SM) brown, very dense, wet, fine grained						
75-76	SPT		22 22 31	64						10.8	24.4	
80-81	SPT		22 27 40	80								
85-86	SPT		2 15 41	67	CL	CLAY with SAND (CL) olive-gray, hard, wet				77.5	35.1	
86-90					SP	SAND (SP) brown, very dense, wet, fine						

COLMA FORMATION (69 to 90 feet)

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A LANGAN COMPANY

Project No.: 750603902

Figure: A-6c

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-6 PAGE 4 OF 5

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Formation
90–91	SPT	30 28 50	94	SP	SAND (SP) (continued)	COLMA FORMATION
95–96	SPT	30 50/6"	60/6"	SP	grades olive-brown	COLMA FORMATION
99–105	S&H	13 19 23	29	CH	CLAY (CH) gray, very stiff, wet	OLD BAY CLAY
105–115				CL	SANDY CLAY (CL) dark blue-green to gray-black, hard, wet, abundant greenstone fragments and gravel	
110–111.5	S&H	12 25 35	42	CL		
115–120	S&H	31 50/6"	35/6"		GREENSTONE gray and green-gray, crushed, low hardness, weak, deep weathering with scattered fragments of more competent greenstone	BEDROCK

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

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Project No.: 750603902 | Figure: A-6d

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B29-6

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
121	S&H		43 50/2"	35/2"		GREENSTONE (continued)	BEDROCK						
122–124													
125	SPT		50/5"	60/5"		SHALE gray, intensely fractured, low hardness, friable to weak, deep weathering							
126–129													
130	SPT		50/1"	60/1"									
131–150													

DRAFT

Boring terminated at a depth of 130.1 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902
Figure: A-6e

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-7
		PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Date started: 8/25/11 | Date finished: 8/30/11

Drilling method: Rotary Wash

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

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

Logged by: M. McKee and R. Chew

LABORATORY TEST DATA

Ground Surface Elevation: 100 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				SM	SILTY SAND with GRAVEL (SM) brown, dry, with brick and wood debris	FILL						
3	S&H	14 44 34	55	CL	SANDY CLAY with GRAVEL (CL) dark brown, hard, moist, some concrete, glass and asphalt fragments, with trace roots							
6					▽ with brick fragments (08/26/11, 9:00 AM)							
8	S&H	3 5 3	6	SC	CLAYEY SAND with GRAVEL (SC) brown-gray, loose, wet, with some serpentinite fragments ▽ (08/25/11, 2:50 PM)							
11	GRAB.			CL	SANDY CLAY (CL) black and olive, soft, wet, with some serpentinite fragments							
12–19.5				GP-GC	GRAVEL with SAND and CLAY (GP-GC) black and blue-gray, loose, wet, with gravel size serpentinite							
14–15	SPT	3 3 2	6									
20	GRAB.			GP	GRAVEL (GP) dark blue gray, medium dense to dense, wet, angular serpentinite fragments up to 2.5 inches diameter (size of fragments may be limited by sampler size)							
21	SPT	5 15 10	30									
23	GRAB.											
26.5–30	SPT	1 3 2	6	CH	CLAY (CH) gray, medium sitff, wet	BAY MUD						

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Project No.: 750603902

Figure: A-7a

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
					CH	CLAY (CH) (continued)						
31												
32	D&M		125 psi			trace shell fragments Consolidation Test, see Figure C-10					59.0	64
							TxUU	2,250	620		46.3	72
33–41						BAY MUD						
42							TxUU	2,850	1,500		55.6	65
43	D&M		125 psi			Consolidation Test, see Figure C-11	PP		460		64.7	61
44–47												
47–48					SM	SILTY SAND (SM) olive brown, dense, wet, fine grained						
49	S&H		18 28 31	41	SC	CLAYEY SAND (SC) olive brown, dense, wet, fine grained						
50	SPT		8 13 14	32								
51–53												
54					CL	CLAY (CL) olive mottled orange with black spots, very stiff, wet, trace sand						
56	S&H		10 17 24	29								
57–60												

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Project No.: 750603902 Figure: A-7b

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
61					CL	CLAY (CL) (continued) grades with increase sand content						
62–64						CLAYEY SAND (SC) yellow brown with olive mottling, dense, wet						
65–66	S&H		10 27 28	39	SC							
67–69												
69–71	SPT		24 37 42	95		SAND with CLAY (SP-SC) yellow brown, very dense, wet, fine grained with coarse sand pockets						
72–74					SP-SC							
75–76	SPT		26 50/6"	60/6"		grades light olive brown				10.2	23.2	
77–79												
80–81	S&H		19 39 41	56		SAND with SILT (SP-SM) light olive brown mottled with yellow brown, very dense, wet						
82–84												
85–86	SPT		33 36 50	103	SP-SM	with coarse sand pockets						
87–90												

COLMA FORMATION

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Treadwell&Rollo A LANGAN COMPANY

Project No.: 750603902

Figure: A-7c

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
90–95	SPT		26 50/6"	60/6"	SP-SM	SAND with SILT (SP-SM) (continued) — COLMA FORMATION						
95–101	S&H		6 17 21	27	CH	CLAY (CH) olive gray to blue gray, very stiff, wet, trace sand and fine gravel — OLD BAY CLAY						
101–107.5	SPT		50/ 3.5"	60/ 3.5"		GREENSTONE dark gray brown with oxidation staining, crushed to intensely fractured to crushed, low hardness, friable, deep weathering to soil-like consistency — BEDROCK						
105	SPT		100/ 3"	120/ 3"								
107	SPT		50/6"	60/6"								

Depth scale: 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120

DRAFT

Boring terminated at a depth of 107.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 9.5 and rose up to 6.6 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B29-8 PAGE 1 OF 4

Boring location: See Site Plan, Figure 2		Logged by: R. Chew and M. McKee
Date started: 8/30/11	Date finished: 8/31/11	
Drilling method: Rotary Wash		
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Automatic	LABORATORY TEST DATA
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)		

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 101.2 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
0–1				CL	SANDY CLAY (CL) gray-brown, with yellow brown speckling, stiff, moist, with concrete, asphalt and brick fragments	FILL						
3	S&H	6 9 9	13									
5.5	S&H	4 10 11	15		very stiff		PP TV		7,500 3,600			
10.5	S&H	5 26 26	36	SM	SILTY SAND with GRAVEL (SM) gray, dense, moist, with some serpentinite and concrete fragments up to 3 inches diameter							
14				GP-GC	GRAVEL with CLAY and SAND (GP-GC) blue-gray and black, medium dense, wet							
15.5	SPT	5 5 5	12									
20.5	SPT	5 6 7	19		angular							
25					grades with increase clay							
25.5	SPT	2 3 3	7									
26				CH	CLAY (CH) gray, soft, wet, some shells	BAY MUD						

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

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Project No.: 750603902	Figure: A-8a

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31–32	D&M		180 psi		CH	CLAY (CH) (continued); trace peat						
32–33					SP-SC	SAND with CLAY (SP-SC) gray, loose, wet, trace shell fragments, medium grained						
33–43					CH	CLAY (CH) gray, soft, wet						
37–38	D&M		160 psi		CH		TV		1,000			
43–52					SP-SM	SAND with SILT (SP-SM) brown, very dense, wet, fine grained, trace clay, weak cementation						
45	S&H		10 36 40	53								
46–47	SPT		10 24 44	94								
52–56					CL	CLAY with SAND (CL) brown, hard, wet						
52–53	SPT		9 11 19	36								
56	GRAB.											
56–58.5					CH	SANDY CLAY (CH) olive, hard, wet						
58.5–60					CL	CLAY with SAND (CL) brown, hard, wet						

BAY MUD

DRAFT

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
						CLAY with SAND (CL) (continued)						
61					CL	grades with increase sand						
62												
63	S&H		11 22 29	36		CLAYEY SAND (SC) yellow brown, dense, very wet						
64												
65												
66					SC							
67												
68												
69	S&H		24 41 50/1"	64/7"		very dense						
70	SPT		22 34 50	101		SAND with CLAY (SP-SC) yellow brown and red brown, very dense, wet						
71												
72												
73												
74												
75												
76	SPT		14 21 39	72		grades with gray-brown mottling, fine sand						
77												
78												
79												
80					SP-SC							
81	SPT		17 40 36	91		grades olive and with trace, angular chert fragments up to 1/8- to 1/4-inch diameter						
82												
83												
84												
85												
86	SPT		36 50/6"	60/6"								
87												
88												
89												
90					CH	CLAY with SAND (CH)						

COLMA FORMATION

DRAFT

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
91	S&H		11 32 40	50	CH	CLAY with SAND (CH) (continued) green-gray, hard, wet, high plasticity, trace fine gravel						
92					CL	SANDY CLAY (CL) brown, hard, wet, trace fine sandstone and serpentinite gravel fragments						
93–100						SERPENTINITE blue-gray and blue-green, crushed to intensely fractured, soft, friable to weak, deep weathering to soil-like consistency, somewhat oxidized (BEDROCK)						
95	SPT		50/5"	60/5"								
100	SPT		50/4"	60/4"								
101–120												

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Boring terminated at a depth of 100.3 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	**Log of Boring B30-1** PAGE 1 OF 5

Boring location: See Site Plan, Figure 2

Logged by: L. Splitter

Date started: 5/6/07 — Date finished: 5/6/07

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches — Hammer type: Rope and Cathead

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

MATERIAL DESCRIPTION — Ground Surface Elevation: 100.6 feet[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	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
0–1				2 inches concrete over 6 inches aggregate base							
1–3			SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, with brick fragments	FILL						
3–6	S&H (3)	19	CL-ML	SANDY SILTY CLAY with GRAVEL (CL-ML) olive-gray, very stiff, moist, with brick fragments LL = 26, PI = 5, see Figure C-38	FILL						
5–6	SPT	17									
6–9			SP	SAND (SP) olive, medium dense, moist, with glass and gravel	FILL						
8	SPT	4	SP	gray-brown, very loose, with brick, rock in shoe, blow count low because pushed into clay	FILL						
9–10			CH	CLAY (CH) gray, very soft, wet	BAY MUD						
10–11	S&H	1	CH		BAY MUD						
13–15	ST	0 to 75 psi	CH	gray, trace sand	BAY MUD						
26			CH	shells at 26 feet	BAY MUD						
28–30	ST	0 to 100 psi	CH	blue-gray, soft Consolidation Test, see Figure C-12	BAY MUD	TxUU	1,200	360		58.6 58.1	63 65

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Project No.: 750603902 — Figure: A-9a

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-1 PAGE 2 OF 5

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30	ST			CH	CLAY (CH) (continued) (BAY MUD)						
43–45	ST		0 to 75 psi	CH	gray						
54	S&H		35	SC	CLAYEY SAND (SC) mottled olive-gray and olive, dense, wet, fine-grained sand grades to yellow-brown in sample						
57.5				CL	CLAY (CL) olive, stiff to very stiff, wet,						
59	S&H		8	CL							

Depth scale: 31 – 60 feet

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Project No.: 750603902

Figure: A-9b

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-1 PAGE 3 OF 5

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60	S&H		8	CL	CLAY (CL) (continued) with gray and yellow-brown mottling at 60.5						
62					SANDY CLAY (CL) yellow-brown with gray mottling, hard, wet, trace fine gravel						
65	S&H		35								
67				SP-SC	SAND with CLAY (SP-SC) orange-brown, medium dense, wet						
70	SPT		20								
74					mottled olive and red-brown, very dense						
75	SPT		52								
77				SP	SAND (SP) olive-brown, very dense, wet						
80	SPT		51								
84.5				CL	SANDY CLAY (CL) olive, hard, wet						
85	SPT		31								
86.5				SP-SC	SAND with CLAY (SP-SC) olive-brown, very dense, wet						
90	SPT		86/ 11"						5.6		

COLMA FORMATION (67 to 90 feet)

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Project No.: 750603902

Figure: A-9c

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-1

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Formation	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
90–92.5				SP-SC	SAND with CLAY (SP-SC) (continued)	COLMA FORMATION						
92.5–97	SPT (94–95)		50/5"	SP	SAND (SP) olive-brown, very dense, wet	COLMA FORMATION						
97–106	SPT (99–100.5)		15	CH	CLAY (CH) gray, stiff to very stiff, wet rock fragments in cuttings at 106 feet	OLD BAY CLAY						
106–110.5	SPT (109–109.5)		50/5"		SERPENTINITE intensely fractured, low hardness, weak, moderately weathered	BEDROCK						
110.5–120	SPT (114–115.5)		71		CLAYSTONE intensely fractured, low hardness, plastic, deeply weathered	BEDROCK						

Depth scale: 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120

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Project No.: 750603902

Figure: A-9d

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-1 PAGE 5 OF 5

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
121					CLAYSTONE (continued)						
122					SERPENTINITE intensely fractured, low hardness, weak, little weathered						
123											
124											
125	SPT		56								
126											
127					SHALE/SERPENTINITE crushed, soft, plastic						
128											
129	SPT		50/2"								
130											
131											
132											
133											
134											
135											
136											
137											
138											
139											
140											
141											
142											
143											
144											
145											
146											
147											
148											
149											
150											

BEDROCK

Boring terminated at a depth of 129.2 feet.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902 | Figure: A-9e

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-2

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 5/3/07 | Date finished: 5/3/07

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope and Cathead

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

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 100.4 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
0–1				2 inches asphalt concrete over 12 inches aggregate base							
1–7	S&H (3–4) SPT (5–6)	17 12	SP	SAND with GRAVEL (SP) olive-brown, medium dense, moist, with angular to subangular gravel, traces of brick and Serpentinite fragments higher brick content, trace fines	FILL						
7–10	SPT (8–9)	9	CH	CLAY with SAND and GRAVEL (CH) dark gray, stiff, moist olive clay was observed from cuttings at 8 feet ▽ (5/3/07 at 7:55 am)	FILL						
10–15	S&H (10–11) SPT (12–13)	7 48	SC	CLAYEY SAND with GRAVEL (SC) green-gray, loose, wet, serpentinite fragments LL = 32, PI = 13, see Figure C-38 gray, dense	FILL				17.6	13.0	
15–25	SPT (17–18) SPT (19–20)	13 14	CH	SANDY CLAY with GRAVEL (CH) dark gray, stiff, wet, with angular to subangular gravel, and shale fragments	FILL						
25–30	ST (29–30)	100 psi	CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						

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Project No.: 750603902

Figure: A-10a

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	ST		100 psi	CH	CLAY (CH) (continued)					66.9	59
32–39											
40	ST		100 psi								
41–49					BAY MUD						
50	ST		100 to 250 psi								
51					sand lense at 51.5 feet						
52–57											
57				CL	CLAY (CL) olive with orange-brown mottling, very stiff, wet						
58											
59–60	S&H		24			TxUU	2,200	2,030		25.5	100

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PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-2 PAGE 3 OF 4

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60	S&H		24	CL	CLAY (CL) (continued)						
62				CL	SANDY CLAY (CL) yellow-brown with olive mottling, hard, wet						
64.5	SPT		38								
67				SP-SC	SAND with CLAY (SP-SC) orange-brown, dense, wet						
69.5	SPT		34								
74.5	SPT		85/11"		very dense						
77				SP	SAND (SP) olive, very dense, wet						
79.5	SPT		87/11.5"								
84.5	SPT		69								
89	SPT		50/3"		SERPENTINITE						

COLMA FORMATION (67 to 89 feet)

BEDROCK (89 to 90 feet)

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Project No.: 750603902

Figure: A-10c

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B30-2

DEPTH (feet)	Sampler Type	Sample		SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91						SERPENTINITE (continued) intensely fractured, weak, moderately weathered, low hardness; BEDROCK						
92												
93												
94	SPT			50/1"								
95												
96												
97												
98												
99												
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 94.1 feet.
Boring backfilled with cement grout.
Groundwater encountered at 9 feet at 7:55 am on 5/3/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Figure: A-10d

PROJECT:	**BLOCKS 29-32** **MISSION BAY** San Francisco, California	**Log of Boring B30-3** PAGE 1 OF 4

Boring location:	See Site Plan, Figure 2		
Date started:	5/2/07	Date finished:	5/2/07
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Rope and Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

Logged by: J. Wong

LABORATORY TEST DATA

Ground Surface Elevation: +100.3 feet[2]

DEPTH (feet)	Sampler Type	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
0–1				2 inches asphalt concrete over 12 inches aggregate base							
1			SC	CLAYEY SAND with GRAVEL (SC) olive-brown, medium dense, moist, with angular to subangular gravel	FILL						
3	S&H	26									
6	SPT	17		olive-gray, with serpentinite fragments							
6.5			CL	SANDY CLAY with GRAVEL (CL) olive-gray, stiff, moist							
8	SPT	9									
9			SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray, medium dense, wet (5/2/07 at 8:15 am)							
11	S&H	18							6.0	11.0	
12	SPT	14									
14.5			GC	CLAYEY GRAVEL with SAND (GC) olive-gray, medium dense, wet							
17	SPT	10							13.6	22.3	
18.5			GP	GRAVEL (GP) dark gray, medium dense, wet							
20	SPT	19									
20			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
25	ST	75 psi		Consolidation Test, see Figure C-13						63.4	62

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Project No.: 750603902

Figure: A-11a

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–34				CH	CLAY (CH) (continued) BAY MUD						
34–36.5	ST		75 to 100 psi	CH							
44–46.5	ST		75 to 100 psi	CH	Consolidation Test, see Figure C-14					72.0	57
54–56.5	ST		150 to 250 psi	CH / CL							
55				CL	CLAY (CL) yellow-brown with olive mottling, hard, wet						
59–60	SPT		37	CL							

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Figure: A-11b

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60–62	SPT		37	CL	CLAY (CL) (continued)						
62–72	S&H (64–65.5)		18	SC	CLAYEY SAND (SC) orange-brown, medium dense, wet						
	SPT (69–70.5)		46		dense, lower fines content						
72–77.5	SPT (74–75.5)		69	SP-SC	SAND with CLAY (SP-SC) orange-brown, very dense, wet				7.7	25.0	
77.5–88	SPT (79–80.5)		34	SC	CLAYEY SAND (SC) olive with orange-brown mottling, dense, wet						
88–90	SPT (89–90)		33	CL	SANDY CLAY (CL) olive and yellow-brown with dark brown mottling, hard, wet						

COLMA FORMATION (62–88 ft)

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Project No.: 750603902

Figure: A-11c

DEPTH (feet)	Sampler Type	Sample		SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT			33	CL	SANDY CLAY (CL) (continued)						
91												
92												
93						SERPENTINITE intensely fractured, weak, moderately weathered, low hardness (BEDROCK)						
94	SPT			50/4"								
95												
96												
97												
98												
99	SPT			50/0"								
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 99 feet.
Boring backfilled with cement grout.
Groundwater encountered at 9.8 feet at 8:15 am on 5/2/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902 | Figure: A-11d

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	**Log of Boring B30-4**

Boring location: See Site Plan, Figure 2

Date started: 5/5/07 | Date finished: 5/5/07

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope and Cathead

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

Logged by: J. Wong

LABORATORY TEST DATA

Ground Surface Elevation: 100.4 feet[2]

DEPTH (feet)	Sampler Type	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
0–1.25				3 inches asphalt concrete over 12 inches aggregate base							
1.25–3			SC	CLAYEY SAND (SC) olive-brown, medium dense, moist	FILL						
3	S&H	15			FILL						
3–5.5			SP	SAND (SP) yellow-brown, medium dense, moist, fine-grained sand	FILL						
6	SPT	13			FILL						
5.5–8			CH	CLAY with GRAVEL (CH) gray, stiff, moist	FILL						
8	SPT	6	CH	▽ (5/5/07 at 8:40 am) green with dark green mottling, medium stiff, wet, with angular serpentinite gravel	FILL						
9.5–11.5			GC	CLAYEY GRAVEL (GC) green-gray, loose, wet, with Serpentinite	FILL						
11	S&H	4			FILL						
11.5–15			SC	CLAYEY SAND with GRAVEL (SC) olive, medium dense, wet	FILL						
12	SPT	12			FILL						
15–25			SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray, medium dense, wet	FILL						
17	SPT	13			FILL						
19.5				very loose to loose	FILL						
20	SPT	4			FILL				6.7	19.9	
25–30			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
29–30	ST	75 psi			BAY MUD	PP		750			

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Project No.: 750603902

Figure: A-12a

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	ST		75 psi	CH	CLAY (CH) (continued)						
32–39											
40–41	ST		75 to 100 psi	CH	Consolidation Test, see Figure C-15 medium stiff	TxUU	1,500	725		63.8 74.4	62 56
42–49					BAY MUD (30–49.5 ft)						
50–51	ST		100 to 250 psi	SP	SAND (SP) gray, loose to medium dense, wet						
52–53				CL	CLAY with SAND (CL) olive with orange-brown mottling, very stiff, wet						
55	S&H		18	CL		TxUU	1,700	3,450		22.3	105
56–58											
59–60	SPT		26		olive with red-brown mottling, very stiff, wet						

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Project No.: 750603902

Figure: A-12b

DEPTH (feet)	Sampler Type	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
60	SPT	26	CL	CLAY with SAND (CL) (continued)							
62			SC	CLAYEY SAND (SC) orange-brown, medium dense, wet, fine-grained sand	COLMA FORMATION						
65	SPT	18									
69				very dense, lower fines content							
70	SPT	58							12.4	23.3	
74				olive, higher fines content							
75	SPT	56									
76.5			SP-SC	SAND with CLAY (SP-SC) orange-brown, very dense, wet							
80	SPT	61									
81			CL	SANDY CLAY (CL) olive, hard, wet							
85	SPT	36		SERPENTINITE intensely fractured, moderately hard, weak, moderately weathered	BEDROCK						
87				SHALE intensely fractured, moderately hard, weak, moderately weathered							
89	SPT	50/ 4.5"									
90											

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Project No.: 750603902

Figure: A-12c

PROJECT:	**BLOCKS 29-32** **MISSION BAY** San Francisco, California	**Log of Boring B30-4** PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample		SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91						SHALE (continued)	BEDROCK						
92						SERPENTINITE intensely fractured, moderately hard, weak, moderately weathered							
93													
94	SPT			50/ 5.5"									
95													
96													
97													
98													
99													
100													
101													
102													
103													
104													
105													
106													
107													
108													
109													
110													
111													
112													
113													
114													
115													
116													
117													
118													
119													
120													

Boring terminated at a depth of 95 feet.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet at 8:40 am on 5/5/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902 | Figure: A-12d

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	**Log of Boring B30-5** PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 5/4/07 | Date finished: 5/4/07

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope and Cathead

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

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 100.3 feet[2])	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2				3 inches asphalt concrete over 12 inches aggregate base and 4 inches concrete						
2			SC	CLAYEY SAND with GRAVEL (SC) olive-gray, medium dense, moist						
3	S&H	16								
5–6	SPT	10		loose to medium dense, with brick fragments						
8	SPT	8	CH	CLAY with SAND (CH) gray, medium stiff to stiff, wet, with brick fragments and serpentinite (5/4/07 at 8:45 am)						
10–11	S&H	11		stiff, no brick						
11			CL-ML	SANDY SILTY CLAY (CL-ML) gray, stiff, wet LL = 23, PI = 7, see Figure C-38						
12	SPT	11								
15			SP-SC	SAND with CLAY and GRAVEL (SP-SC) green-gray, medium dense, wet						
17	SPT	11						10.8	16.1	
19–20	SPT	6		loose						
24–25	SPT	6		green with orange-brown mottling				11.9	24.1	
29–30	SPT	8								

FILL (2 to 30 feet)

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Project No.: 750603902 | Figure: A-13a

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B30-5

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		8		SAND with CLAY and GRAVEL (SP-SC) (continued)	FILL						
31–33				CH	CLAY (CH) gray, soft, wet	BAY MUD						
34–36	ST		75 to 300 psi	CH	sand lens at 35.5 to 37 feet	BAY MUD						
39–40	S&H		2	CH	with shell fragments	BAY MUD						
44–46	ST		75 to 150 psi	CH		BAY MUD						
52–53				CL	CLAY (CL) yellow-brown with orange-brown mottling, hard, wet, with trace fine-grained sand							
54–55	SPT		35	CL								
58				SC	CLAYEY SAND (SC) orange-brown, dense, wet	COLMA FORMATION						
59–60	SPT		36	SC		COLMA FORMATION				29.2	18.9	

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Project No.: 750603902

Figure: A-13b

TEST GEOTECH LOG- B30-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B30-5 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60	SPT		36	SC	CLAYEY SAND (SC) (continued)	COLMA FORMATION						
63				CL	CLAY (CL) olive, very stiff, wet							
65	SPT		22									
69	SPT		50/ 4.5"		SANDSTONE intensely fractured, friable, low hardness	BEDROCK						
72.5					SERPENTINITE intensely fractured, friable, low hardness							
75	SPT		85/ 10"									
79	SPT		50/ 5.5"									

Boring terminated at a depth of 79.5 feet.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet at 8:55 am on 5/4/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902

Figure: A-13c

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B31-2

Boring location: See Site Plan, Figure 2

Logged by: T. Shu

Date started: 6/10/11 | Date finished: 6/11/11

Drilling method: Rotary Wash

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

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

Ground Surface Elevation: 101 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
0–0.5					6-inches Asphalt Concrete						
0.5–0.75					3-inches Aggregate Base						
1–2	GRAB.			SM	SILTY SAND with GRAVEL (SM) gray-brown, medium dense, dry to moist, fine- to coarse gravel, trace brick and concrete fragments						
2–3.5	SPT	2, 4, 6	12	SM							
4				SM	grades clayey						
5–6.5	S&H	10, 11, 15	18	SP	SAND with GRAVEL (SP) gray-brown, medium dense, moist, coarse gravel, trace brick and serpentinite fragments				10.7	12.3	115
7.5–8.5					12-inches Reinforce Concrete with Rebar						
8.5–10				SP	SAND (SP) yellow-brown to brown, medium dense, moist, fine-grained sand, trace gravel						
10–11.5	SPT	19, 10, 14	29	GP	GRAVEL with SAND (GP) gray to gray-brown, medium dense, wet, fine- to coarse rounded gravel, trace concrete fragments						
13				GP	grades clayey						
14	GRAB.			GP	with serpentinite fragments, cobbles and boulders						
19–20.5	SPT	12, 12, 35	56	GC	CLAYEY GRAVEL (GC) blue-gray, very dense, wet, fine- to coarse gravel						
22–25	D&M (23–24.5)		200 psi	CH	CLAY (CH) gray, medium stiff, wet						
25–26.5	S&H	7, 19, 11	21	SC	CLAYEY SAND (SC) yellow-brown to olive-brown, medium dense, wet, fine-grained sand					18.6	116

FILL (0.75 to 22 feet)

BAY MUD (22 to 25 feet)

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Project No.: 750603902

Figure: A-14a

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B31-2 PAGE 2 OF 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
					SC	CLAYEY SAND (SC) (continued)						
31												
32												
33												
34												
35												
36	S&H		13 18 20	27		with black spots grades slightly less clay						
37												
38												
39												
40												
41												
42												
43												
44												
45					CL	SANDY CLAY (CL) yellow-brown, hard, wet				63.9	17.2	
46	S&H		16 29 50	55			TxUU	3,450	7,480		17.1	116
47	SPT		6 16 23	47								
48												
49												
50												
51												
52												
53												
54												
55												
56	S&H		23 35 45	56	SP-SM	SAND with SILT (SP-SM) red-brown, very dense, wet, fine-grained, with some clay COLMA FORMATION						
57	SPT		10 27 50/6"	92						8.5	25.1	
58												
59												
60												

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Project No.: 750603902

Figure: A-14b

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
					SP-SM	SAND with SILT (SP-SM) (continued) [COLMA FORMATION]						
61												
62												
63												
64												
65	S&H		24 50/6"	35/6"		CLAYSTONE olive-gray to gray, intensely fractured, moderately hard, weak, moderate weathering [BEDROCK]						
66												
67												
68												
69												
70												
71						SHEARED SHALE olive-gray to black, crushed, soft to low hardness, plastic, deep weathering						
72												
73												
74												
75												
76	SPT		15 21 38	71		gravel observed in cuttings						
77						SERPENTINITE olive-gray, intensely fractured, low hardness, weak to moderately strong, deep weathering						
78												
79												
80												
81												
82												
83												
84												
85	S&H		28 48 50/3"	69/9"		with white veins						
86												
87												
88												
89												
90												

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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
90–105						SERPENTINITE (continued) BEDROCK						
105	S&H		50/2"	35/2"								
	SPT		50/3"	50/3"		SHALE gray and olive-green, intensely fractured, low hardness, friable to weak, deep weathering						

91 92 93 94 95 96 97 98 99 100 101 102 103 104 105 106 107 108 109 110 111 112 113 114 115 116 117 118 119 120

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Boring terminated at a depth of 105.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

PROJECT: **BLOCKS 29-32 MISSION BAY** San Francisco, California

Log of Boring B31-3

Boring location: See Site Plan, Figure 2

Logged by: T. Shu

Date started: 8/29/11 | Date finished: 8/30/11

Drilling method: Rotary Wash

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

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

Ground Surface Elevation: 102.6 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	GRAB.			SP-SM	SAND with SILT (SP-SM) gray-brown, moist, fine to coarse grained, trace gravel and clay (FILL)						
2–3.5	S&H	14, 20, 19	27	SP	SAND (SP) brown, medium dense, dry to moist, fine grained, trace gravel and silt, with abundant serpentinite fragments					10.3	78
3.5–5	SPT	11, 5, 8	16	SP-SC	SAND with CLAY (SP-SC) dark gray and red-brown, medium dense, moist, coarse grained, trace gravel and asphalt fragments						
10–11.5	SPT	5, 4, 4	10	GC	(09/30/11, 7:40 AM) CLAYEY GRAVEL (GC) olive brown to gray, loose, moist, with coarse sand and gravel size serpentinite						
15–16.5	S&H	5, 2, 3	4	CH	CLAY (CH) gray, soft, wet, trace shell fragments (BAY MUD)						
17–18.5	D&M	50 psi		CH		TV		600			
26–27.5	D&M	150 psi		CH		TV		600			

FILL (0–15 ft); BAY MUD (15–30 ft)

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Project No.: 750603902 | Figure: A-15a

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B31-3

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
30–35					CH	CLAY (CH) (continued)						
36	D&M		50 psi		CH	BAY MUD	TV		600			
37–43					CH							
44	D&M		400 psi		SP	SAND (SP) gray, dense, wet, fine grained, trace clay and silt						
46	SPT		15 20 20	48	SP							
46–50					SP-SC	SAND with CLAY (SP-SC) brown, dense, wet, fine grained						
51	S&H		17 32 39	50	CL	SANDY CLAY (CL) brown to yellow-brown, hard, wet, fine sand, orange-brown speckling				58.9	15.5	
54–55					CL	grades with olive gray mottled gray with orange and black spots						
56	S&H		11 24 50	52	SC	CLAYEY SAND (SC) brown, very dense, wet				49.0	22.3	111
57–60					SC	COLMA FORMATION						

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Project No.: 750603902

Figure: A-15b

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
61	SPT		15 22 25	56	SP-SC	SAND with CLAY (SP-SC) yellow brown with red mottling, very dense, wet, fine grained						
64					SC	CLAYEY SAND (SC) gray-brown, very dense, wet						
65	S&H		35 50/4"	35/4"	SC	CLAYEY SAND (SC) brown, very dense, wet, fine grained						
71	SPT		15 11 19	36	SC	dense, grades light olive gray with reddish-brown speckling				22.9	21.6	
76	SPT		15 18 14	38	SC	grades with increase in clay content						
80						COLMA FORMATION (61 to 80 feet)						
81	SPT		10 17 24	49		SHEARED SHALE light olive gray, crushed, low hardness, weak, deep weathering						
86	SPT		31 31 33	77		grades to dark gray to black, scattered pockets of calcium carbonate						

Boring terminated at a depth of 86.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10.5 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902 | Figure: A-15c

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B31-4

Boring location:	See Site Plan, Figure 2	Logged by:	T. Shu
Date started:	9/1/11	Date finished:	9/1/11
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)		

Ground Surface Elevation: 97 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	GRAB.			SP-SM	SILTY SAND with GRAVEL (SP-SM) light brown, dry, trace concrete fragments	FILL						
2–3.5	S&H	28 19 15	24	SP-SC	SAND with CLAY (SP-SC) gray-brown, medium dense, moist, trace brick fragments and gravel							
3.5–5	SPT	3 5 4	11									
5–6.5	SPT	2 2 4	7		loose							
7					cobbles in cuttings from 7 to 10 feet							
12.5				CH	CLAY (CH) gray, soft, wet, trace shell fragments	BAY MUD						
14–15.5	D&M	100 psi					TV		400 to 500			
21–22.5	D&M	90 psi					TV		500			
28–29.5	D&M	90 psi			grades with dark brown organic veins		TV		600			

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Project No.: 750603902

Figure: A-16a

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B31-4 PAGE 2 OF 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
31–34				CH	CLAY (CH) (continued) grades sandy in cuttings (BAY MUD)						
35–36	S&H	10, 31, 31	43	SP-SC	SAND with CLAY (SP-SC) olive, dense, wet, fine grained						
37	SPT	10, 16, 22	46	SP-SC	grades olive gray						
40–41	SPT	6, 7, 8	18	SP-SC	medium dense, grades to light brown with red-brown mottling, grades with increase in clay content						
45–46	S&H	11, 17, 22	27	CL	SANDY CLAY (CL) gray-brown with red-brown mottling, very stiff, moist, fine sand						
50–51	S&H	15, 20, 24	31	SP-SM	SAND with SILT (SP-SM) brown to red-brown, dense, wet, fine grained, trace clay (COLMA FORMATION)						
52	SPT	7, 9, 13	26	SP-SM	medium dense						
55–56	SPT	11, 13, 17	36	SP-SM	dense						
60											

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Project No.: 750603902

Figure: A-16b

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
61	SPT		22 27 40	80	SP-SM	SAND with SILT (SP-SM) (continued) very dense	COLMA FORMATION						
62–64													
65–67	SPT		18 31 33	77									
68–70					GC	GRAVELLY CLAY with SAND (GC) red-brown, hard, wet, abundant coarse gravels of chert, greenstone and serpentinite							
71	SPT		22 22 41	63		SERPENTINIZED GREENSTONE blue-green and gray, crushed to intensely fractured, low hardness, weak, deep weathering, serpentinized in fracture surface, increase in weathering toward bottom of hole	BEDROCK						
75	SPT		50 50/4"	60/4"									
76–90													

Boring terminated at a depth of 75.7 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902 | Figure: A-16c

Log of Boring B31-5

PROJECT: **BLOCKS 29-32 MISSION BAY**, San Francisco, California

Boring location:	See Site Plan, Figure 2
Date started:	8/25/11
Date finished:	8/26/11
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by:	T. Shu

Ground Surface Elevation: 102 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					3-inch Asphalt Concrete (AC)						
1	GRAB.			SP-SM	SAND with SILT and GRAVEL (SP-SM) dark brown, medium dense, dry to moist, fine to coarse grained, trace brick fragments and clay						
2–3	S&H	11, 13, 12	17	SC	CLAYEY SAND (SC) dark brown, medium dense to dense, moist						
4	SPT	8, 13, 16	35								
6					grades with concrete fragment						
9					with cobbles						
10					(08/26/11, 7:30 am)						
10–11	SPT	7, 17, 8	30	GP	GRAVEL with SAND (GP) olive-gray, medium dense, wet, fine-grained, coarse sand, trace serpentinite fragments and clay, with gravel up to 1-1/2-inch diameter						
15–16	SPT	3, 3, 5	10		loose, grades blue-gray, with clay angular						
21–22	SPT	2, 3, 3	7								
25–26	SPT	5, 4, 4	10								
28				CH	CLAY (CH) gray, soft, wet						
30	D&M	100 psi									

FILL (0 to 28 feet)

BAY MUD (28 to 30 feet)

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Project No.: 750603902

Figure: A-17a

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	D&M				CH	CLAY (CH) (continued)	BAY MUD						
32–33	D&M		90 psi										
34–36	SPT		13 11 12	28	SP-SC	SAND with CLAY (SP-SC) brown to yellow-brown, medium dense, wet, fine grained							
40–41	S&H		10 20 36	39		dense, grades less clayey, trace silt							
45–46	SPT		10 14 14	34		grades clayey with orange seams							
50–51	SPT		8 9 12	25	CL	SANDY CLAY (CL) yellow-brown to mottled olive, very stiff, wet, fine sand							
53					SC	CLAYEY SAND (SC) yellow-brown, dense, wet	COLMA FORMATION						
55–56	S&H		14 24 32	39		grades less clayey							
57	SPT		10 13 25	46							20.5	19.5	
58–61													

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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
62–64					SC	CLAYEY SAND (SC) (continued)	COLMA FORMATION						
65–66	SPT		25 31 40	85	SC	SAND with CLAY (SP-SC) brown to olive-brown, very dense, wet, fine grained, trace red-brown silty sand	COLMA FORMATION				10.6	23.3	
70–71	SPT		6 13 20	40	SC	grades dense, with increased clay	COLMA FORMATION						
75–76	SPT		9 12 21	40	SC		COLMA FORMATION						
80–81	SPT		0 11 15	31	CL	CLAY (CL) gray, hard, wet, trace gravel size chert and shale fragments							
85–86	S&H		23 42 50/3"	64/ 9"	CL								
86–89						SHEARED SHALE gray, intensely fractured, low hardness, friable to weak, deep weathering	BEDROCK						
90	SPT		50/ 1.5"	60/ 1.5"			BEDROCK						

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Boring terminated at a depth of 90.2 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B31-6

Boring location:	See Site Plan, Figure 2
Date started: 8/30/11	Date finished: 8/31/11
Drilling method:	Rotary Wash
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by:	T. Shu

LABORATORY TEST DATA

Ground Surface Elevation: 103 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
0–2	GRAB.			SP-SM	SAND with SILT and GRAVEL (SP-SM) brown, dry, fine to coarse grained, trace brick and concrete fragments	FILL						
2–3	S&H	43 32 26	41		~3-inch Concrete Slab	FILL						
3–10				SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray-brown, dense, moist, medium to coarse grained, trace brick and concrete fragments, trace silt	FILL						
4	SPT	12 9 18	32	SP-SC		FILL						
8				SP-SC	grades with increase concrete fragments from 8 to 9 feet	FILL						
10–15	SPT	16 17 22	47	GP	▽ (08/31/11, 7:30 AM) GRAVEL with SAND (GP) blue-gray, dense, wet, coarse grained, with gravel size serpentinite, trace clay	FILL						
15–17.5	SPT	8 6 6	14	GP-GC	GRAVEL with CLAY and SAND (GP-GC) blue-gray, medium dense, wet	FILL						
17.5–23	D&M	25 psi		CH	CLAY (CH) gray, soft, wet, trace shell fragments	BAY MUD	TV		500			
19.5	D&M	100 psi		CH		BAY MUD	TV		500			
21				CH	grades slightly sandy	BAY MUD						
23–30				CL	SANDY CLAY (CL) olive-brown, hard, wet							
25	S&H	10 22 32	38	CL	grades red brown, with silty sand							

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Figure: A-18a

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		14 18 18	43	SP-SC	SAND with CLAY (SP-SC) olive, dense, wet, fine grained						
35–36	SPT		14 17 16	40		grades blue-gray						
40–41	S&H		13 18 40	41	SC	CLAYEY SAND (SC) orange-brown mottled olive, dense, wet, fine grained, trace red-brown silty sand and silt						
45–46	SPT		12 16 18	41	SP-SC	SAND with CLAY (SP-SC) orange-brown to brown, dense, wet, fine grained, with trace coarse sand						
50–51	SPT		24 19 18	44	SC	CLAYEY SAND (SC) olive-brown, dense, wet, fine to medium grained						
55–56	S&H		14 34 35	48		SHALE black to olive, crushed to intensely fractured, low hardness, friable, deep weathering with fragments of hard competent shale, trace olive to yellow-brown clay, talc-like texture, powdery, some planar bedding fabric						

COLMA FORMATION (40–51 ft)

BEDROCK (56–60 ft)

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Figure: A-18b

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
61	SPT		20 11 18	35		SHALE (continued)						
62–64						BEDROCK						
65	SPT		50/1"	60/1"		dark gray to black, intensely fractured, moderately hard, weak, moderately weathering						
66–90												

Boring terminated at a depth of 65.1 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10.5 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902

Figure: A-18c

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B31-7 PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: T. Shu

Date started: 8/31/11 | Date finished: 8/31/11

Drilling method: Rotary Wash

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

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

Ground Surface Elevation: 96.2 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
0–2	GRAB.			SP-SM	SILTY SAND with GRAVEL (SP-SM) light-brown, dry, trace brick fragments	FILL						
2–3.5	S&H	9, 4, 3	5	SP	SAND (SP) gray-brown to dark brown, very loose, moist, trace gravel and brick fragments, trace silt and clay	FILL						
3.5–5	SPT	1, 1, 2	4	SP		FILL						
10–11.5	D&M	25 psi		CH	CLAY (CH) gray, soft, wet, trace shell fragments	BAY MUD	TV		600			
11.5–13	D&M	80 psi		CH		BAY MUD	TxUU	1,450	500		59.5	63
15–16.5	S&H	0, 1, 1	1	CH		BAY MUD						
16				CH	SANDY CLAY (CH) gray, soft, wet, fine sand	BAY MUD						
19				SP-SM	SAND with SILT (SP-SM) brown to dark brown, dense, wet, fine grained	COLMA FORMATION						
20–21.5	SPT	4, 8, 13	25	SP-SM		COLMA FORMATION						
25–26.5	SPT	11, 18, 18	43	SP-SM	grades to yellow-brown, dense	COLMA FORMATION						

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Project No.: 750603902 | Figure: A-19a

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

Log of Boring B31-7

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

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
31	S&H	10 21 33	38	SC	CLAYEY SAND (SC) yellow-brown to brown, dense, wet, fine grained	COLMA FORMATION						
35–36	S&H	15 21 33	38		grades with red-brown silty sand							
40	S&H	47 50/5"	35/5"		GREENSTONE red-brown, intensely fractured, low hardness with moderately hard zones, friable to weak, deep weathering, highly oxidized	BEDROCK						
45	SPT	50/ 4.5"	60 4.5"									

Depth scale: 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60

Boring terminated at a depth of 45.4 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on SFCD + 100 feet. Elevations based on Topographic Survey titled "X-Site-Survey" by Sherwood Design Engineers, (October 2011).

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Project No.: 750603902
Figure: A-19b

TEST GEOTECH LOG B29-1 TO 8 AND B31-2 TO 7.GPJ TR.GDT 12/19/11

Boring location:	See Site Plan, Figure 2		
Date started:	4/30/07	Date finished:	5/1/07
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Rope and Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

Logged by: J. Wong

LABORATORY TEST DATA

Ground Surface Elevation: +105 feet[2]

DEPTH (feet)	Sampler Type	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
0–1			SC	CLAYEY SAND with GRAVEL (SC) olive-brown, medium dense, moist trace brick and subangular gravel	FILL						
3	S&H	13		LL = 20, PI = NP, see Figure C-38							
5–6	SPT	3		very loose							
8	SPT	7		loose, with serpentinite fragments					25.9	13.7	
10–11	S&H	5									
11			CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
12	SPT	5		▽ (4/30/07 at 1:40 pm)							
17–18	ST	50 to 75 psi		Consolidation Test, see Figure C-16		TxUU	1,050	275		66.8 59.1	60 64
24–26	ST	50 to 75 psi		Consolidation Test, see Figure C-17						57.6	66
30											

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Project No.: 750603902

Figure: A-20a

TEST GEOTECH LOG- B32-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Geologic Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–42				CH	CLAY (CH) (continued)	BAY MUD						
34–36.5	ST		0 to 75 psi									
42–47				SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet							
44–45.5	SPT		26									
47–54				CL	CLAY (CL) olive, stiff to very stiff, wet, with trace silt							
49–50.5	SPT		16									
54–58.5				SC	CLAYEY SAND (SC) yellow-brown, medium dense to dense, wet	COLMA FORMATION						
54–55.5	S&H		30									
58.5–60					SILTY SAND (SM) orange-brown, medium dense, wet							
59–60	SPT		28							14.9	24.7	

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PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B32-1

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Geologic Unit	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60	SPT		28	SM	SILTY SAND (SM) (continued)	COLMA FORMATION						
61												
62				SP-SC	SAND with CLAY (SP-SC) orange-brown, very dense, wet, trace fines, medium grained sand							
63												
64												
65	SPT		46/ 5.5"							11.7	22.8	
66–69												
70	SPT		56							8.8	22.6	
71–74												
75	SPT		59									
76–79												
80	SPT		56		olive, fine-grained sand							
81–82												
83					SERPENTINITE intensely fractured, weak, moderate weathering, moderately hard	BEDROCK						
84												
85	SPT		88/3"									
86–89												
90	SPT		56		plastic, soft							

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Project No.: 750603902

Figure: A-20c

TEST GEOTECH LOG- B32-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B32-1

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		56		SERPENTINITE (continued)	BEDROCK						
91												
92												
93												
94	SPT		50/3"		friable, low hardness							
95												
96												
97												
98												
99	SPT		50/3"		weak							
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

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Boring terminated at a depth of 99.25 feet.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 12.5 feet at 1:40 pm on 4/30/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902

Figure: A-20d

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B32-2

Boring location:	See Site Plan, Figure 2	Logged by:	J. Wong
Date started:	4/27/07	Date finished:	4/30/07
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Rope and Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: +101 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
0–4.5	S&H (3)	12	SC	SAND with CLAY (SP-SC) gray-brown, medium dense, moist, with traces of brick and angular gravel	FILL						
4.5–11	SPT (6)	14	SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, with fragments of bricks	FILL						
	SPT (8)	4		▽ (4/27/07 at 2:45 pm) olive-brown, very loose to loose, wet							
	SPT (11)	2		very loose							
11–30	ST (12–14.5)	0 to 75 psi	CH	CLAY (CH) gray, soft, wet, with shell fragments	BAY MUD						
19	ST (19–21.5)	50 to 150 psi	CH		BAY MUD	TxUU	850	345		57.2	65
29	ST (29–30)										

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Project No.: 750603902 | Figure: A-21a

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					CLAY (CH) (continued)	BAY MUD						
31	ST		0 to 150 psi		sandy at 30.5 feet							
32												
33												
34												
35	S&H		4	CH	soft to medium stiff							
36												
37												
38												
39												
40	ST		100 to 300 psi	SP	SAND (SP) gray, wet							
41												
42					CLAY (CL) olive, very stiff, wet							
43												
44												
45	S&H		16				TxUU	1,660	1,540		29.6	94
46												
47				CL								
48												
49												
50	S&H		13		stiff							
51												
52												
53					CLAYEY SAND (SC) yellow-brown, dense, wet	COLMA FORMATION						
54												
55	SPT		34							18.7	22.0	
56												
57				SC								
58												
59												
60	SPT		31									

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PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B32-2

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60–61	SPT		31	SC	CLAYEY SAND (SC) (continued)	COLMA FORMATION						
62–69.4	SPT (64)		50/3"		SERPENTINITE intensely fractured, friable, moderate weathering, low harness	BEDROCK						
	SPT (69)		50/ 4.5"									

Depth scale: 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90

Boring terminated at a depth of 69.4 feet.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet at 2:45 pm on 4/27/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902
Figure: A-21c

TEST GEOTECH LOG- B32-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B32-3 PAGE 1 OF 3

Boring location:	See Site Plan, Figure 2
Date started:	4/25/07
Date finished:	4/26/07
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Rope and Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by:	J. Wong

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: +99.5 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			SC	CLAYEY SAND with GRAVEL (SC) dark gray, loose, moist, with fragments of brick and concrete (FILL)						
3	S&H	5								
5	SPT	9		olive-brown, trace gravel						
7				∇ (4/25/07 at 3:30 pm)						
8	SPT	4	CL	CLAY (CL) black, soft to medium stiff, wet, majority of sample is wood						
10	S&H	6	SC	CLAYEY SAND with GRAVEL (SC) dark brown, loose, wet, with fragments of bricks				13.8	23.6	
12	SPT	9								
13			CH	CLAY (CH) gray, soft, wet, with shell fragments (BAY MUD)						
17	ST	50 psi								
24.5	ST	75 psi		Consolidation Test, see Figure C-18					50.9	71
30										

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Figure: A-22a

TEST GEOTECH LOG- B32-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B32-3 PAGE 2 OF 3

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31–34				CH	CLAY (CH) (continued)	BAY MUD						
35–38	ST		100 to 250 psi	SP	SAND (SP) gray, wet							
39–41	SPT		51	CL	CLAY with GRAVEL (CL) yellow-brown with olive mottling, hard, wet							
42–43					CLAYSTONE intensely fractured, weak, moderate weathering, low hardness	BEDROCK						
44	SPT		50/3"									
50	SPT		64		plastic							
52–53					SHALE intensely fractured, friable, moderate weathering, low hardness							
54	SPT		50/5"									
59–60	SPT		69		plastic							

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Figure: A-22b

TEST GEOTECH LOG- B32-1 TO 5 - NO BLOWS.GPJ TR.GDT 12/19/11

PROJECT:	BLOCKS 29-32 MISSION BAY San Francisco, California	Log of Boring B32-3 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
60	SPT		69		SHALE (continued)						
64	SPT		50/3"		friable						
69	SPT		50/3"		BEDROCK (60 to 69.25 ft)						

Depth scale: 61–90 feet.

Boring terminated at a depth of 69.25 feet.
Boring backfilled with cement grout.
Groundwater encountered at 7 feet at 3:30 pm on 4/25/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902
Figure: A-22c

PROJECT: **BLOCKS 29-32**
MISSION BAY
San Francisco, California

Log of Boring B32-4

Boring location: See Site Plan, Figure 2

Logged by: J. Wong

Date started: 4/25/07 | Date finished: 4/25/07

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope and Cathead

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

LABORATORY TEST DATA

Ground Surface Elevation: +96 feet[2]

DEPTH (feet)	Sampler Type	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
0–4.5			SC	CLAYEY SAND with GRAVEL (SC) olive-brown, medium dense, moist, with Serpentinite fragments and subangular gravel (FILL)						
3	S&H	17								
4.5–14			SC	CLAYEY SAND (SC) olive-brown, medium dense, moist, with brick and concrete fragments (FILL)						
6	SPT	16						14.4	10.8	
7				∇ (4/25/07 at 8:30 am) wet, with gravel fines: LL = 28, PI = 10, see Figure C-38						
8	S&H	19								
11	SPT	13						13.3	17.0	
14–25			CH	CLAY (CH) gray, soft, wet, with shell fragments (BAY MUD)						
18	ST	50 psi		Consolidation Test, see Figure C-19					50.9	58
25	ST	50 to 250 psi	CL	CLAY (CL) yellow-brown, stiff, wet, with trace fine-grained sand	PP		2,500			
29.5	S&H	13								
30										

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Project No.: 750603902 | Figure: A-23a

PROJECT: **BLOCKS 29-32** **MISSION BAY** San Francisco, California

Log of Boring B32-4

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30	S&H		13	CL	CLAY (CL) (continued)						
32.5					CLAYSTONE intensely fractured, plastic, moderate weathering, soft (BEDROCK)						
34	S&H		50/6"								
37.5					SERPENTINITE intensely fractured, plastic, moderate weathering, soft (BEDROCK)						
39	SPT		50/ 5.5"								
42.25					SHALE intensely fractured, friable, moderate weathering, moderately hard (BEDROCK)						
44	SPT		50/ 4.5"								
49	SPT		50/4"								
53.5	SPT		50/ 0.5"								

Depth scale: 31–60 feet.

Boring terminated at a depth of 54 feet.
Boring backfilled with cement grout.
Groundwater encountered at 7 feet at 8:30 am on 4/25/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Project No.: 750603902

Figure: A-23b

PROJECT: **BLOCKS 29-32 MISSION BAY** San Francisco, California

Log of Boring B32-5

Boring location:	See Site Plan, Figure 2	Logged by:	J. Wong
Date started:	4/26/07	Date finished:	4/27/07
Drilling method:	Rotary Wash		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Rope and Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

MATERIAL DESCRIPTION — Ground Surface Elevation: +93 feet[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	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
0–4.5			CL	SANDY CLAY with GRAVEL (CL) olive-brown, stiff, wet, with fragments of concrete and brick, traces angular to subangular gravels						
2				▽ (4/27/07 at 7:00 am)						
3	S&H	15								
4.5			SC	CLAYEY SAND with GRAVEL (SC) olive-brown, loose, wet, with brick						
6	SPT	7								
8	SPT	18		medium dense				18.7	12.1	
10.5	S&H	50/3"		concrete obstruction at 10.5 feet						
11.5			CH	CLAY (CH) gray, soft to medium stiff, wet, with shell fragments						
12	SPT	4								
14	ST	75 to 100 psi								
14.75			SP	SAND (SP) gray, wet						
16			SC	CLAYEY SAND (SC) olive, medium dense, wet						
18	SPT	21								
18.75			CL	CLAY with SAND (CL) olive with red-brown mottling, stiff, wet						
20	SPT	9								
24.5	S&H	26		orange-brown, very stiff	TxUU	850	4,450		15.7	118
26.5			CL	CLAY (CL) yellow-brown with orange-brown mottling, hard, wet, with bedrock fragments						
29.5	SPT	35								

FILL (0 to 11.5 feet)

BAY MUD (11.5 to 14.75 feet)

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Project No.: 750603902

Figure: A-24a

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30	SPT		35	CL	CLAY (CL) (continued)						
31											
32					SERPENTINITE intensely fractured, friable, deep weathering, low hardness BEDROCK (32 to 44.4 feet)						
33											
34	SPT		50/ 5.5"								
35											
36											
37											
38											
39	SPT		50/5"								
40											
41											
42											
43											
44	SPT		50/5"								
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

Boring terminated at a depth of 44.4 feet.
Boring backfilled with cement grout.
Groundwater encountered at 2.5 feet at 7:00 am on 4/27/07.

[1] S&H blow counts converted to SPT N-values using a factor of 0.6.
[2] Elevation based on SFCD + 100 feet. Elevations based on Topographic Survey by Winzler and Kelly (June 2006).

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Figure: A-24b

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
- Sample taken with Dames & Moore U-Type split-barrel sampler with a 3.5-inch outside diameter and a 2.5-inch inside diameter
- Disturbed sample, hand auger
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample, grab groundwater
- Sample taken with Direct Push sampler

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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
D&M U	Dames & Moore U-Type split-barrel sampler with a 3.5-inch outside diameter and a 2.5-inch inside diameter
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

BLOCKS 29-32
MISSION BAY
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CLASSIFICATION CHART

Date 11/15/11 | Project No. 750603902 | Figure A-25

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV **WEATHERING -** The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V **CONSOLIDATION OF SEDIMENTARY ROCKS:** usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

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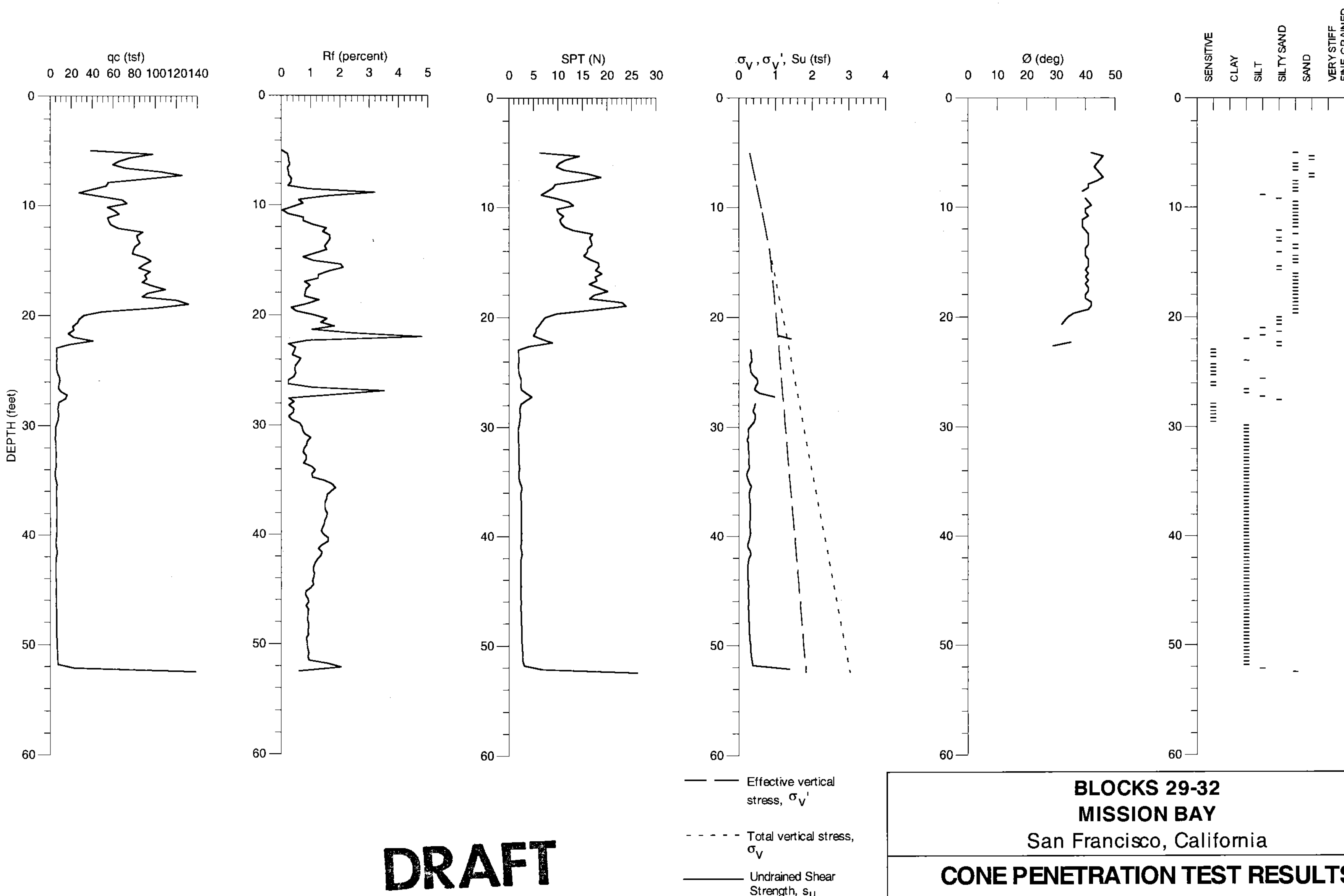
VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

APPENDIX B

Cone Penetration Tests



DRAFT

Terminated at 52.65 feet
Groundwater not measured during test.
Date performed: 08/25/11.
Ground surface elevation: 101.8 feet, San Francisco City Datum plus 100 feet.

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
CONE PENETRATION TEST RESULTS **CPT-C29-3**		
Date 11/15/11	Project No. 750603902	Figure B-1

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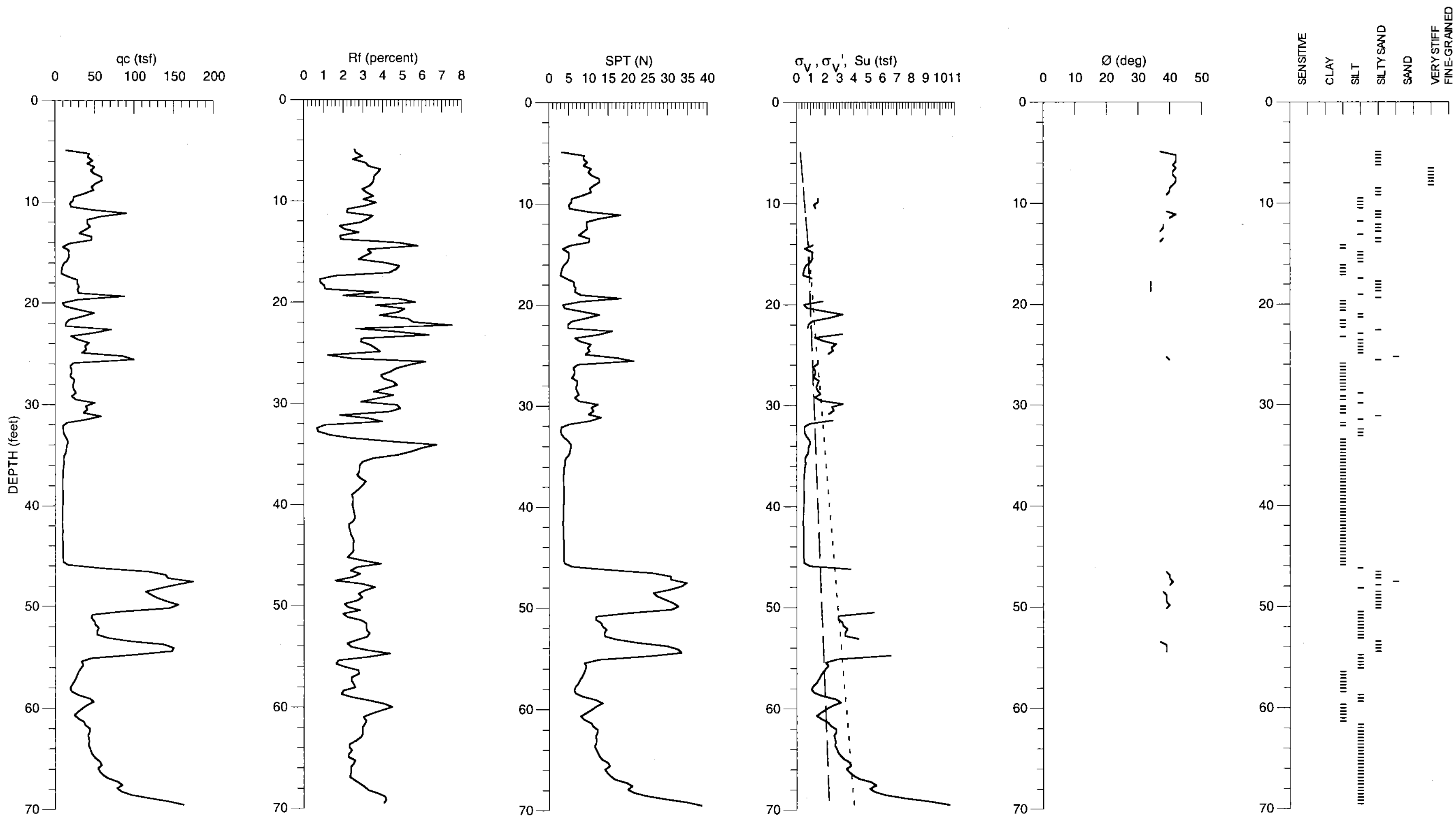

Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, s_u

DRAFT

Terminated at 69.88 feet
Groundwater not measured during test.
Date performed: 08/25/11.
Ground surface elevation: 100.5 feet, San Francisco City Datum plus 100 feet.

BLOCKS 29-32
MISSION BAY
San Francisco, California

CONE PENETRATION TEST RESULTS
CPT-C29-4

Date 11/15/11	Project No. 750603902	Figure B-2

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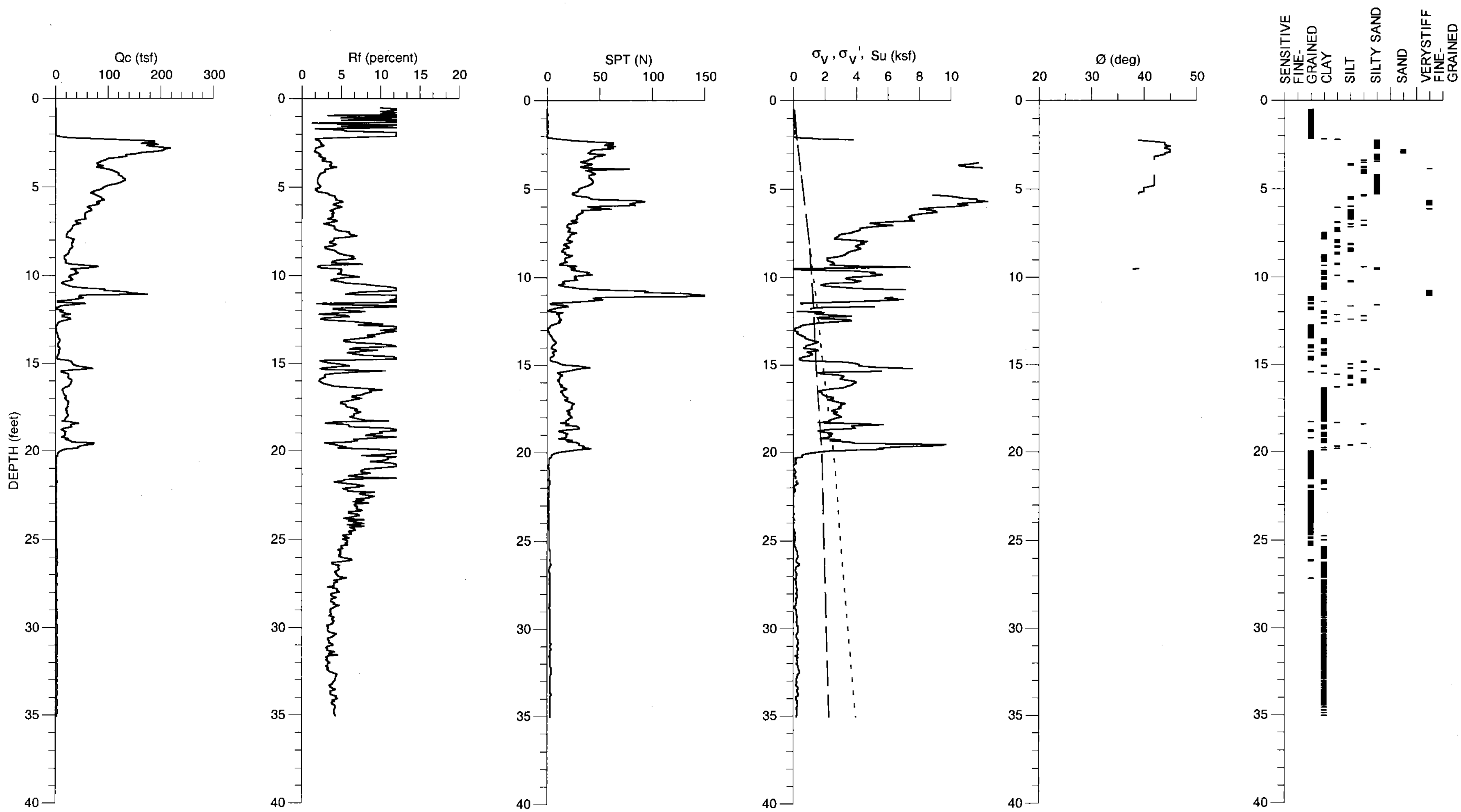

Effective vertical stress, σ_V'

Total vertical stress, σ_V

Undrained Shear Strength, s_u

DRAFT

Terminated at 35.0 feet.
Groundwater assumed to be at a depth of 8.0 feet bgs.
Date performed: 05/04/07.
Elevation: 100.3 feet, Datum: San Francisco City Datum plus 100 feet.

BLOCKS 29-32
MISSION BAY
San Francisco, California

CONE PENETRATION TEST RESULTS
C30-1

Date 11/15/11	Project No. 750603902	Figure B-3

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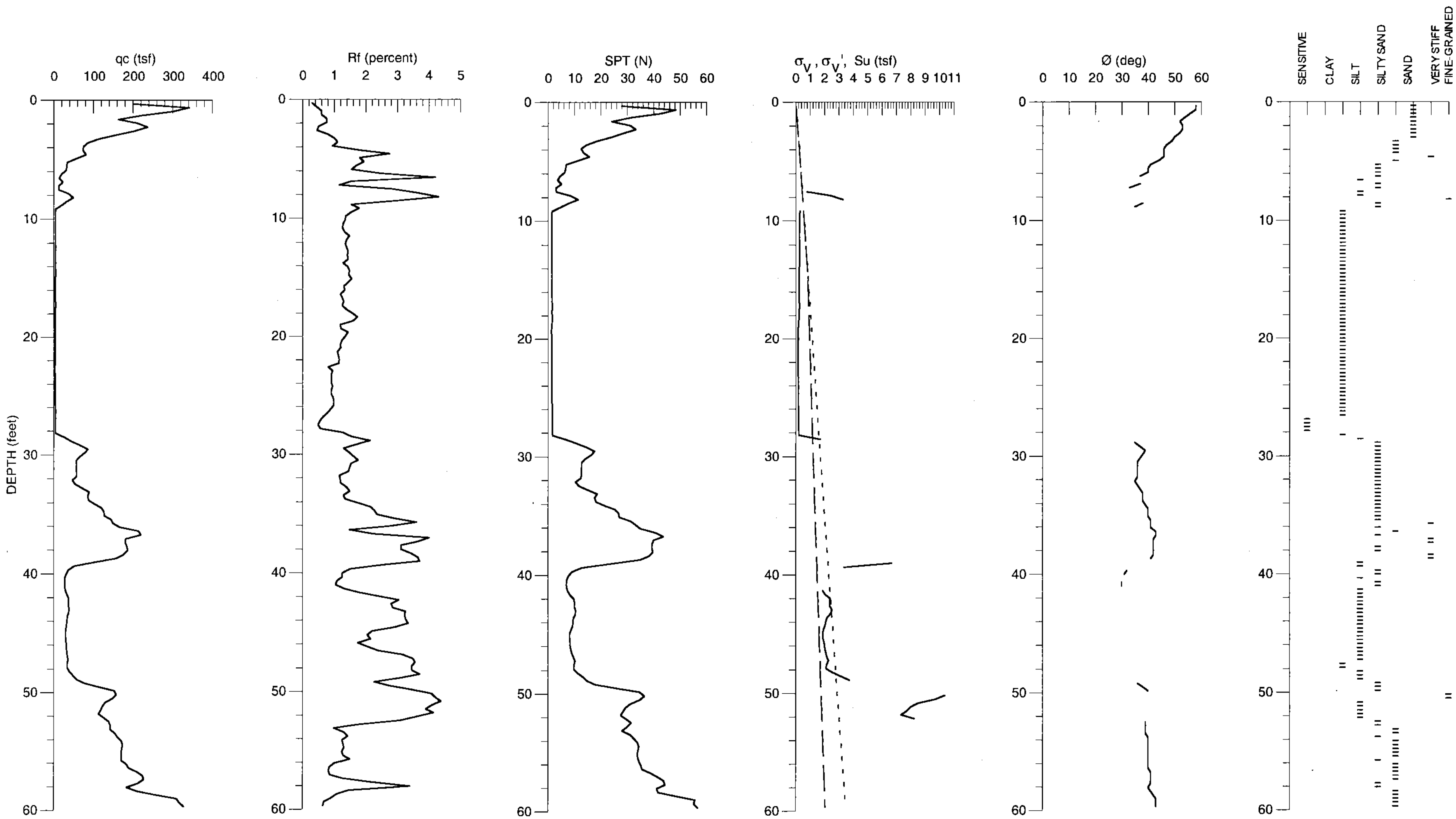

— — Effective vertical stress, σ_v'

- - - - Total vertical stress, σ_v

——— Undrained Shear Strength, s_u

DRAFT

Terminated at 60.03 feet
Groundwater not measured during test.
Date performed: 08/25/11.
Ground surface elevation: 100 feet, San Francisco City Datum plus 100 feet.

BLOCKS 29-32
MISSION BAY
San Francisco, California

CONE PENETRATION TEST RESULTS
CPT-C31-2

Date 11/15/11	Project No. 750603902	Figure B-4

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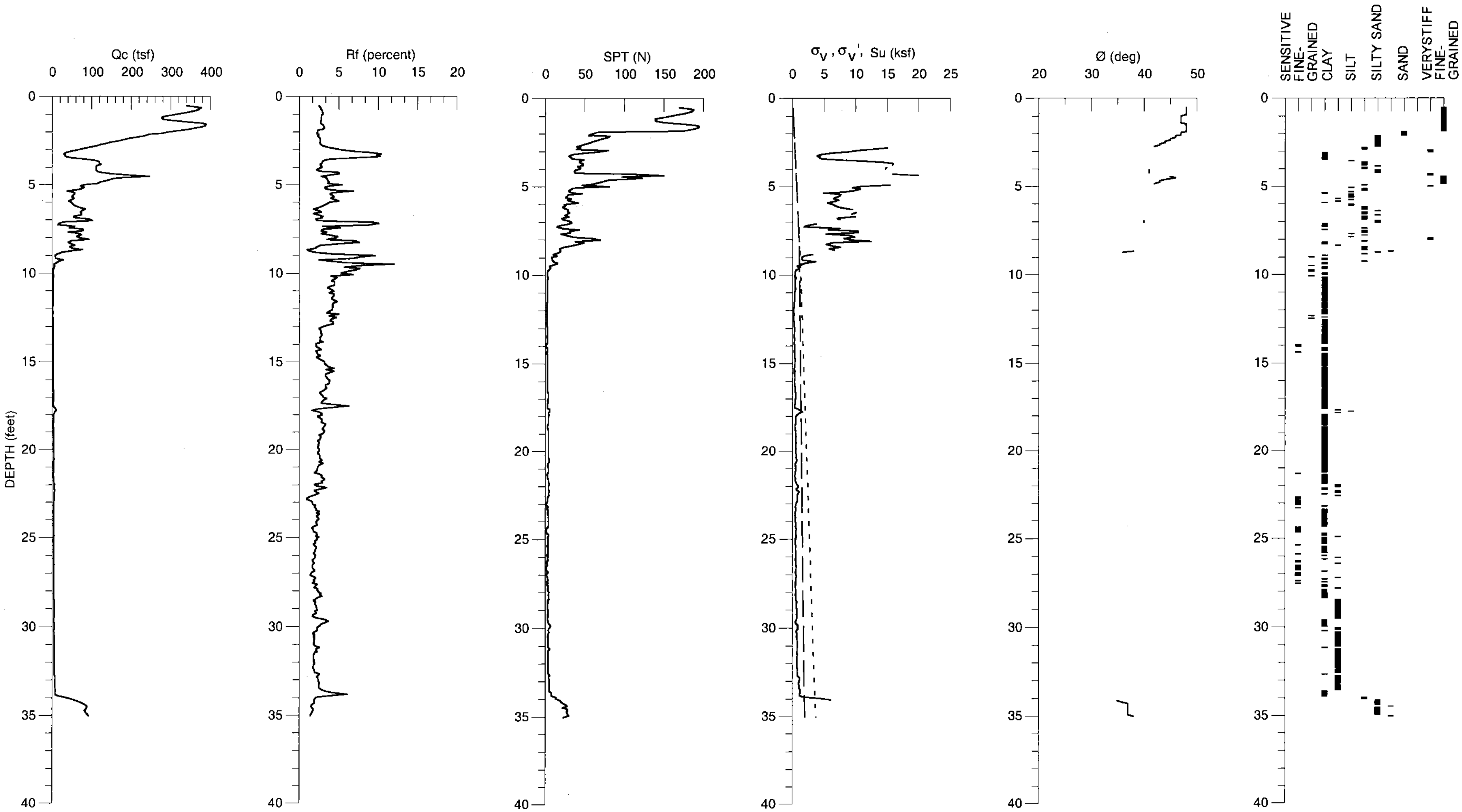

Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, s_u

DRAFT

Terminated at 79.6 feet.
Groundwater assumed to be at a depth of 7.0 feet bgs.
Date performed: 08/08/06.
Elevation: 100.6 feet, Datum: San Francisco City Datum +100 feet.

BLOCKS 29-32
MISSION BAY
San Francisco, California

CONE PENETRATION TEST RESULTS
C32-1

Date 11/15/11	Project No. 750603902	Figure B-5

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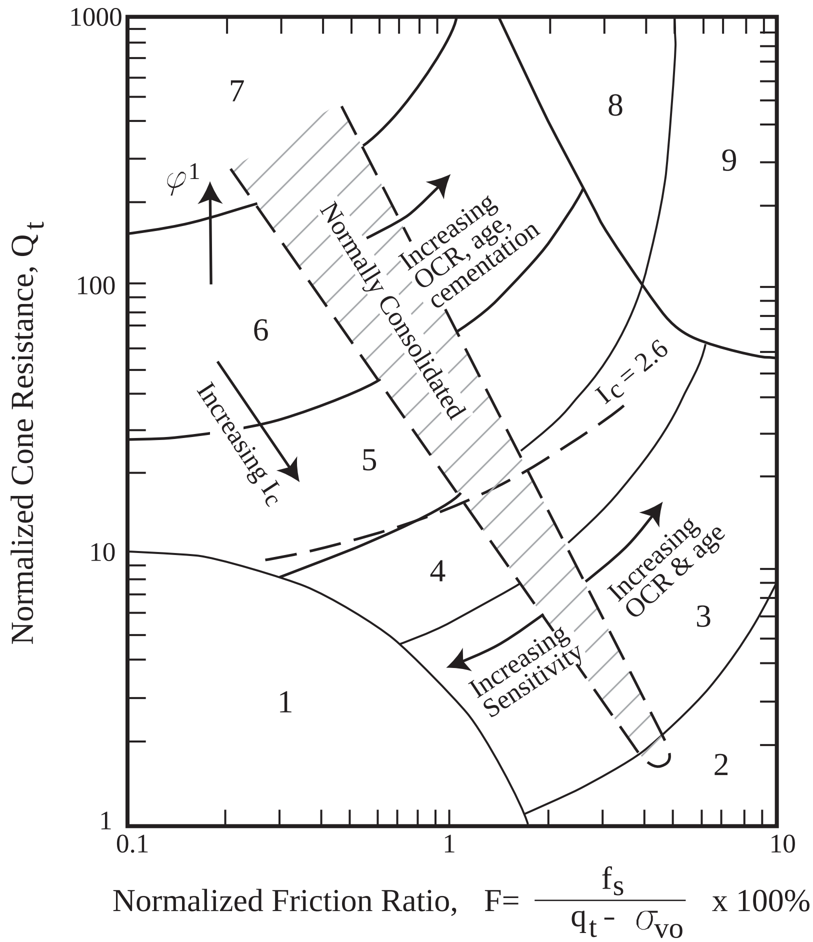

ZONE	SOIL BEHAVIOR TYPE
1	Sensitive Fine Grained
2	Organic Material
3	SILTY CLAY to CLAY
4	CLAYEY SILT to SILTY CLAY
5	SILTY SAND to SANDY SILT
6	SANDS to SILTY SAND
7	GRAVELLY SAND to Dense SAND
8	Very Dense SAND to CLAYEY SAND
9	Very Stiff, Fine Grained

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$Q_t = (q_t - \sigma_{vo})/\sigma'_{vo}$ = Normalized Cone Resistance

$q_t = q_c + (1\text{-}a)u_2$ = Corrected Cone Resistance

q_c = Measured Cone Resistance

a = 0.8 = Area Ratio of Cone

u_2 = Pore Pressure Measured Behind Cone During Test

σ_{vo} = Total Vertical Stress

σ'_{vo} = Total Effective Vertical Stress

$F = f_s/(q_t - \sigma_{vo})$ x 100% = Normalized Friction Ratio

f_s = Measured unit Sleeve Friction Resistance

Note Testing Performed in Accordance with ASTM D5778-95

Reference: Lunne, T., Robertson, P.K., and Powell, J.J.M., 1997.

BLOCKS 29-32
MISSION BAY
San Francisco, California

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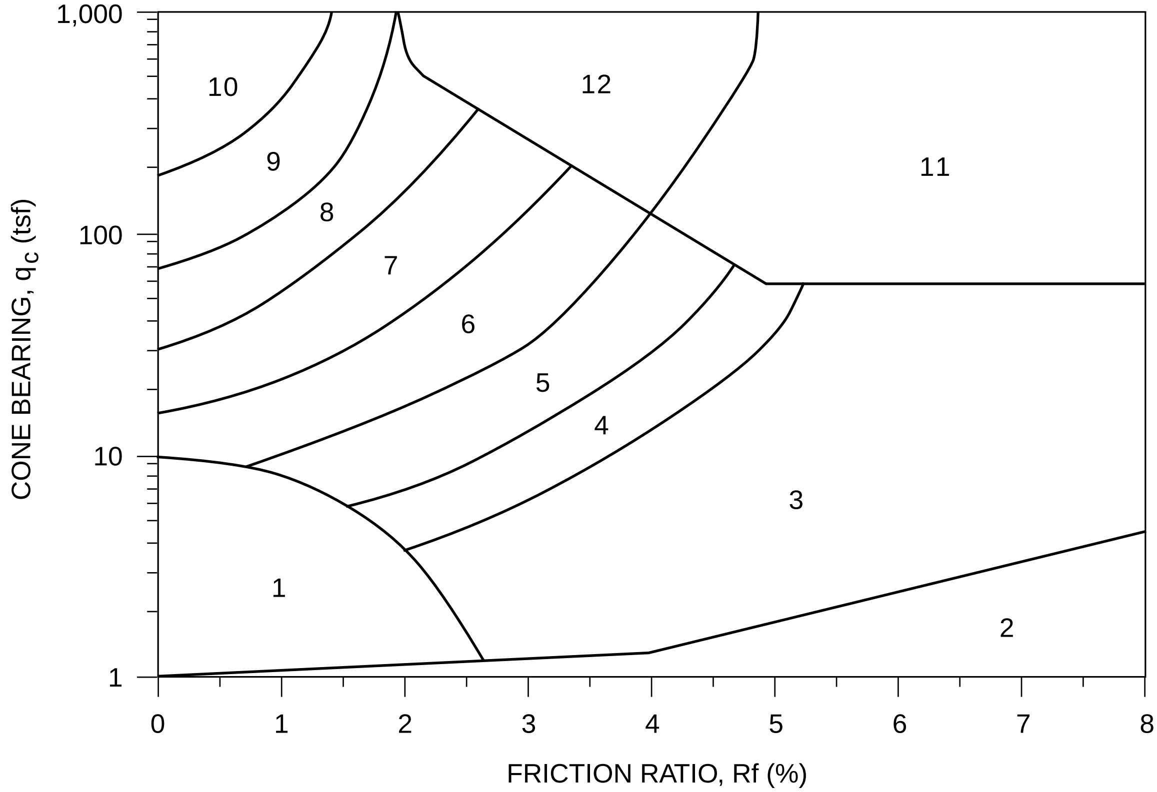
GREGG

CLASSIFICATION CHART FOR CONE PENETRATION TESTS FOR C29-3, C29-4 AND C31-2

Date 11/15/11	Project No. 750603902	Figure B-6



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 x 100 = Friction Ratio

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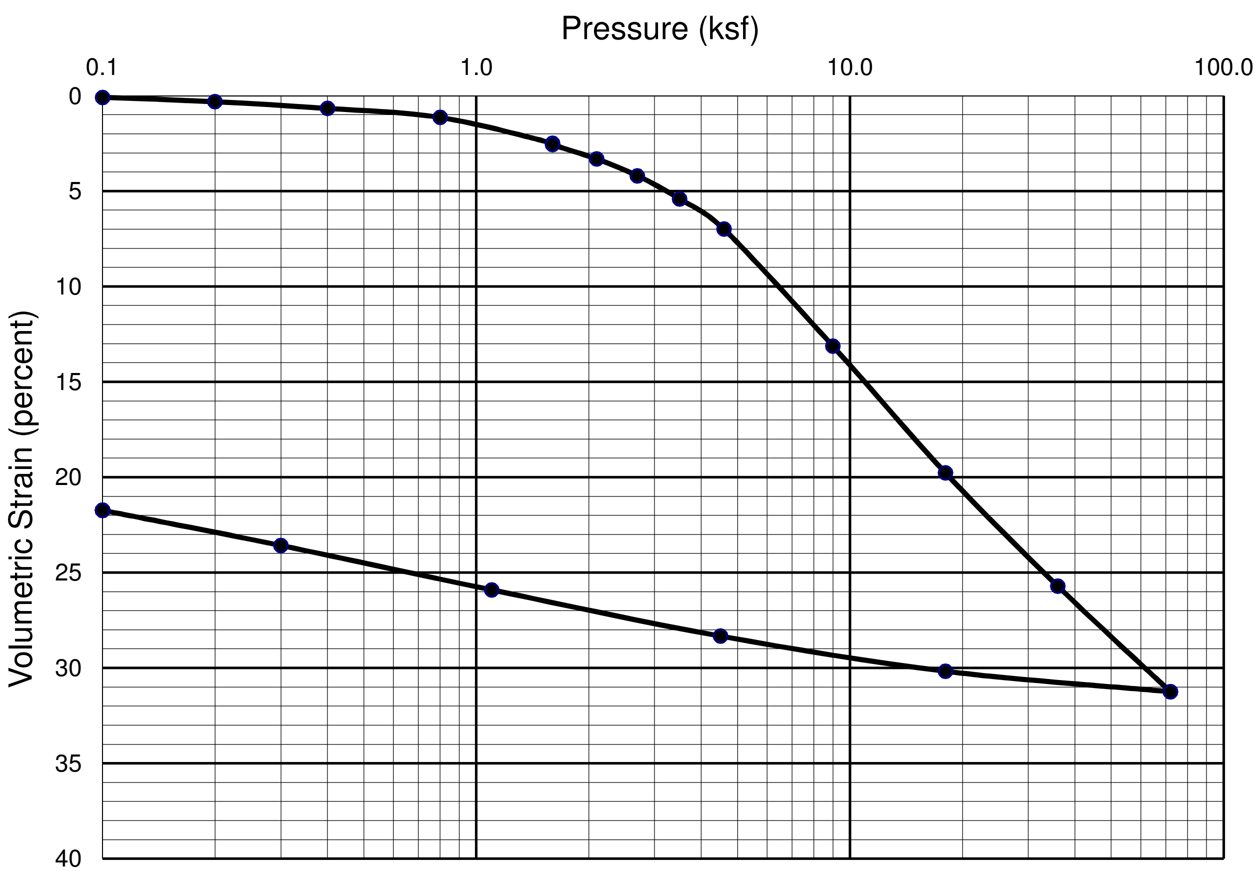
Note: Testing performed in accordance with ASTM D3441.

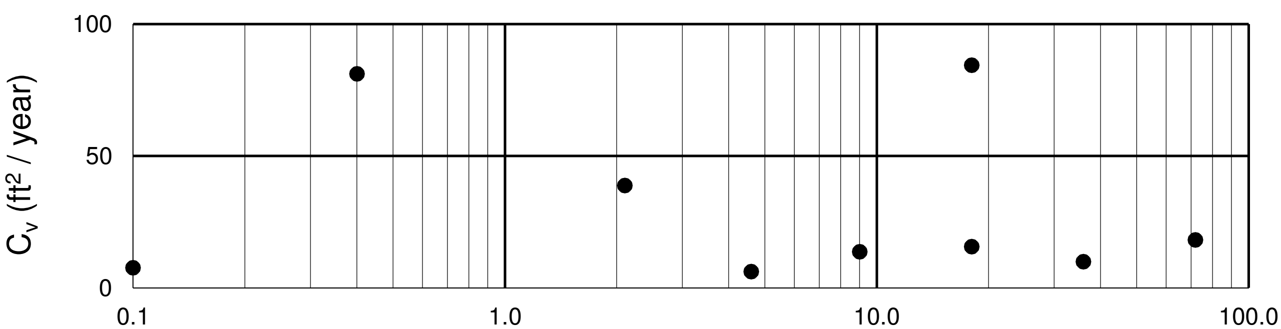
References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud $q_c \leq 9$).
Estimated from local experience (fine-grained soils $q_c > 9$).

BLOCKS 29-32 MISSION BAY San Francisco, California	CLASSIFICATION CHART FOR CONE PENETRATION TESTS FOR C30-1 AND C32-1
Treadwell&Rollo A LANGAN COMPANY	Date 11/15/11 | Project No. 750603902 | Figure B-7

APPENDIX C

Laboratory Test Results





DRAFT

Sampler Type: Dames & Moore		Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	49.4 %	w_f	30.3 %	
Overburden Pressure, p_o	1,900 psf	Void Ratio	e_o	1.33	e_f	0.80	
Preconsol. Pressure, p_c	3,500 psf	Saturation	S_o	100 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.2	Dry Density	γ_d	72 pcf	γ_d	94 pcf	
LL --	PL --	PI --		G_s 2.70 (assumed)			
Classification CLAY (CH), gray		Source		B29-1 at 20 feet			

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
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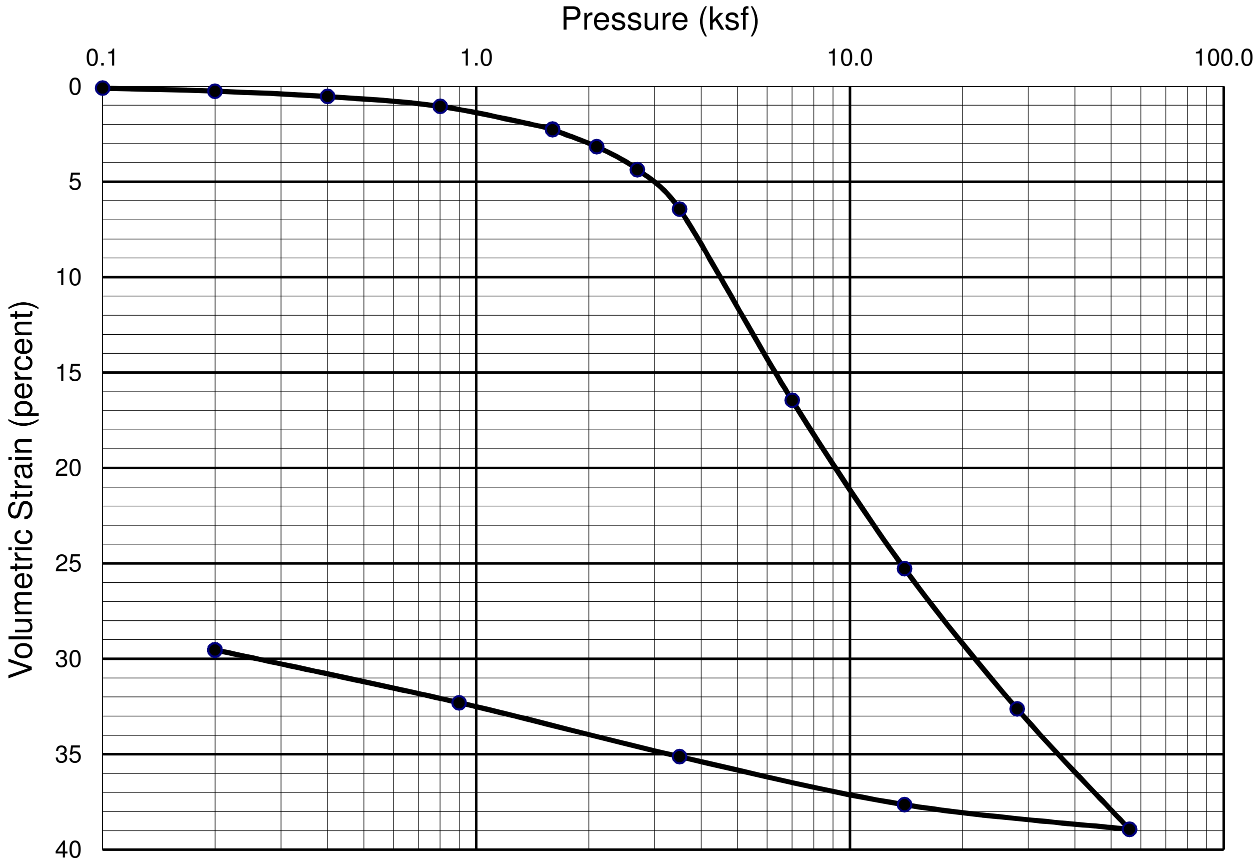
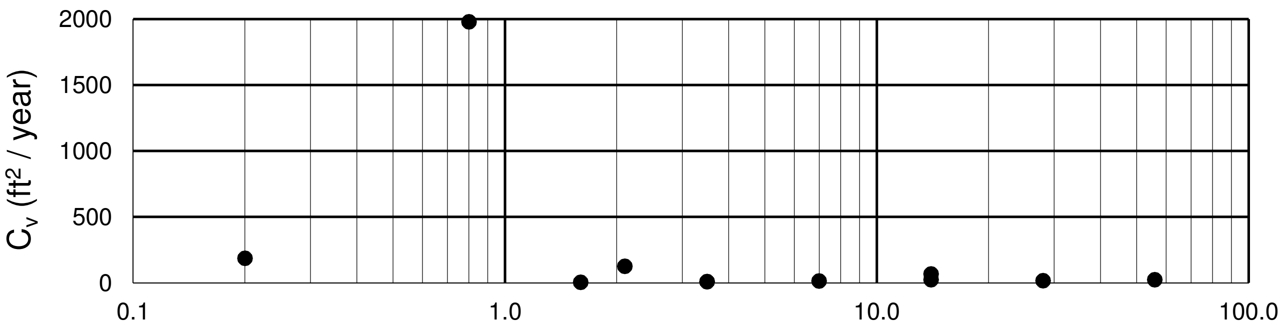
CONSOLIDATION TEST REPORT

Date 11/22/11	Project No. 750603902	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	71.6	%	w_f	42.0 %
Overburden Pressure, p_o	2,250	psf		Void Ratio	e_o	1.99		e_f	1.11
Preconsol. Pressure, p_c	3,000	psf		Saturation	S_o	97	%	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.28			Dry Density	γ_d	56	pcf	γ_d	80 pcf
LL --	PL --		PI --		G_s	2.70	(assumed)		
Classification CLAY (CH), gray				Source		B29-1 at 30 feet			

BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
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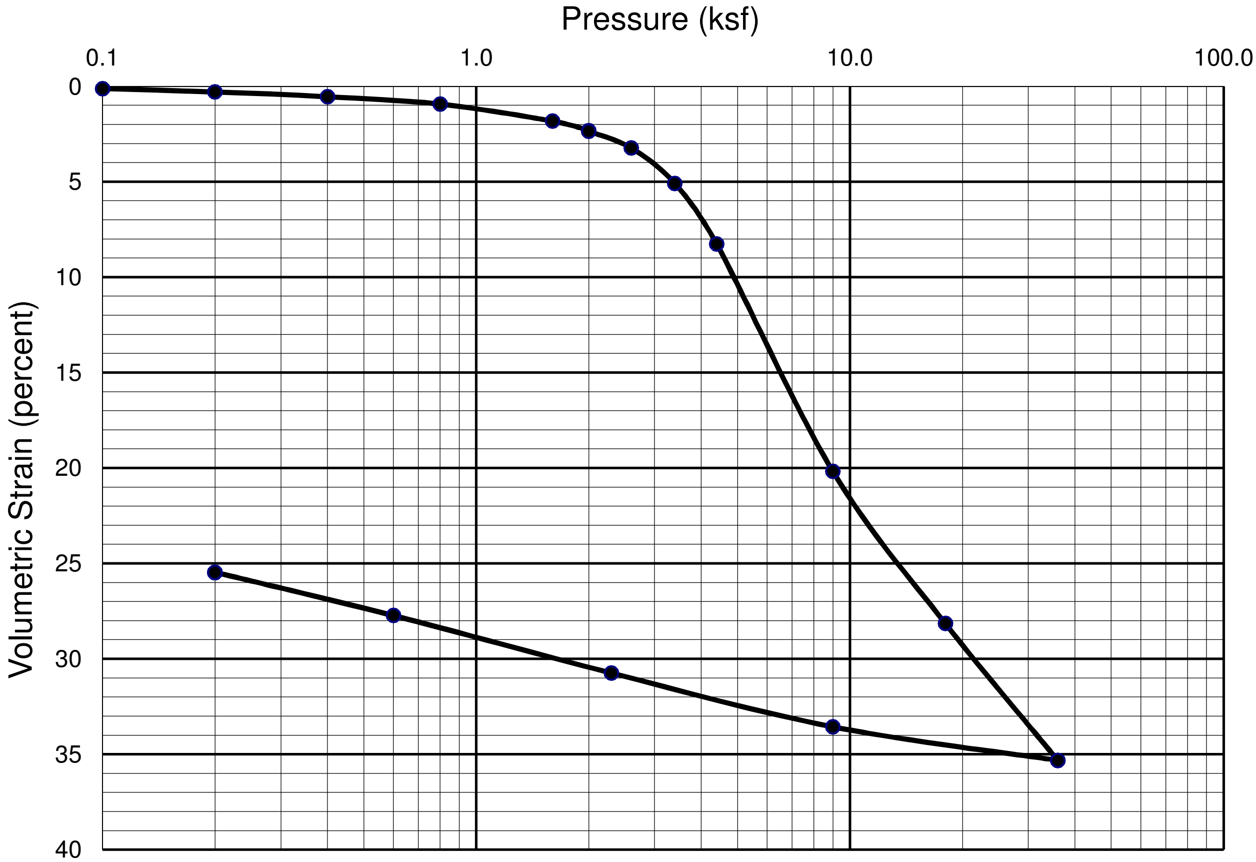
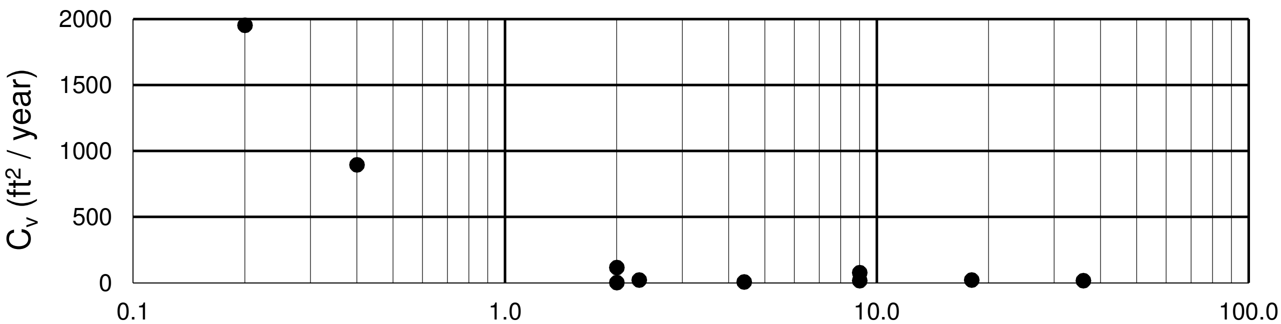
CONSOLIDATION TEST REPORT

Date 11/22/11 | Project No. 750603902 | Figure C-2





DRAFT

Sampler Type: Dames & Moore				Condition		Before Test			After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	73.2	%	w_f	46.4	%
Overburden Pressure, p_o	2,630	psf		Void Ratio	e_o	2.00		e_f	1.24	
Preconsol. Pressure, p_c	3,300	psf		Saturation	S_o	99	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.31			Dry Density	γ_d	56	pcf	γ_d	75	pcf
LL --		PL --		PI --				G_s 2.70	(assumed)	
Classification CLAY (CH), gray				Source		B29-1 at 40 feet				

BLOCKS 29-32
MISSION BAY
San Francisco, California

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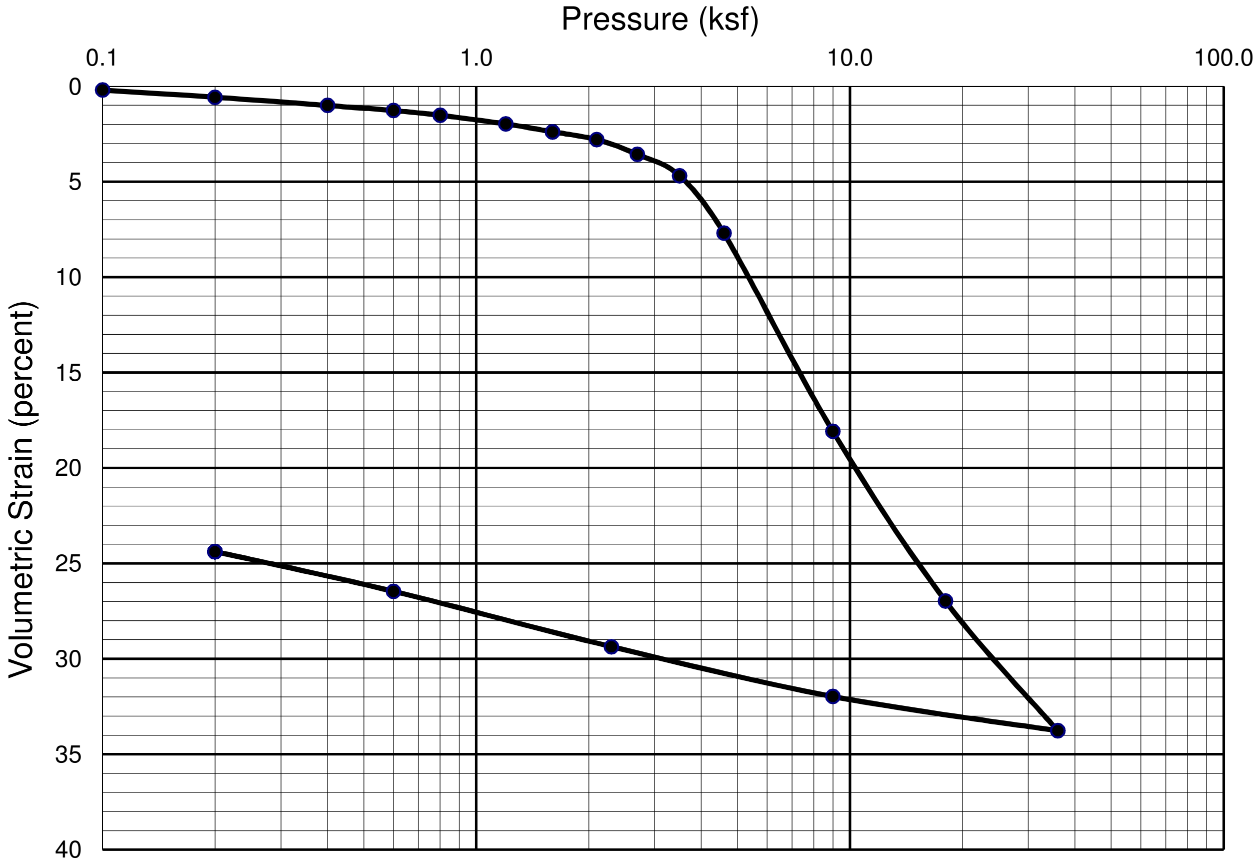
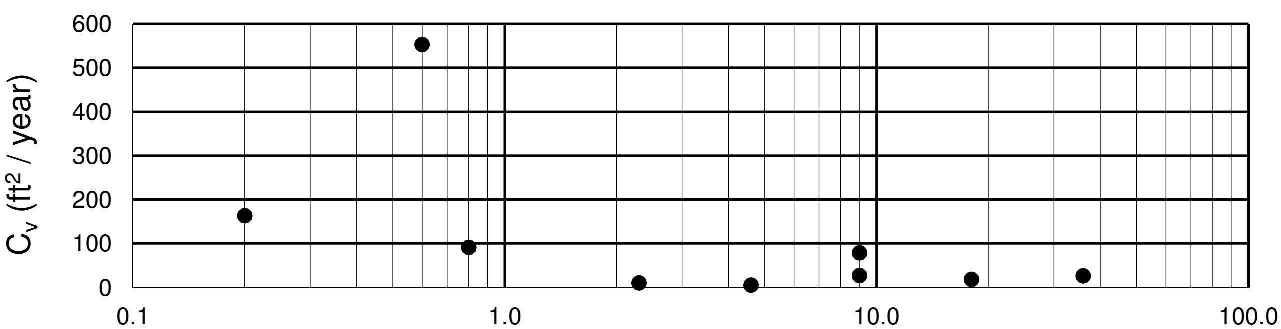
CONSOLIDATION TEST REPORT

Date	11/22/11	Project No.	750603902	Figure	C-3





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Sampler Type: Dames & Moore				Condition	Before Test			After Test	
Diameter (in) 2.42	Height (in) 1.00			Water Content	w_o	65.1 %	w_f	41.6 %	
Overburden Pressure, p_o	3,010	psf		Void Ratio	e_o	1.78	e_f	1.10	
Preconsol. Pressure, p_c	3,800	psf		Saturation	S_o	99 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.33			Dry Density	γ_d	61 pcf	γ_d	80 pcf	
LL --	PL --			PI --			G_s 2.70	(assumed)	
Classification CLAY (CH), gray				Source		B29-1 at 50 feet			

BLOCKS 29-32
MISSION BAY
San Francisco, California

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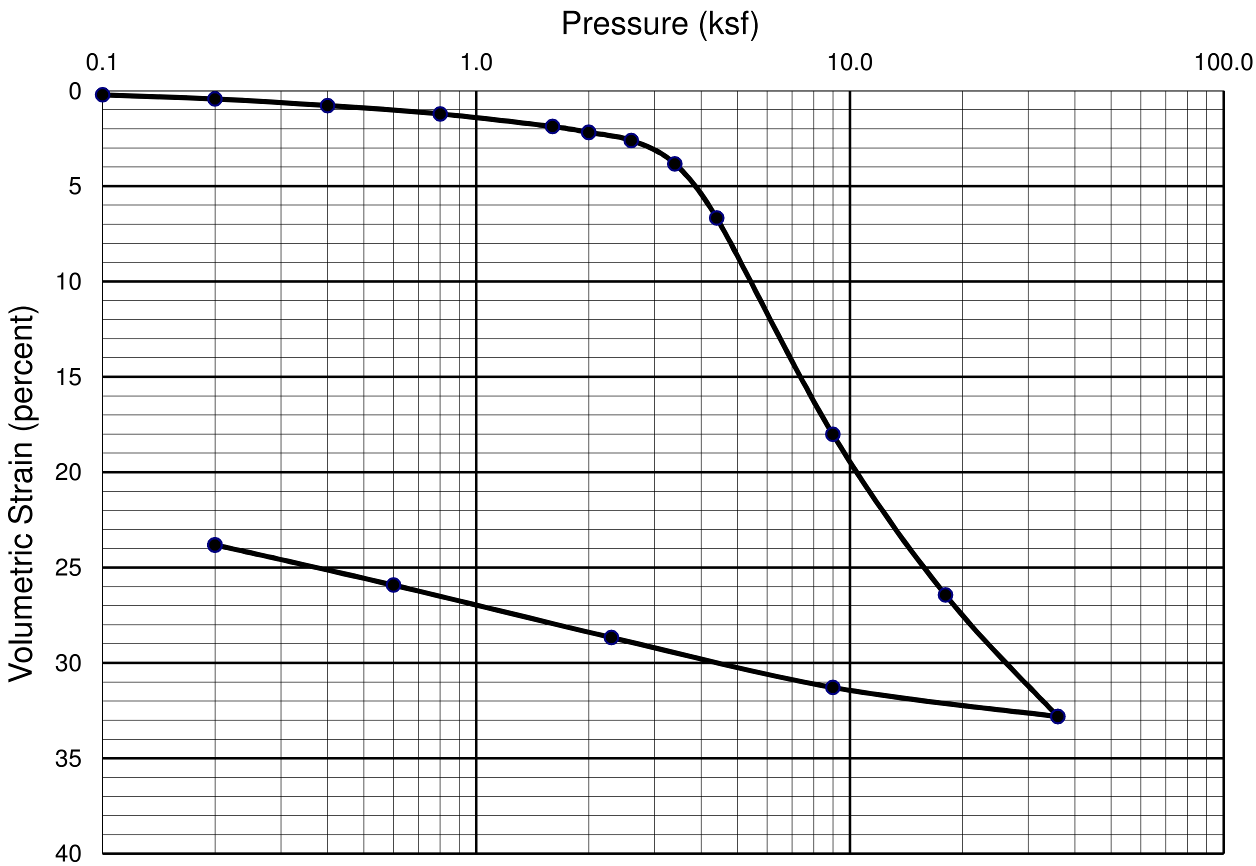
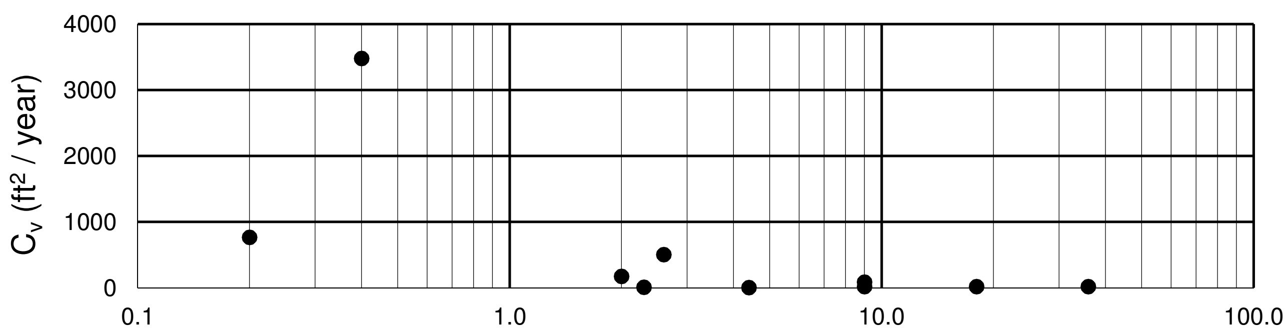
CONSOLIDATION TEST REPORT

Date 11/22/11 | Project No. 750603902 | Figure C-4





DRAFT

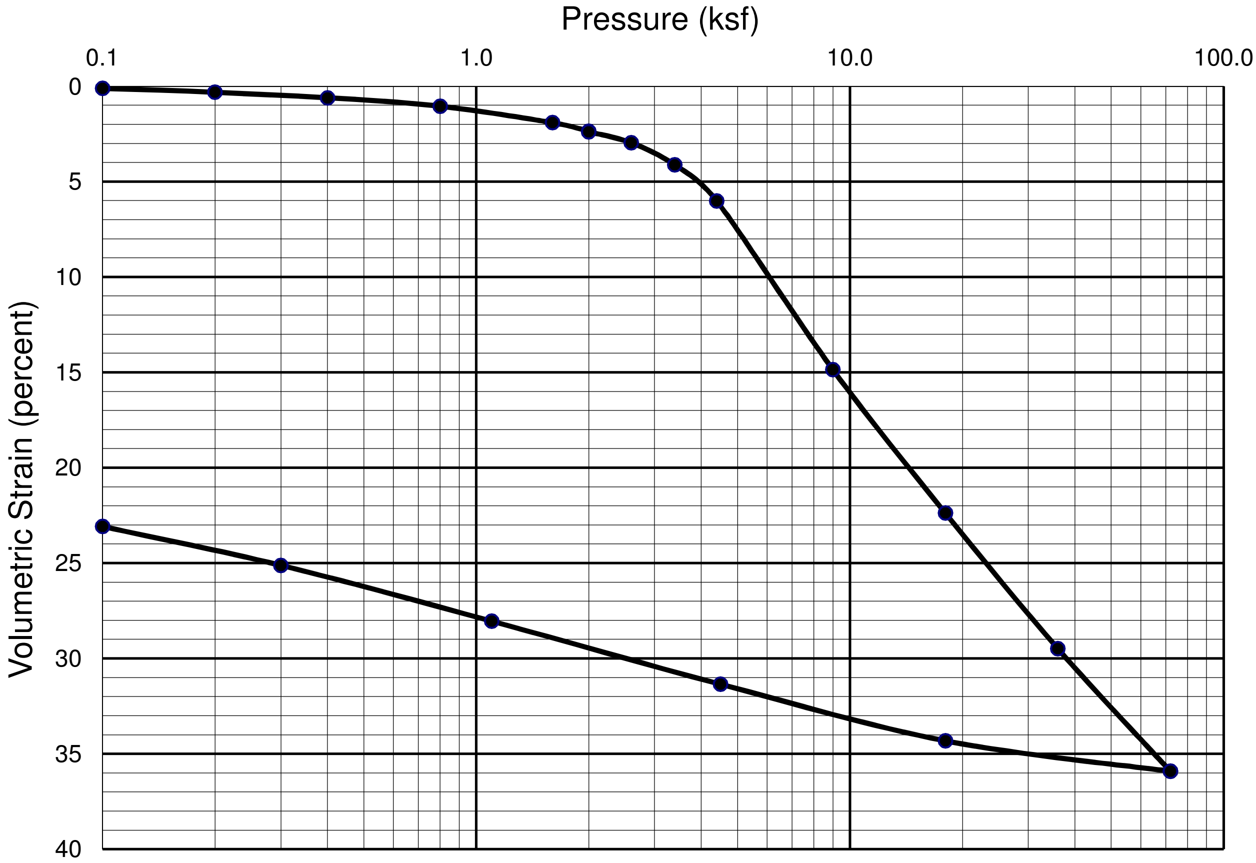
Sampler Type: Dames & Moore			Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	63.0 %	w_f	40.2 %	
Overburden Pressure, p_o	2,400	psf	Void Ratio	e_o	1.71	e_f	1.07	
Preconsol. Pressure, p_c	3,500	psf	Saturation	S_o	99 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.34		Dry Density	γ_d	62 pcf	γ_d	82 pcf	
LL --	PL --		PI --			G_s	2.70	(assumed)
Classification CLAY (CH), gray			Source		B29-2 at 30 feet			

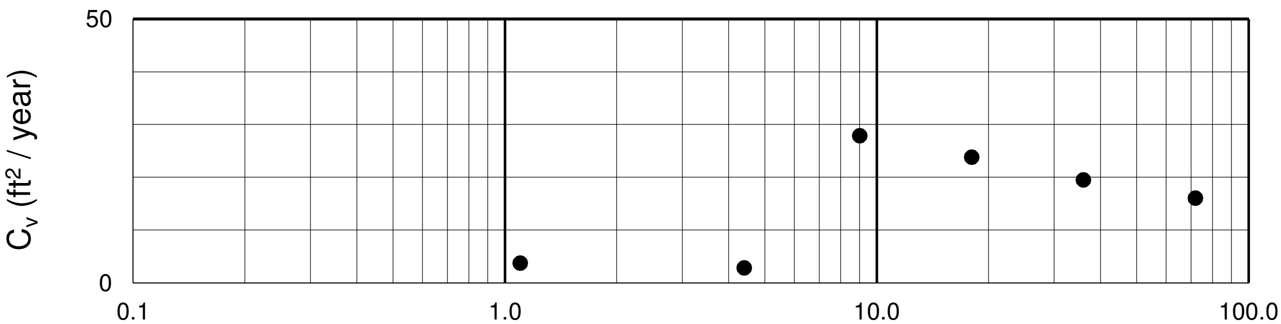
BLOCKS 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

CONSOLIDATION TEST REPORT

Date	11/22/11	Project No.	750603902	Figure	C-5





DRAFT

Sampler Type: Dames & Moore				Condition		Before Test		After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	64.1 %	w_f	42.4 %	
Overburden Pressure, p_o	2,700	psf		Void Ratio	e_o	1.80	e_f	1.15	
Preconsol. Pressure, p_c	3,800	psf		Saturation	S_o	96 %	S_f	99 %	
Compression Ratio, $C_{\varepsilon c}$	0.26			Dry Density	γ_d	60 pcf	γ_d	78 pcf	
LL --		PL --		PI --			G_s	2.70	(assumed)
Classification CLAY (CH), gray				Source		B29-2 at 38 feet			

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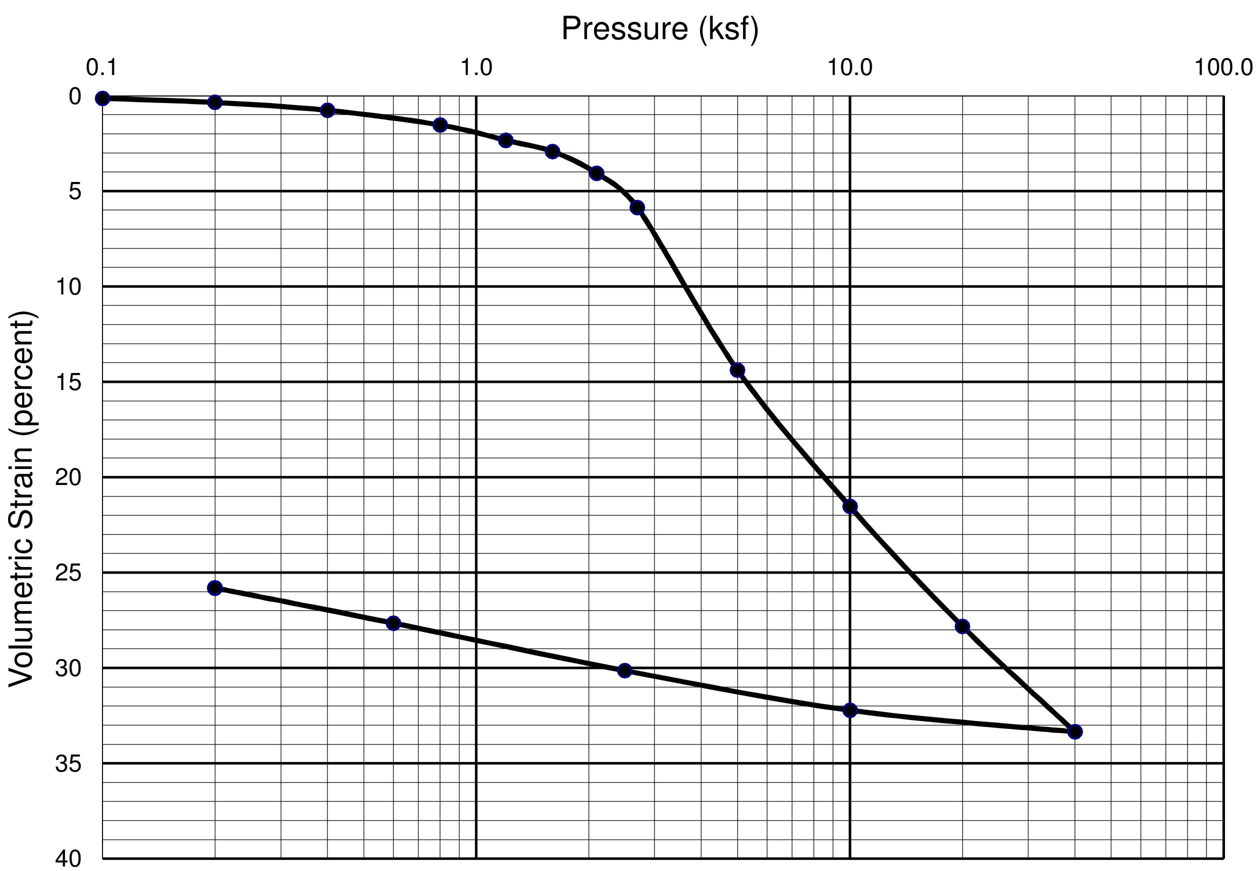
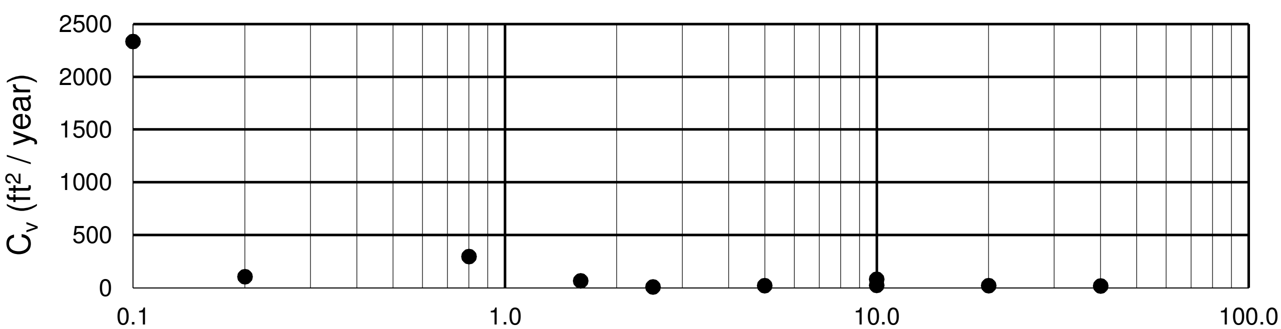
CONSOLIDATION TEST REPORT

Date	11/22/11	Project No.	750603902	Figure	C-6





DRAFT

Sampler Type: Dames & Moore		Condition	Before Test			After Test	
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	57.5 %	w_f	34.7 %	
Overburden Pressure, p_o	1,640 psf	Void Ratio	e_o	1.59	e_f	0.92	
Preconsol. Pressure, p_c	2,000 psf	Saturation	S_o	98 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.31	Dry Density	γ_d	65 pcf	γ_d	88 pcf	
LL --	PL --	PI --		G_s 2.70 (assumed)			
Classification CLAY (CH), gray		Source	B29-5 at 22 feet				

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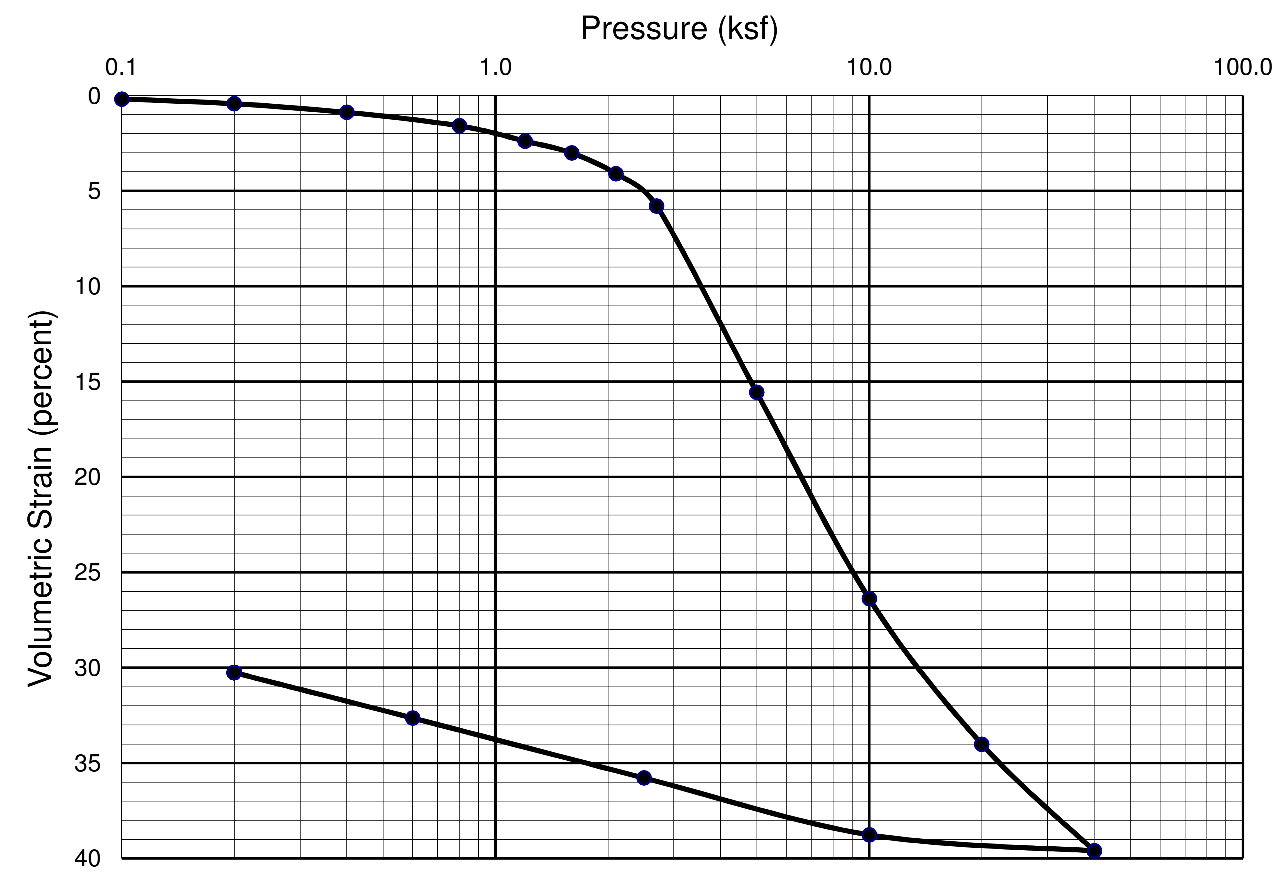
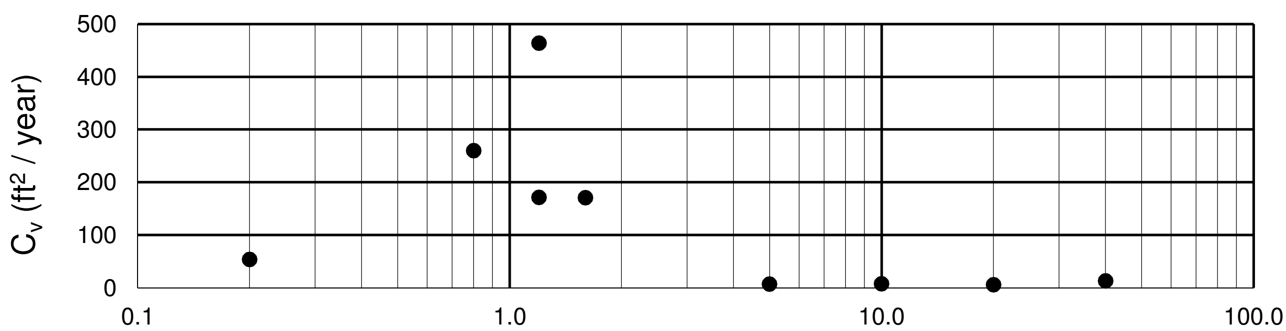
CONSOLIDATION TEST REPORT

Date 11/22/11 | Project No. 750603902 | Figure C-7





DRAFT

Sampler Type: Dames & Moore			Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	80.0 %	w_f	49.0 %	
Overburden Pressure, p_o	1,680	psf	Void Ratio	e_o	2.29	e_f	1.30	
Preconsol. Pressure, p_c	2,300	psf	Saturation	S_o	94 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.38		Dry Density	γ_d	51 pcf	γ_d	73 pcf	
LL --	PL --		PI --			G_s	2.70	(assumed)
Classification CLAY (CH), gray			Source		B29-5 at 34 feet			

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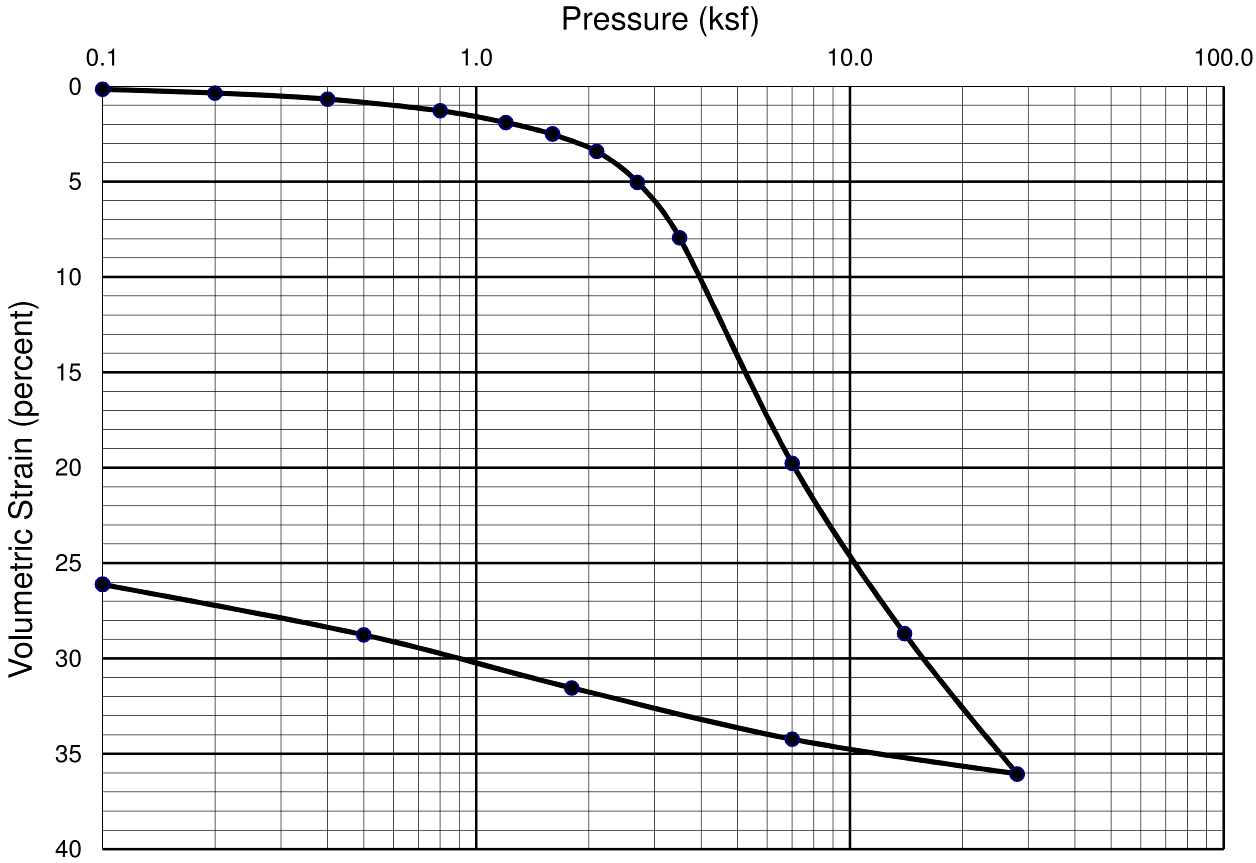
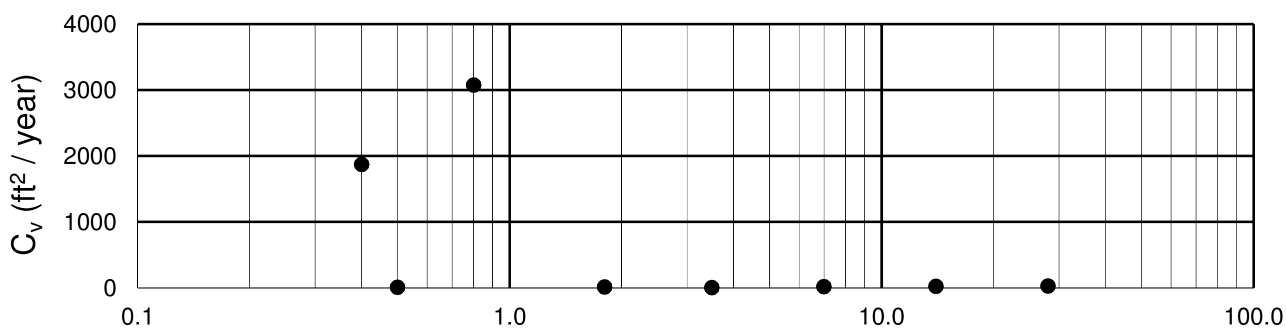
CONSOLIDATION TEST REPORT

Date	11/22/11	Project No.	750603902	Figure	C-8





DRAFT

Sampler Type: Dames & Moore		Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	74.3 %	w_f	48.1 %	
Overburden Pressure, p_o	1,940 psf	Void Ratio	e_o	2.06	e_f	1.26	
Preconsol. Pressure, p_c	2,800 psf	Saturation	S_o	97 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.32	Dry Density	γ_d	55 pcf	γ_d	74 pcf	
LL --	PL --	PI --		G_s 2.70 (assumed)			
Classification CLAY (CH), gray		Source	B29-5 at 41 feet				

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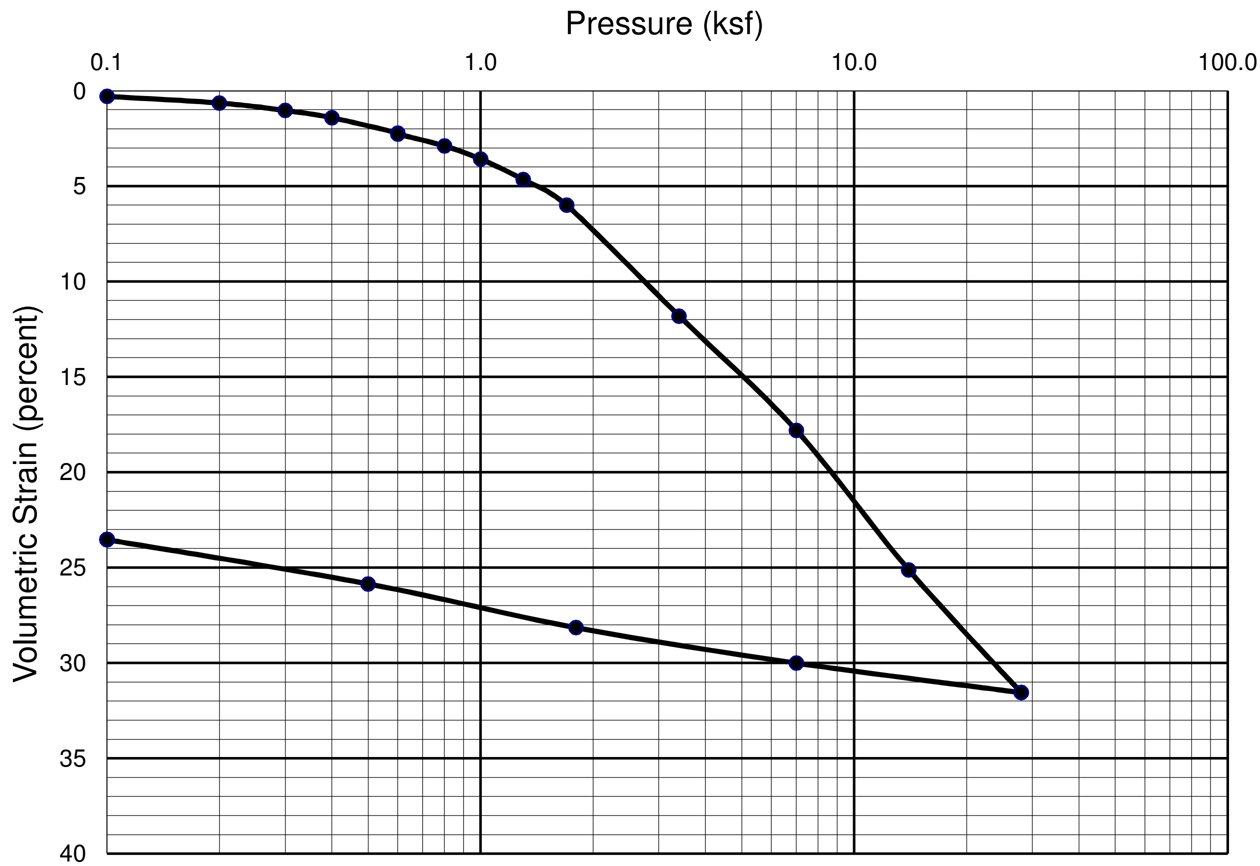
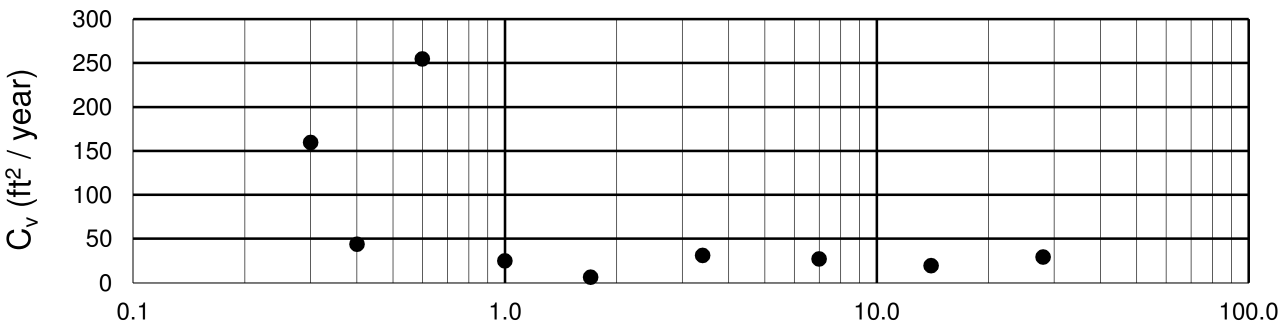
CONSOLIDATION TEST REPORT

Date 11/22/11 | Project No. 750603902 | Figure C-9





DRAFT

Sampler Type: Dames & Moore			Condition	Before Test			After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	59.0 %		w_f	39.8 %
Overburden Pressure, p_o	1,950	psf	Void Ratio	e_o	1.65		e_f	1.03
Preconsol. Pressure, p_c	1,950	psf	Saturation	S_o	96 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.22		Dry Density	γ_d	64 pcf		γ_d	83 pcf
LL --	PL --		PI --			G_s 2.70	(assumed)	
Classification CLAY (CH), gray, trace shell fragments			Source		B29-7 at 31 feet			

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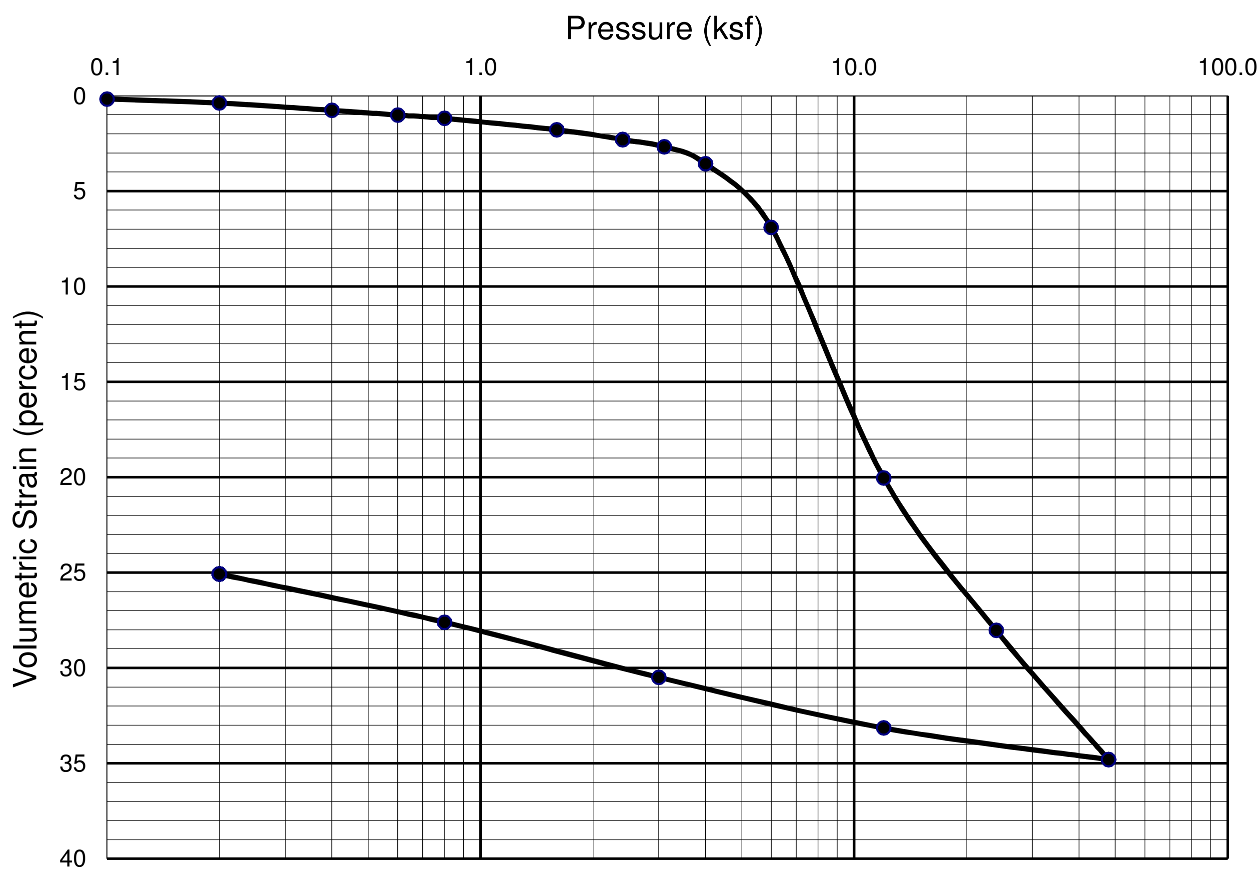
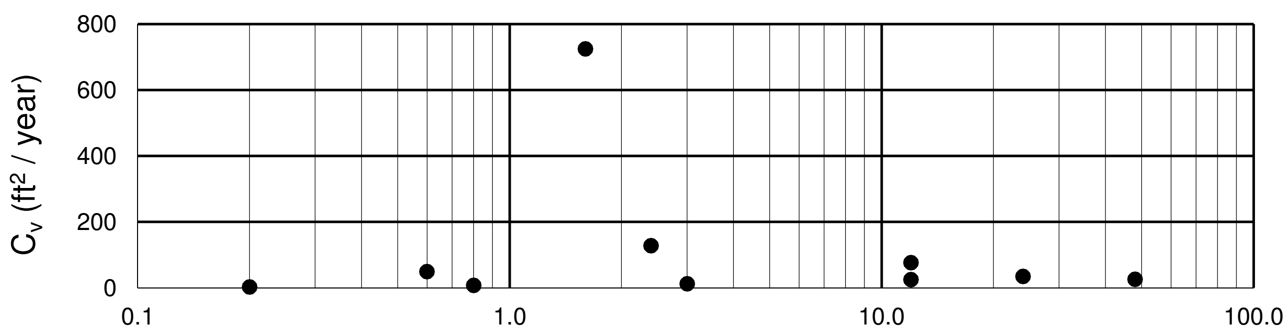
CONSOLIDATION TEST REPORT

Date	11/22/11	Project No.	750603902	Figure	C-10





DRAFT

Sampler Type: Dames & Moore			Condition	Before Test			After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	64.7 %		w_f	40.6 %
Overburden Pressure, p_o	2,540	psf	Void Ratio	e_o	1.75		e_f	1.06
Preconsol. Pressure, p_c	4,800	psf	Saturation	S_o	100 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.33		Dry Density	γ_d	61 pcf		γ_d	82 pcf
LL --	PL --		PI --			G_s 2.70	(assumed)	
Classification CLAY (CH), gray			Source	B29-7 at 42 feet				

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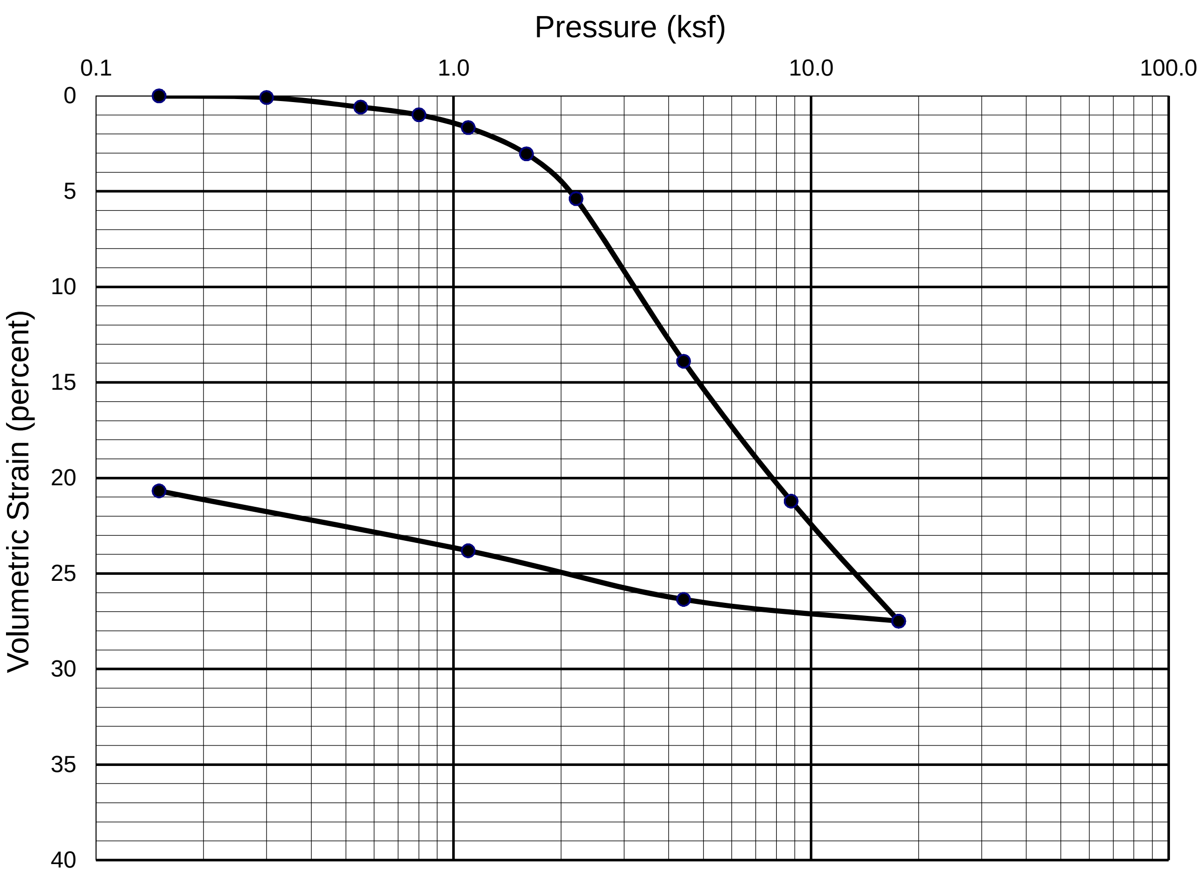
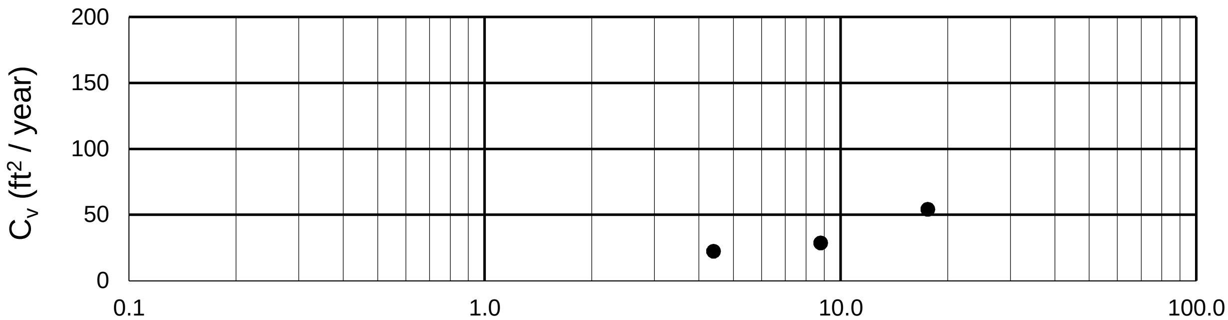
Date 11/22/11 | Project No. 750603902 | Figure C-11





DRAFT

Sampler Type: Shelby Tube		Condition		Before Test		After Test	
Diameter (in) 2.41	Height (in) 1.00	Water Content	w_o	58.6 %		w_f	42.3 %
Overburden Pressure, p_o 1,700 psf		Void Ratio	e_o	1.66		e_f	1.14
Preconsol. Pressure, p_c 1,900 psf		Saturation	S_o	95 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$ 0.26		Dry Density	γ_d	63 pcf		γ_d	79 pcf
LL --	PL --		PI --		G_s 2.70 (assumed)		
Classification CLAY (CH), blue-gray			Source	B30-1 at 28 feet			

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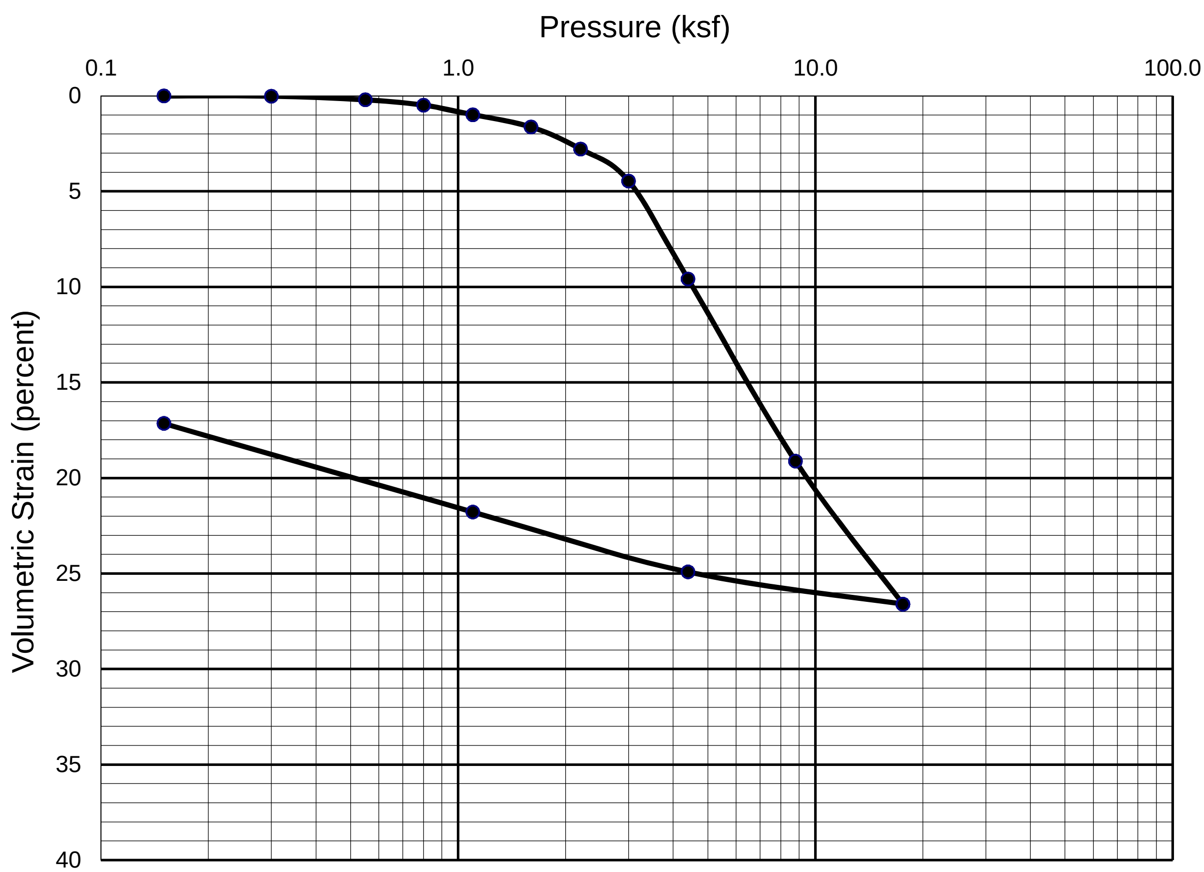
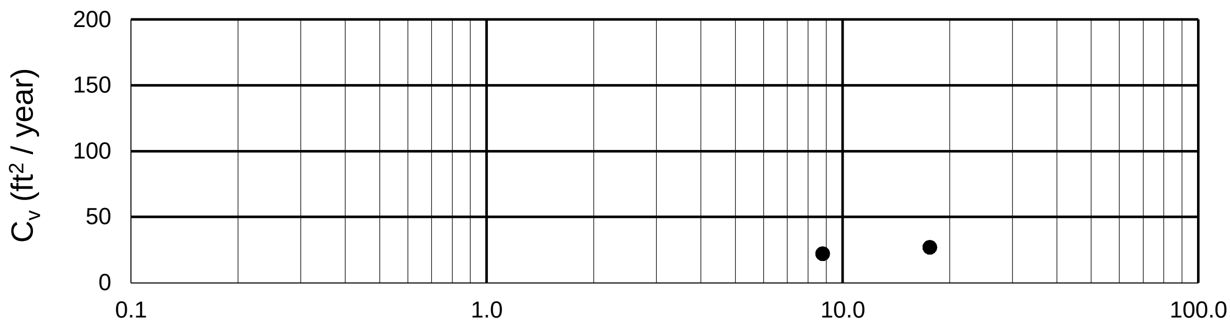
CONSOLIDATION TEST REPORT

Date 11/11/11 | Project No. 750603902 | Figure C-12





DRAFT

Sampler Type: Dames & Moore				Condition		Before Test		After Test	
Diameter (in) 2.41	Height (in) 1.01			Water Content	w_o	63.4 %	w_f	47.7 %	
Overburden Pressure, p_o	1,800	psf		Void Ratio	e_o	1.73	e_f	1.29	
Preconsol. Pressure, p_c	2,100	psf		Saturation	S_o	99 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.31			Dry Density	γ_d	62 pcf	γ_d	74 pcf	
LL --	PL --			PI --			G_s	2.70	(assumed)
Classification CLAY (CH), gray				Source		B30-3 at 24 feet			

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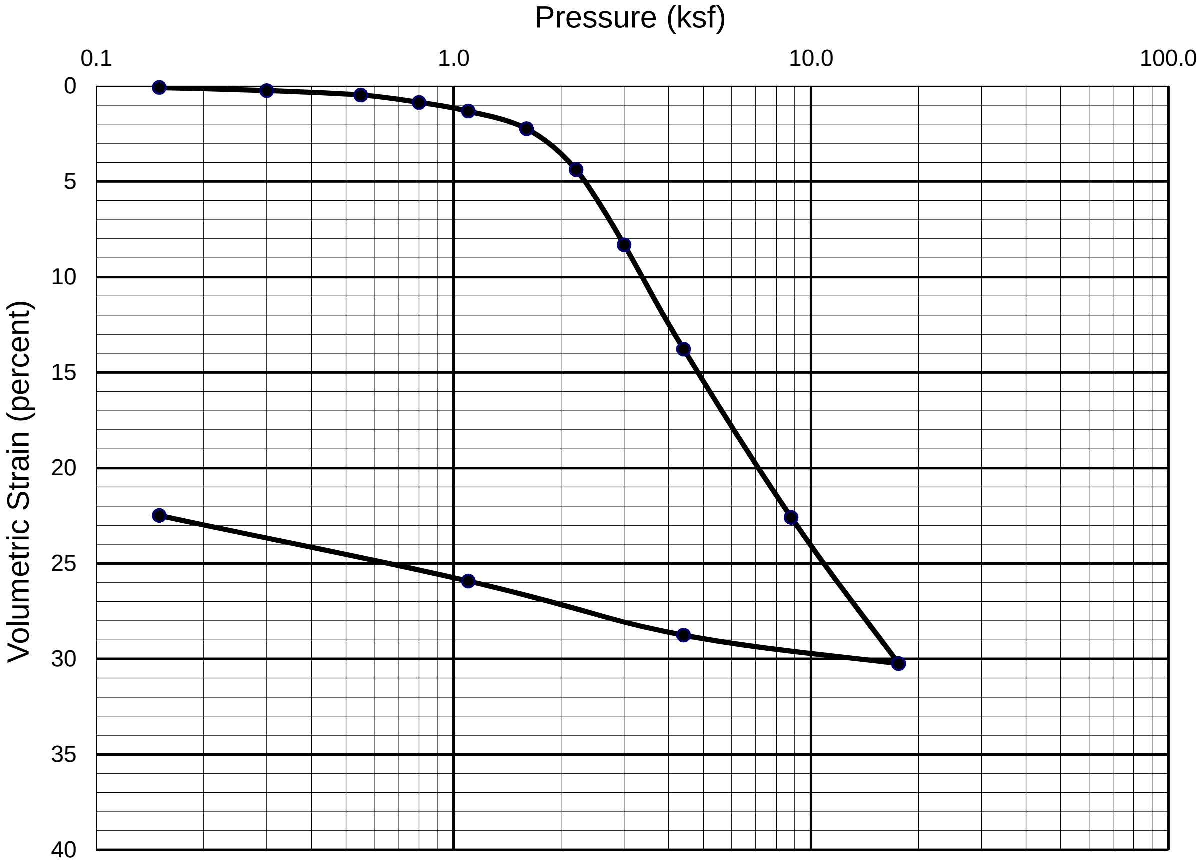
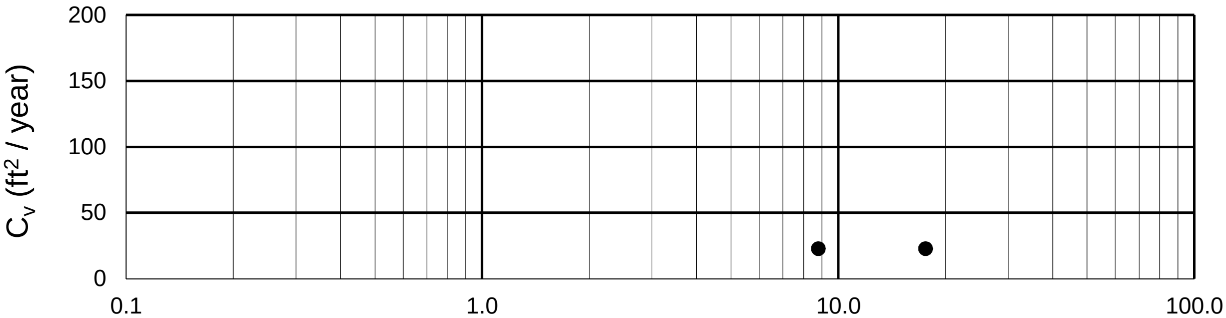
Date 11/11/11 | Project No. 750603902 | Figure C-13





DRAFT

Sampler Type: Shelby Tube			Condition		Before Test		After Test	
Diameter (in) 2.41	Height (in) 1.01		Water Content	w_o	72.0 %	w_f	48.3 %	
Overburden Pressure, p_o	2,550	psf	Void Ratio	e_o	1.96	e_f	1.30	
Preconsol. Pressure, p_c	2,600	psf	Saturation	S_o	99 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.29		Dry Density	γ_d	57 pcf	γ_d	73 pcf	
LL --	PL --		PI --			G_s	2.70	(assumed)
Classification CLAY (CH), gray			Source		B30-3 at 44 feet			

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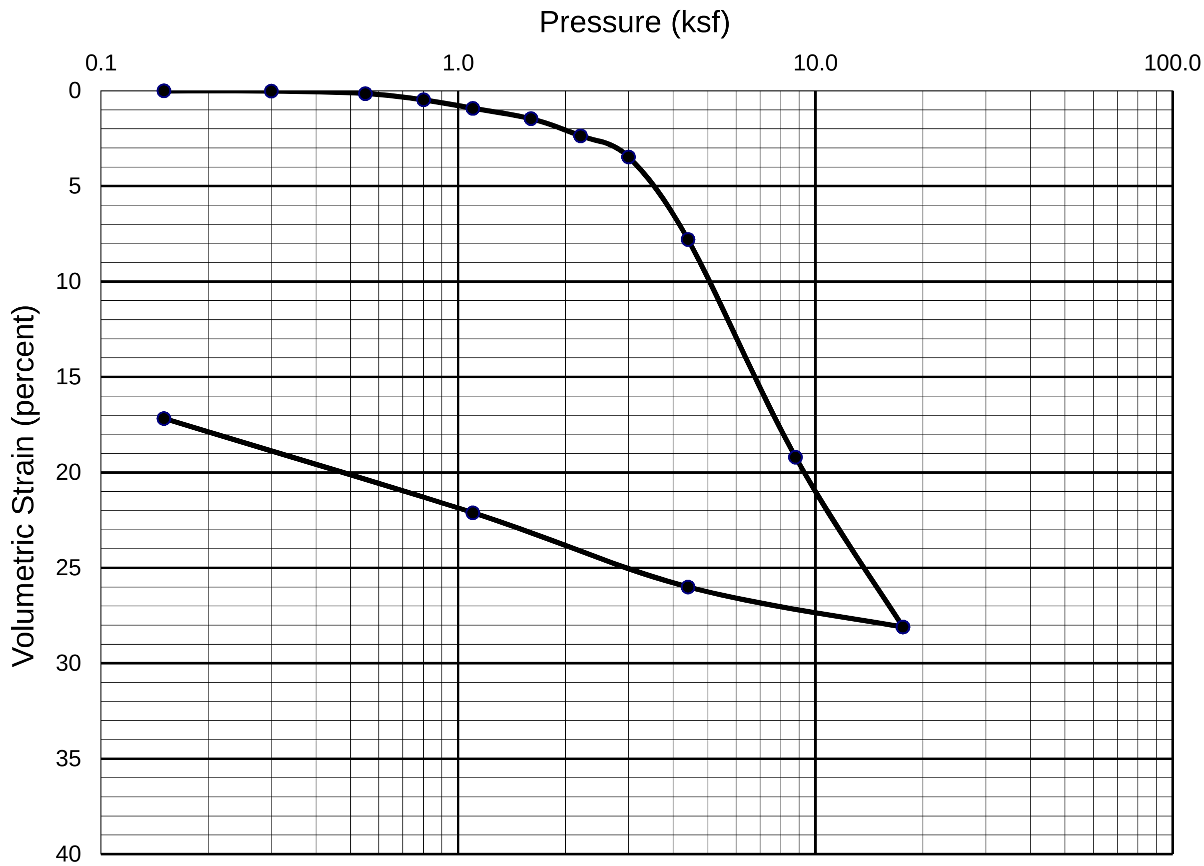
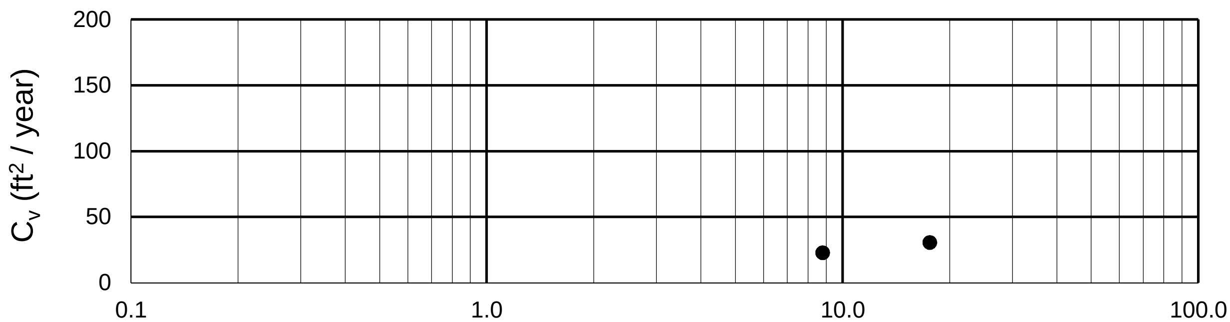
CONSOLIDATION TEST REPORT

Date	11/11/11	Project No.	750603902	Figure	C-14





DRAFT

Sampler Type: Shelby Tube				Condition		Before Test			After Test	
Diameter (in)	2.41	Height (in)	1.01	Water Content	w_o	74.4	%	w_f	56.3	%
Overburden Pressure, p_o	2,450	psf		Void Ratio	e_o	2.02		e_f	1.52	
Preconsol. Pressure, p_c	3,300	psf		Saturation	S_o	100	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.35			Dry Density	γ_d	56	pcf	γ_d	67	pcf

LL --	PL --	PI --	G_s	2.70	(assumed)	
Classification	CLAY (CH), gray	Source	B30-4 at 39 feet			

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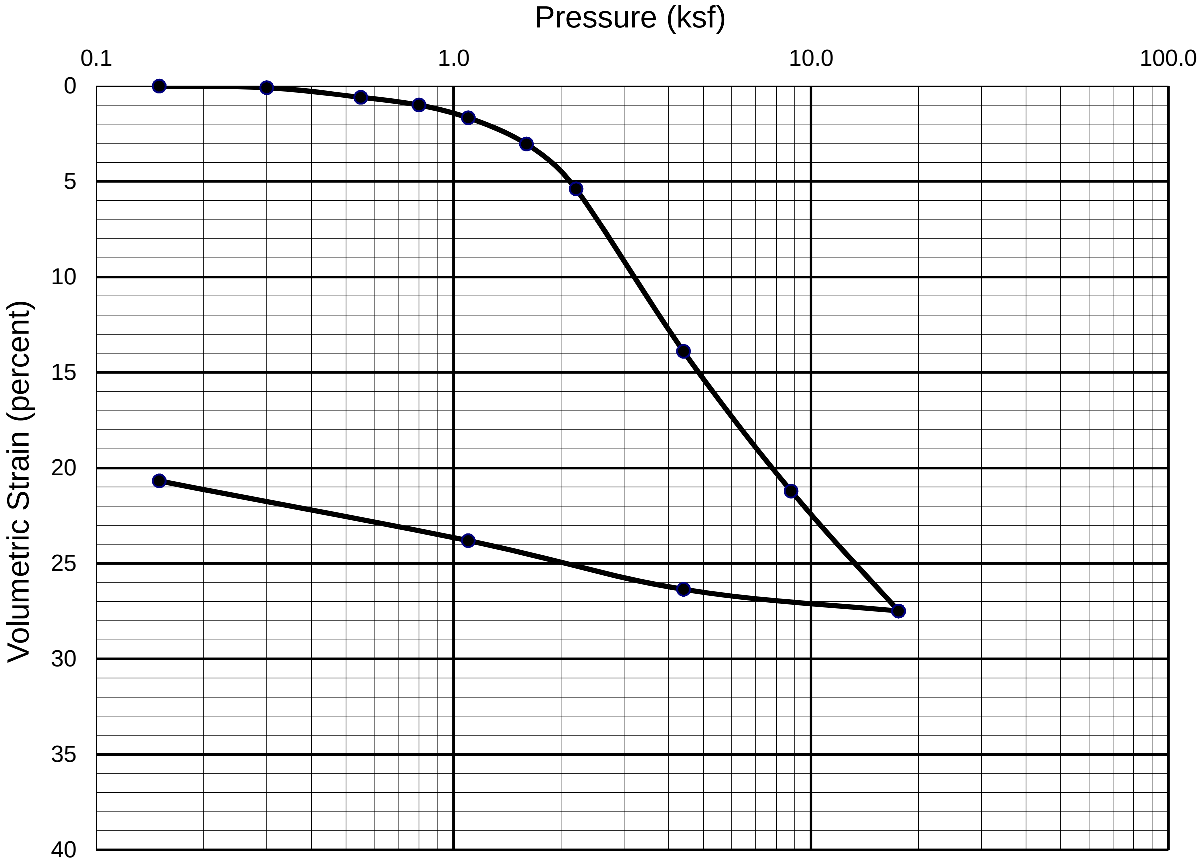
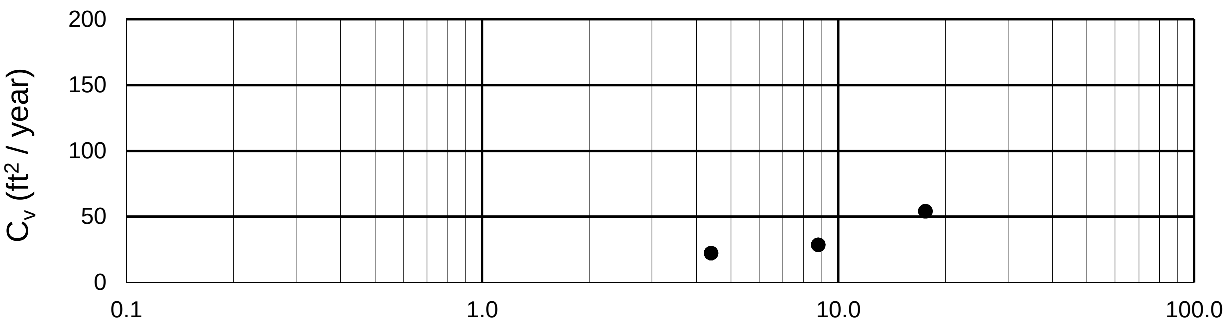
CONSOLIDATION TEST REPORT

Date	11/11/11	Project No.	750603902	Figure	C-15





DRAFT

Sampler Type: Shelby Tube			Condition		Before Test		After Test	
Diameter (in) 2.41	Height (in) 1.00		Water Content	w_o	66.8 %	w_f	43.9 %	
Overburden Pressure, p_o	1,650	psf	Void Ratio	e_o	1.83	e_f	1.18	
Preconsol. Pressure, p_c	2,000	psf	Saturation	S_o	98 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.31		Dry Density	γ_d	60 pcf	γ_d	77 pcf	
LL --	PL --		PI --			G_s 2.70	(assumed)	
Classification CLAY (CH), gray			Source	B32-1 at 16.5 feet				

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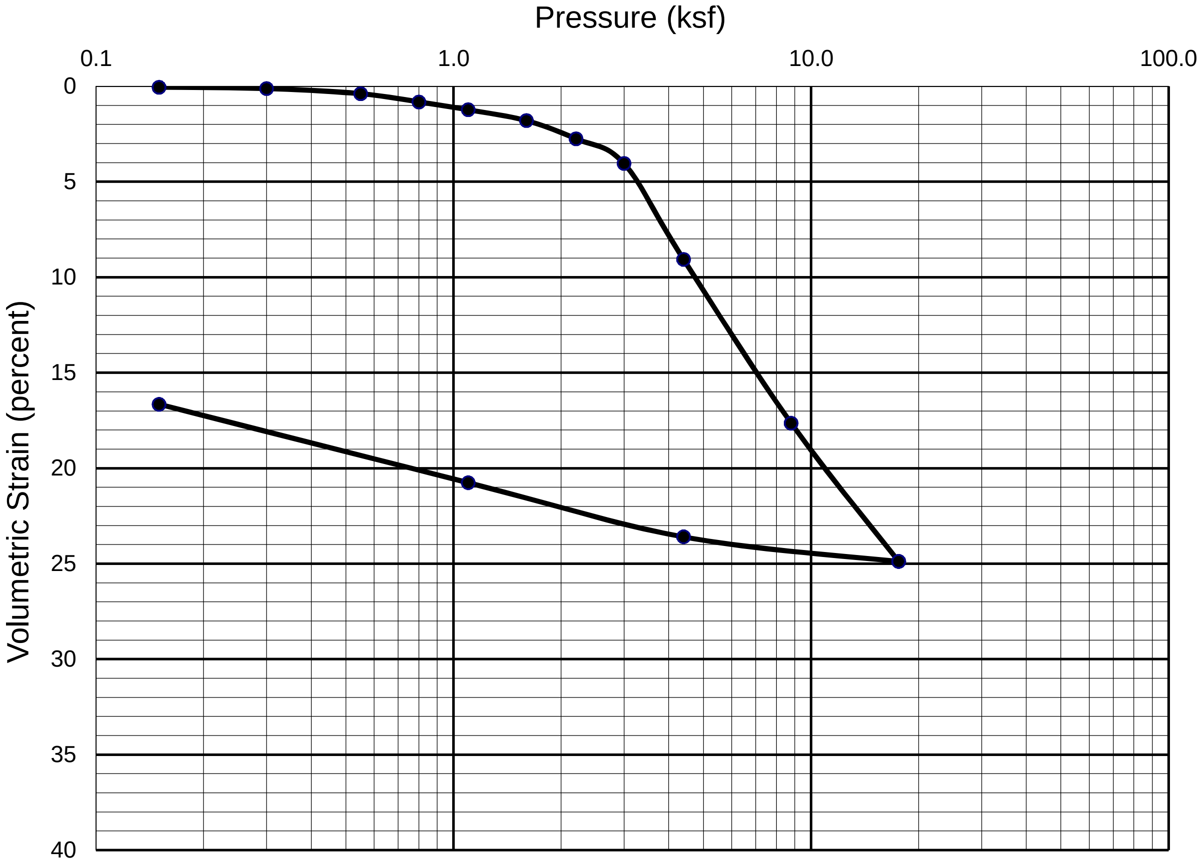
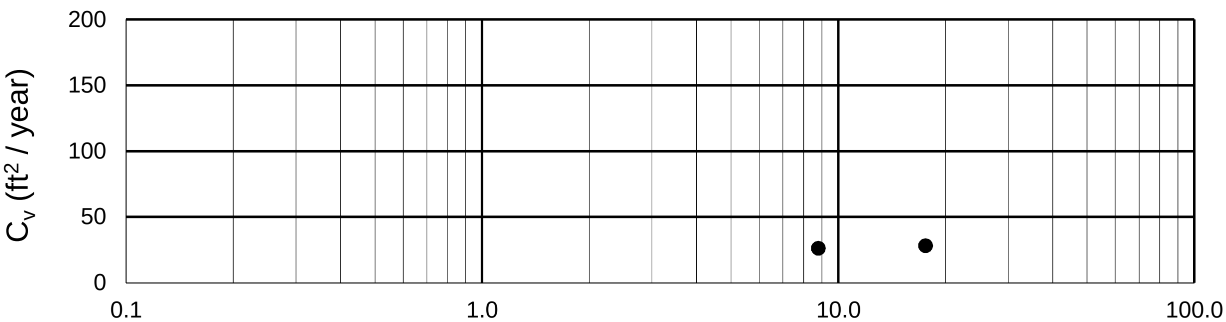
CONSOLIDATION TEST REPORT

Date 11/11/11 | Project No. 750603902 | Figure C-16





DRAFT

Sampler Type: Shelby Tube		Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.01	Water Content	w_o	57.6 %	w_f	43.3 %	
Overburden Pressure, p_o 1,900 psf		Void Ratio	e_o	1.56	e_f	1.17	
Preconsol. Pressure, p_c 2,900 psf		Saturation	S_o	100 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$ 0.29		Dry Density	γ_d	66 pcf	γ_d	78 pcf	
LL --	PL --	PI --		G_s 2.70 (assumed)			
Classification CLAY (CH), gray		Source B32-1 at 24 feet					

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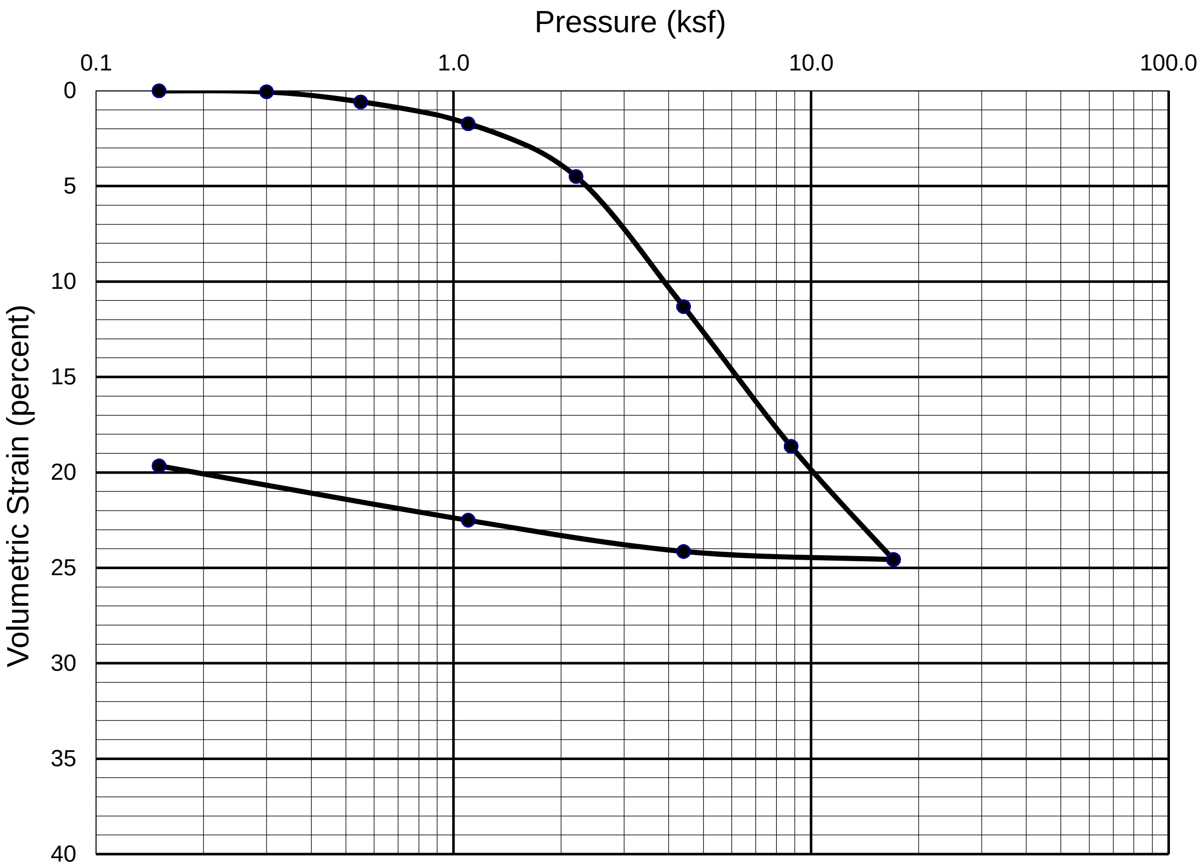
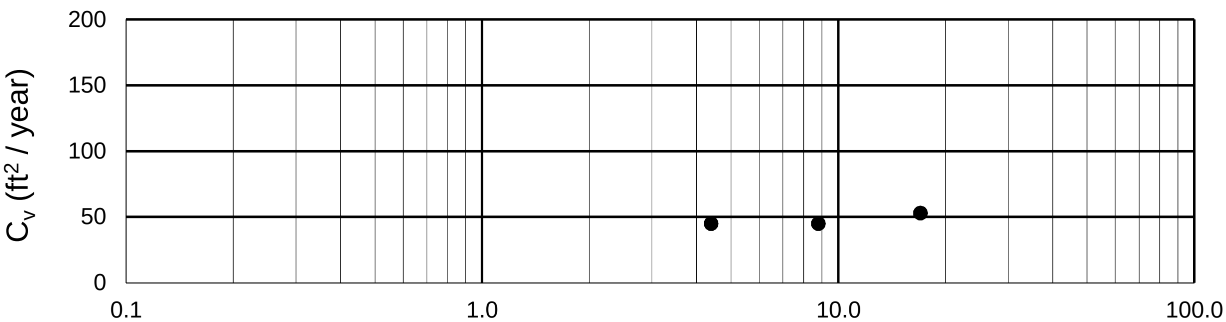
CONSOLIDATION TEST REPORT

Date 11/11/11 | Project No. 750603902 | Figure C-17





DRAFT

Sampler Type: Shelby Tube				Condition		Before Test		After Test	
Diameter (in) 2.41		Height (in) 1.00		Water Content	w_o	50.9	%	w_f	35.9 %
Overburden Pressure, p_o	1,600	psf		Void Ratio	e_o	1.39		e_f	0.97
Preconsol. Pressure, p_c	2,000	psf		Saturation	S_o	99	%	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.25			Dry Density	γ_d	71	pcf	γ_d	86 pcf
LL --	PL --			PI --				G_s 2.70	(assumed)
Classification CLAY (CH), gray				Source		B32-3 at 24 feet			

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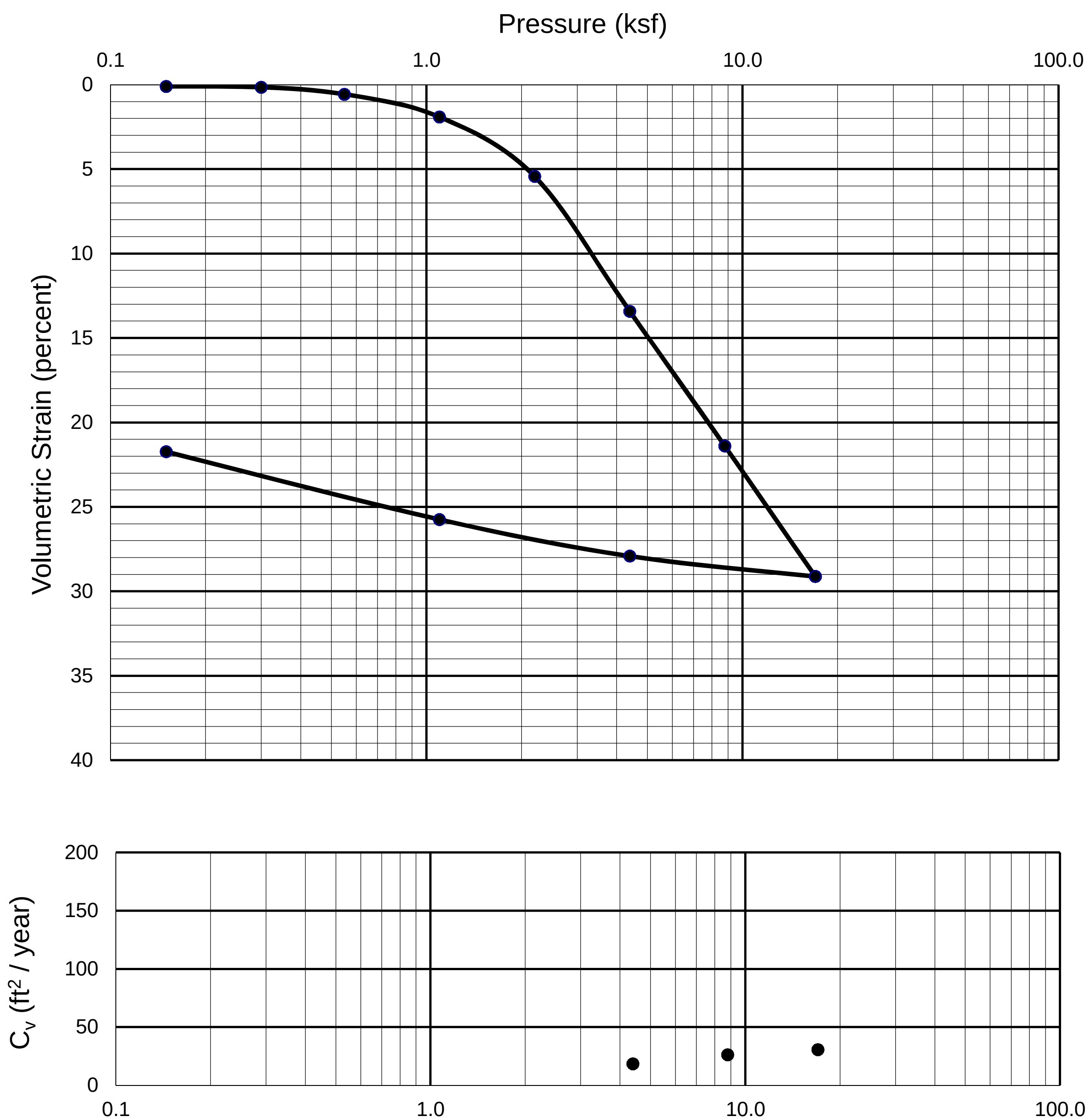
CONSOLIDATION TEST REPORT

Date	11/11/11	Project No.	750603902	Figure	C-18



DRAFT

Sampler Type: Shelby Tube			Condition		Before Test		After Test	
Diameter (in) 2.41	Height (in) 1.01		Water Content	w_o	50.9 %		w_f	49.4 %
Overburden Pressure, p_o	1,350	psf	Void Ratio	e_o	1.93		e_f	1.33
Preconsol. Pressure, p_c	1,700	psf	Saturation	S_o	100 %		S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.28		Dry Density	γ_d	58 pcf		γ_d	72 pcf
LL --	PL --		PI --			G_s 2.70 (assumed)		
Classification CLAY (CH), gray			Source	B32-4 at 16.5 feet				

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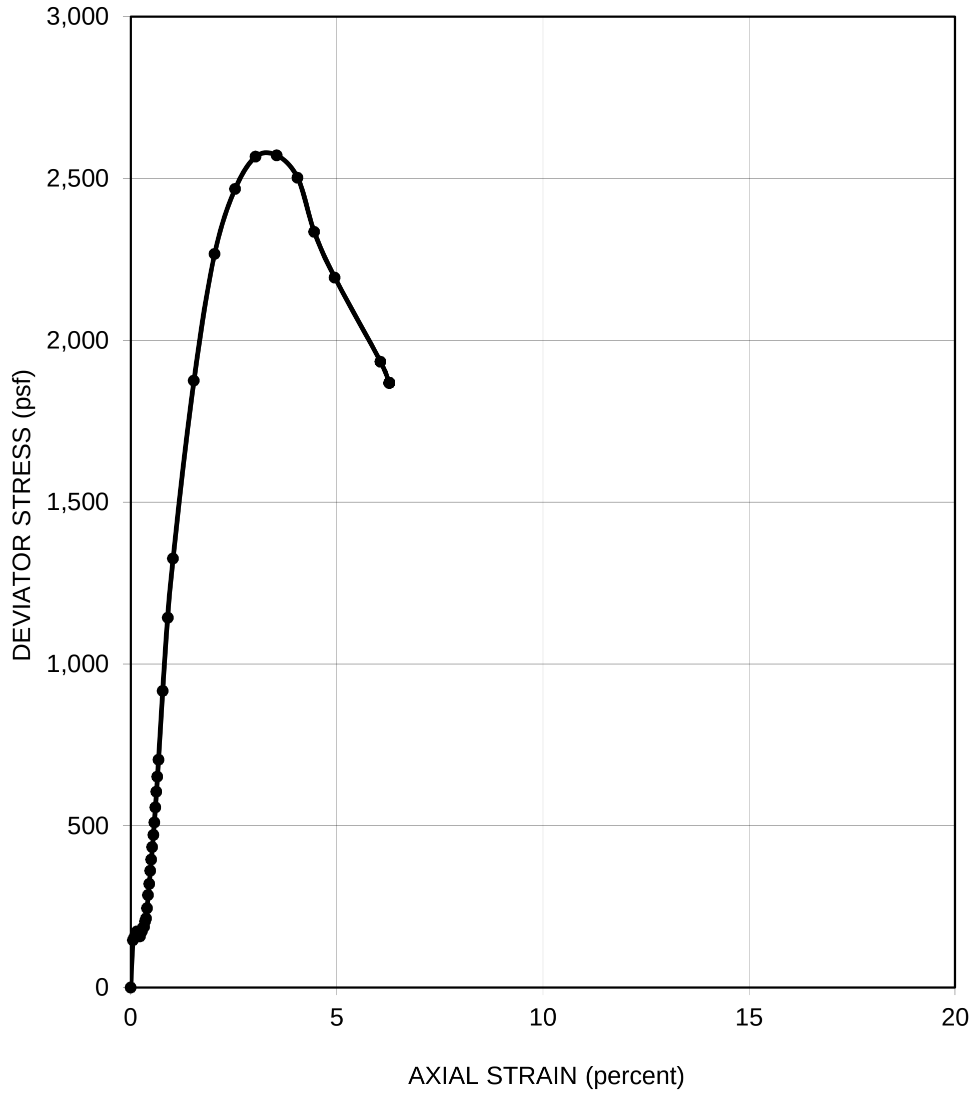
CONSOLIDATION TEST REPORT

Date	11/11/11	Project No.	750603902	Figure	C-19



DRAFT

SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	1,290	psf
DIAMETER (in.)	2.43	HEIGHT (in.) 5.51		STRAIN AT FAILURE	3.5	%
MOISTURE CONTENT		66.0	%	CONFINING PRESSURE	2,700	psf
DRY DENSITY		59	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CH), gray			SOURCE	B29-2 at 38 feet	

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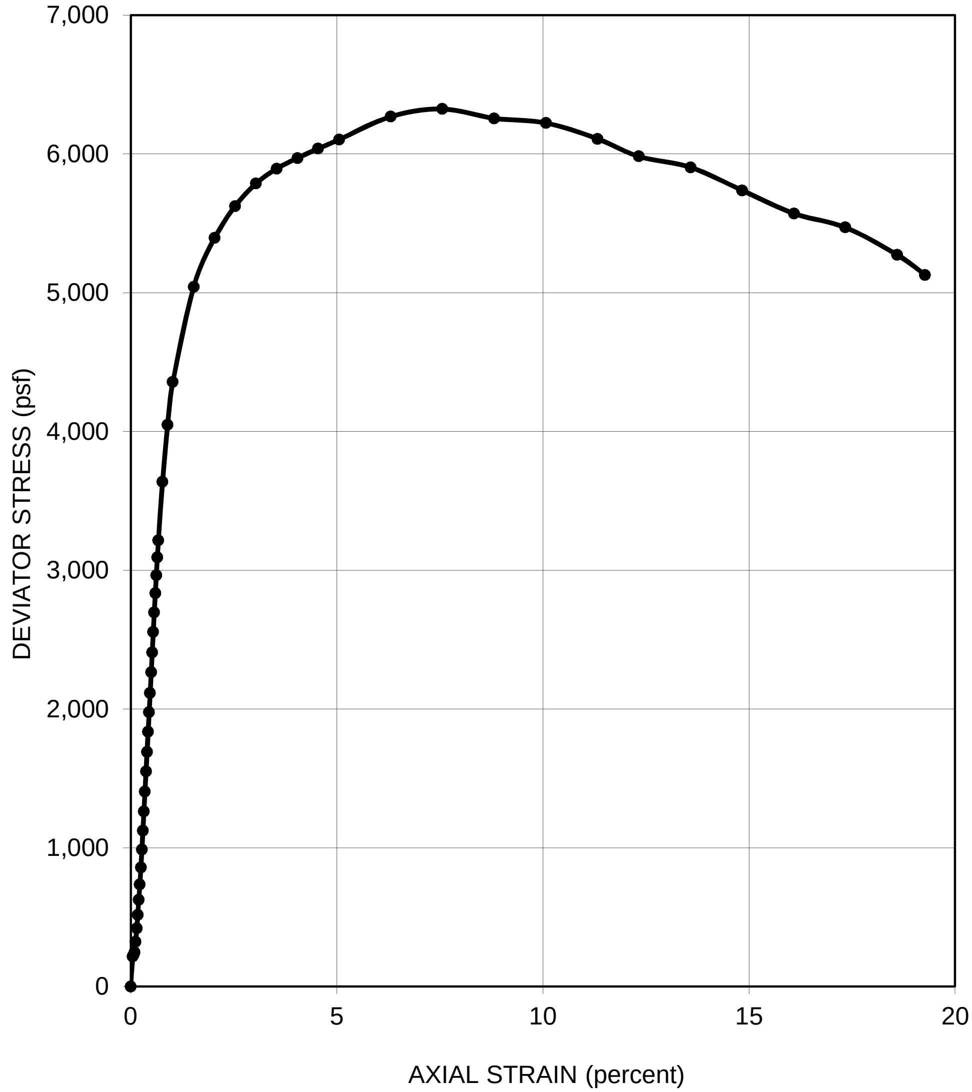
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 11/11/11 | Project No.750603902 | Figure C-20



DRAFT

SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	3,160	psf
DIAMETER (in.)	2.43	HEIGHT (in.) 5.97		STRAIN AT FAILURE	7.6	%
MOISTURE CONTENT		21.4	%	CONFINING PRESSURE	3,700	psf
DRY DENSITY		108	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY with SAND (CL) yellowish-brown with gray mottling			SOURCE	B29-2 at 56 feet	

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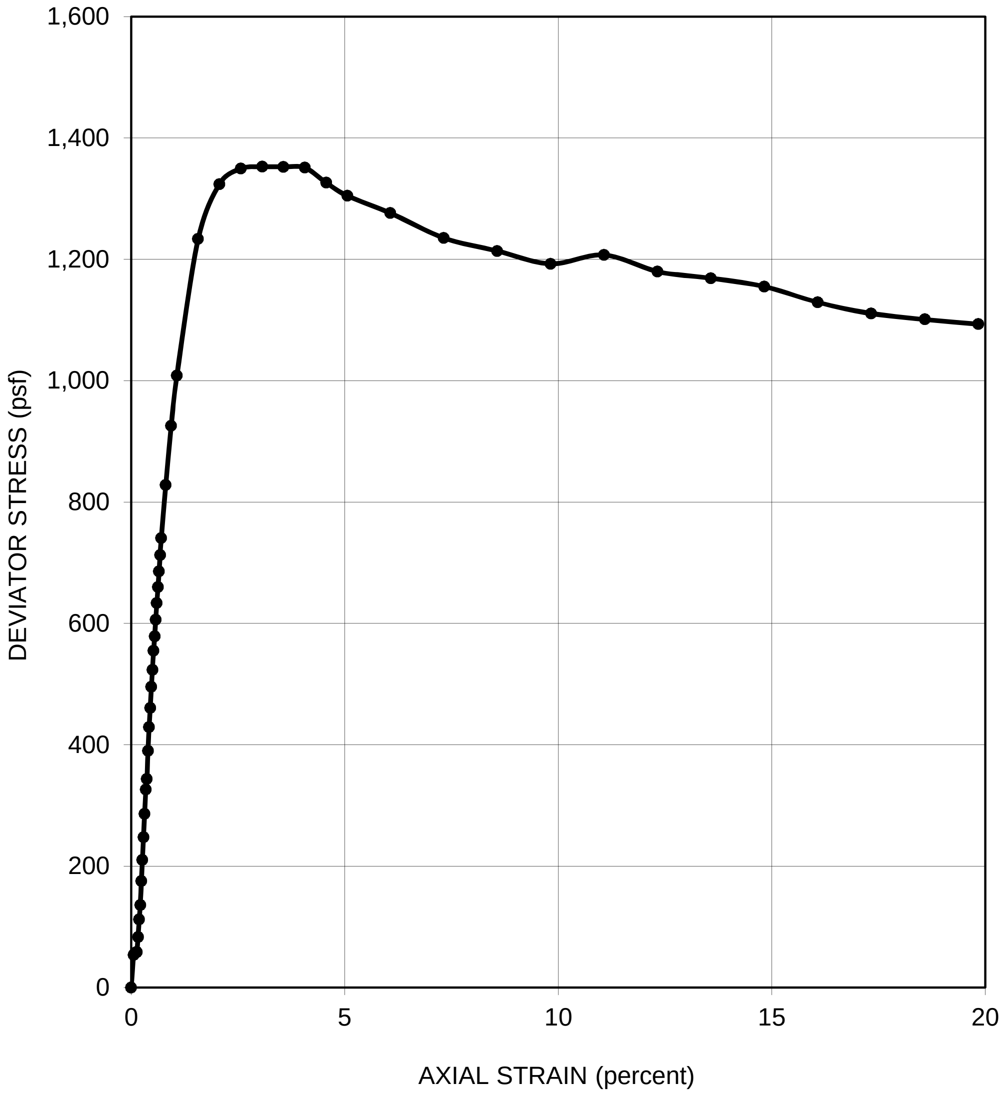
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date	11/11/11	Project No.	750603902	Figure	C-21



DRAFT

SAMPLER TYPE	Dames and Moore		SHEAR STRENGTH	680	psf
DIAMETER (in.) 2.42	HEIGHT (in.) 5.61		STRAIN AT FAILURE	3.1	%
MOISTURE CONTENT	54.7	%	CONFINING PRESSURE	1,550	psf
DRY DENSITY	67	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), gray		SOURCE	B29-4 at 17 feet	

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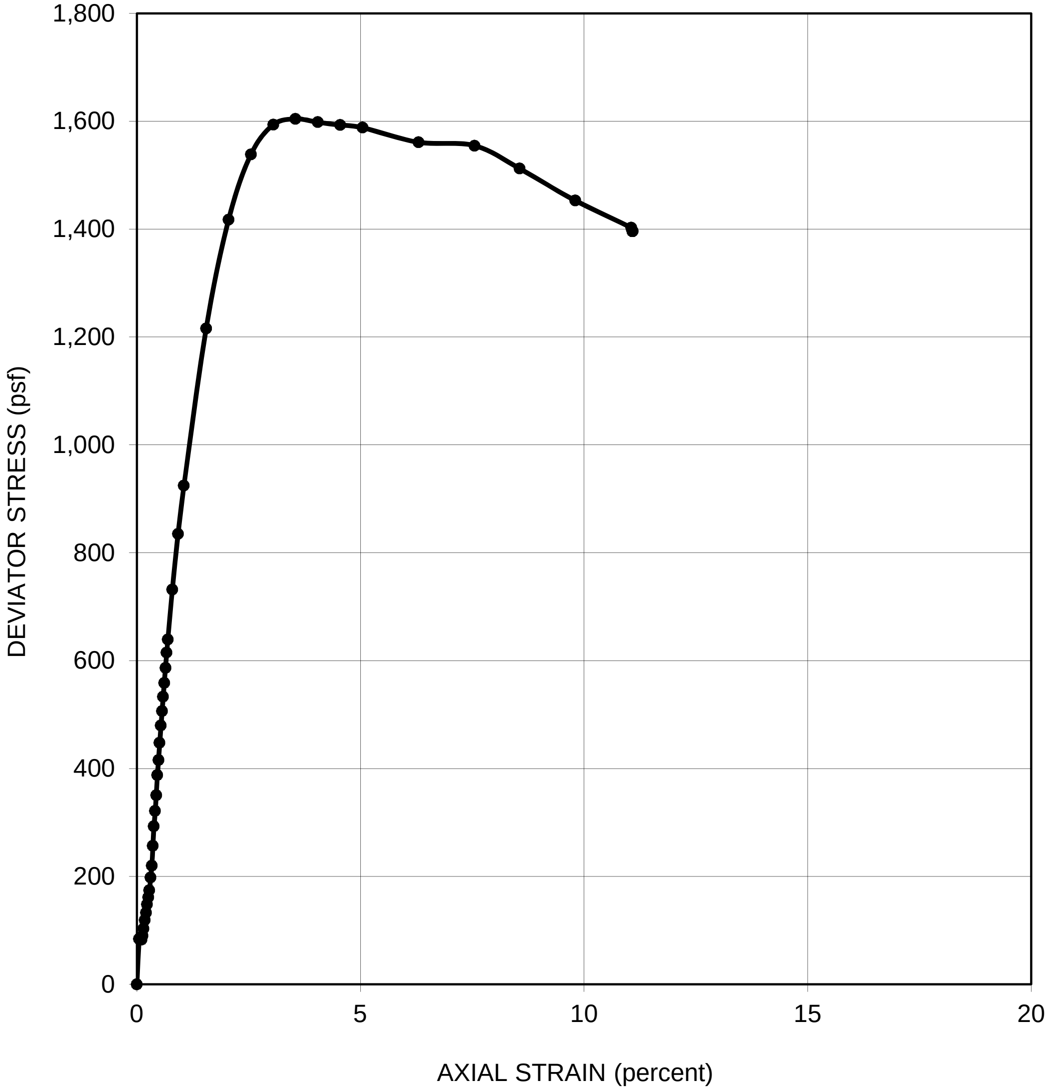
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 11/11/11 | Project No. 750603902 | Figure C-22



DRAFT

SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	800	psf
DIAMETER (in.)	2.42	HEIGHT (in.) 5.55		STRAIN AT FAILURE	3.6	%
MOISTURE CONTENT		45.8	%	CONFINING PRESSURE	2,075	psf
DRY DENSITY		71	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), gray			SOURCE	B29-4 at 31 feet	

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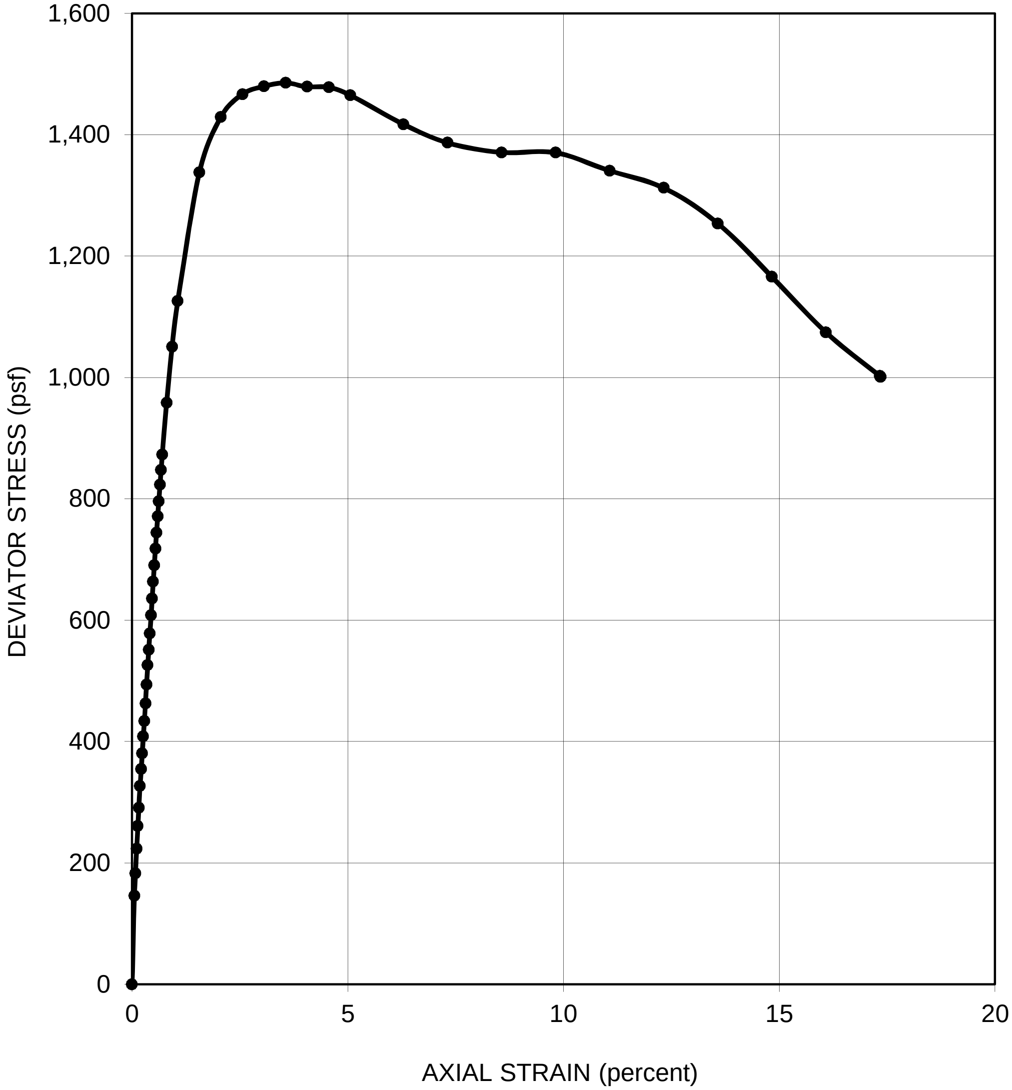
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date	11/11/11	Project No.	750603902	Figure	C-23



DRAFT

SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	740	psf
DIAMETER (in.)	2.42	HEIGHT (in.) 5.61		STRAIN AT FAILURE	3.6	%
MOISTURE CONTENT		47.1	%	CONFINING PRESSURE	2,600	psf
DRY DENSITY		72	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), gray				SOURCE	B29-4 at 45 feet

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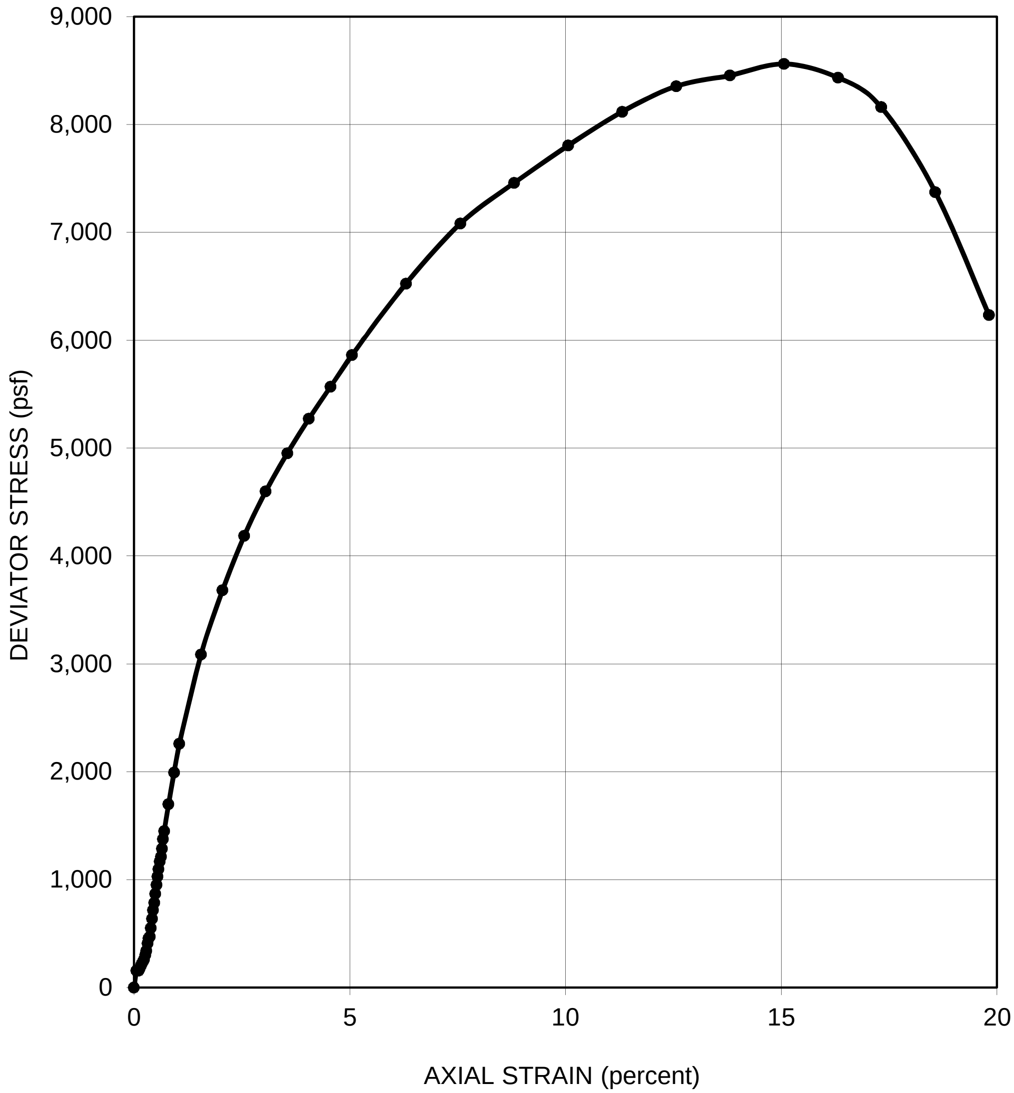
Date	11/11/11	Project No.	750603902	Figure	C-24



DRAFT

SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	4,280	psf
DIAMETER (in.)	2.42	HEIGHT (in.)	5.54	STRAIN AT FAILURE	15.1	%
MOISTURE CONTENT		16.4	%	CONFINING PRESSURE	3,750	psf
DRY DENSITY		115	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), gray				SOURCE	B29-4 at 64 feet

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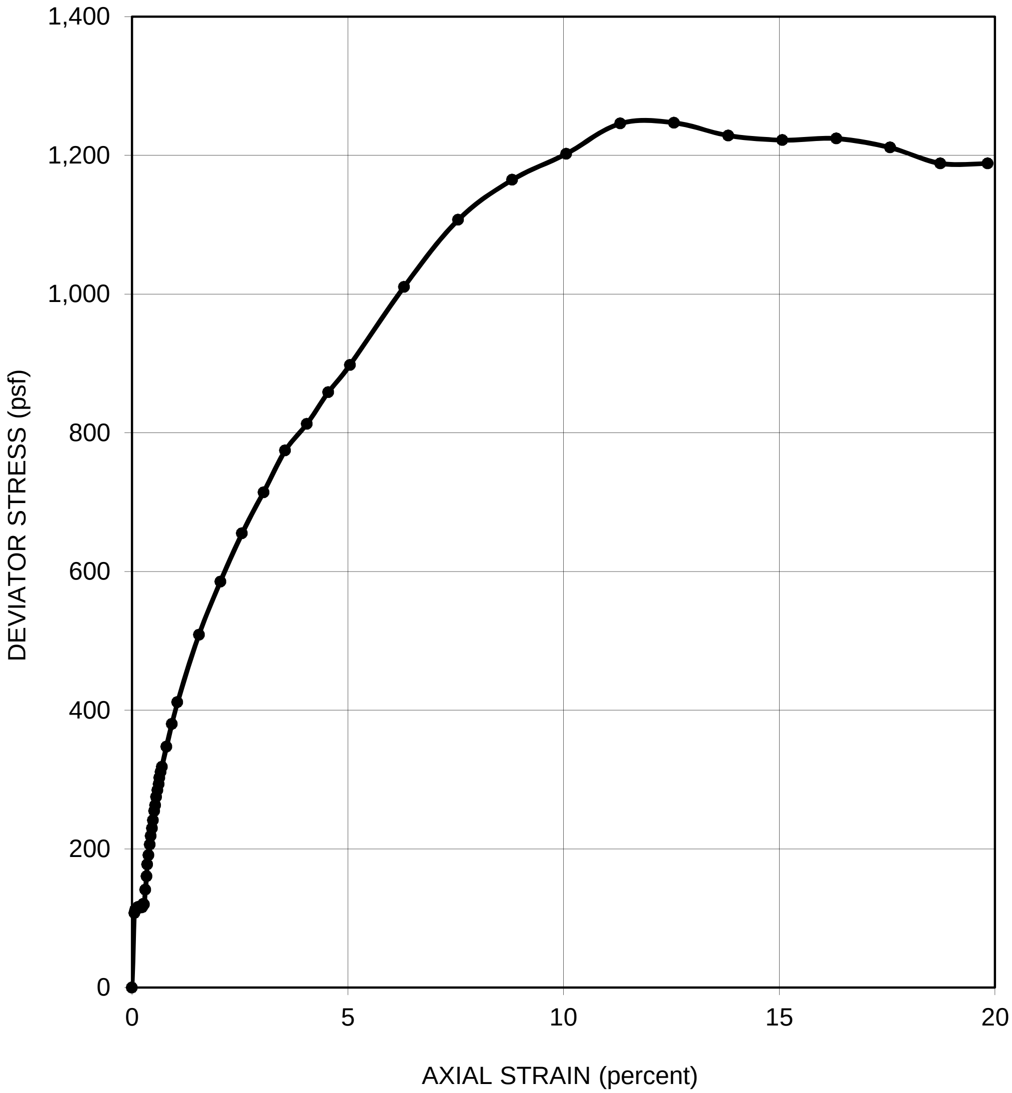
Date	11/11/11	Project No.	750603902	Figure	C-25



DRAFT

SAMPLER TYPE	Dames and Moore			SHEAR STRENGTH	620	psf
DIAMETER (in.)	2.43	HEIGHT (in.) 5.58		STRAIN AT FAILURE	12.6	%
MOISTURE CONTENT		46.3	%	CONFINING PRESSURE	2,250	psf
DRY DENSITY		72	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), gray				SOURCE	B29-7 at 31 feet

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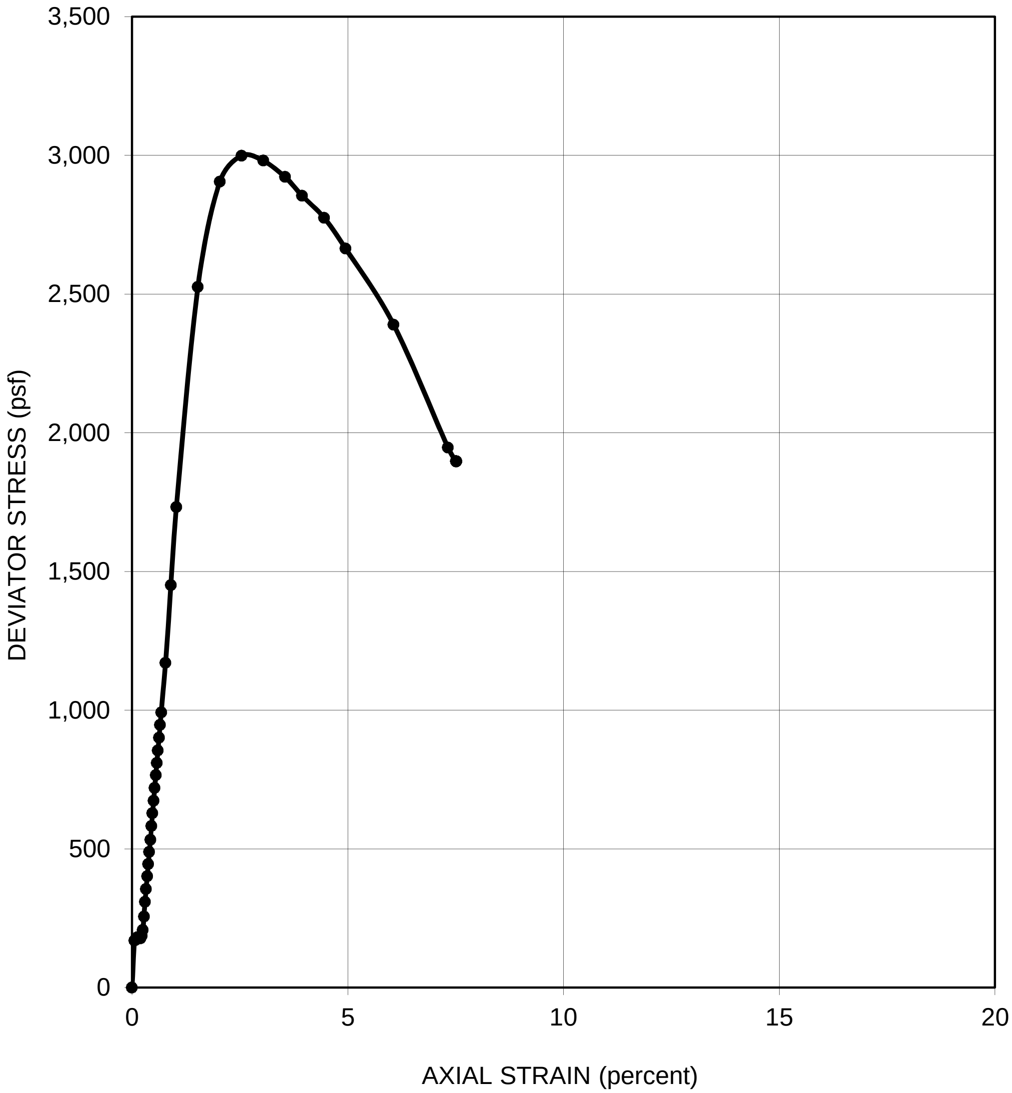
Date	11/11/11	Project No.	750603902	Figure	C-26



DRAFT

SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	1,500	psf
DIAMETER (in.)	2.43	HEIGHT (in.) 5.54		STRAIN AT FAILURE	2.5	%
MOISTURE CONTENT		55.6	%	CONFINING PRESSURE	2,850	psf
DRY DENSITY		65	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CH), gray				SOURCE	B29-7 at 42 feet

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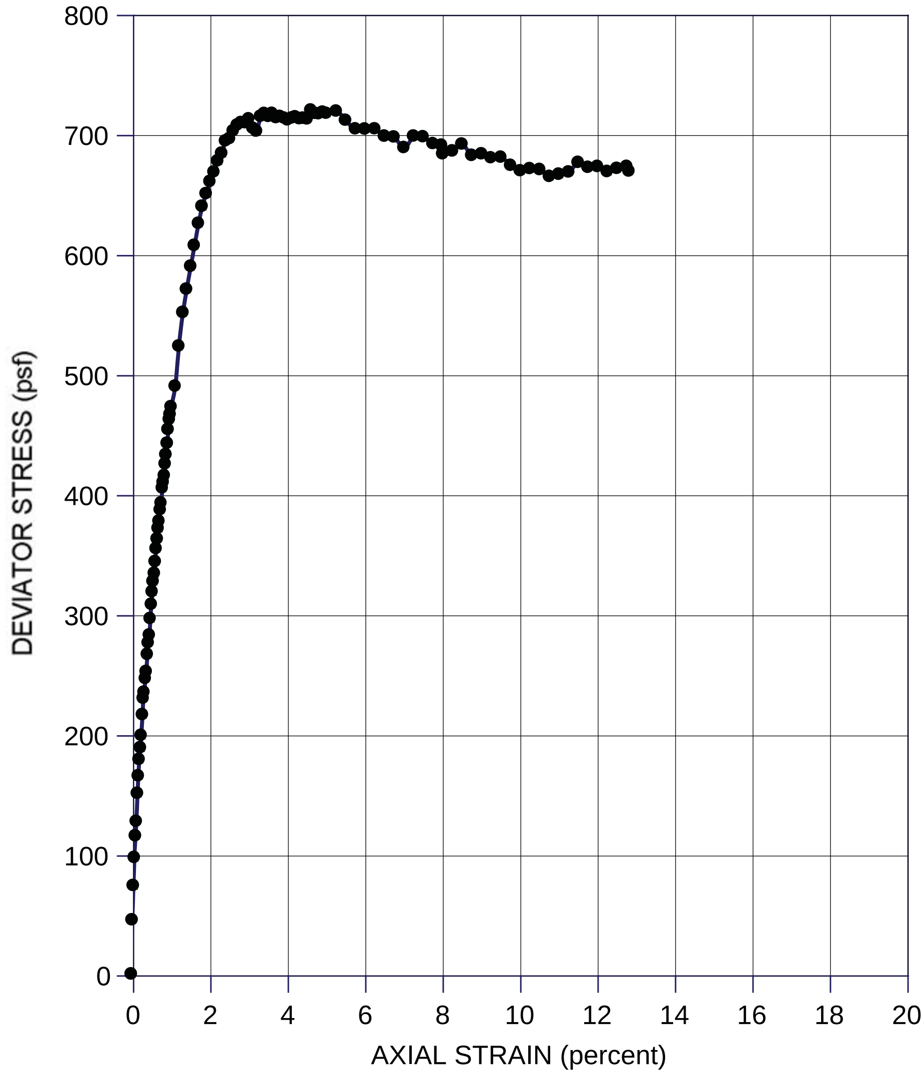
Date	11/11/11	Project No.	750603902	Figure	C-27



DRAFT

SAMPLER TYPE Shelby Tube (ST)		SHEAR STRENGTH	360 psf
DIAMETER (in) 2.86	HEIGHT (in) 6.02	STRAIN AT FAILURE	3.40 %
MOISTURE CONTENT	58.1 %	CONFINING PRESSURE	1,200 psf
DRY DENSITY	65 pcf	STRAIN RATE	1.0 % /min.
DESCRIPTION CLAY (CH), gray		SOURCE B30-1 at 28 feet	

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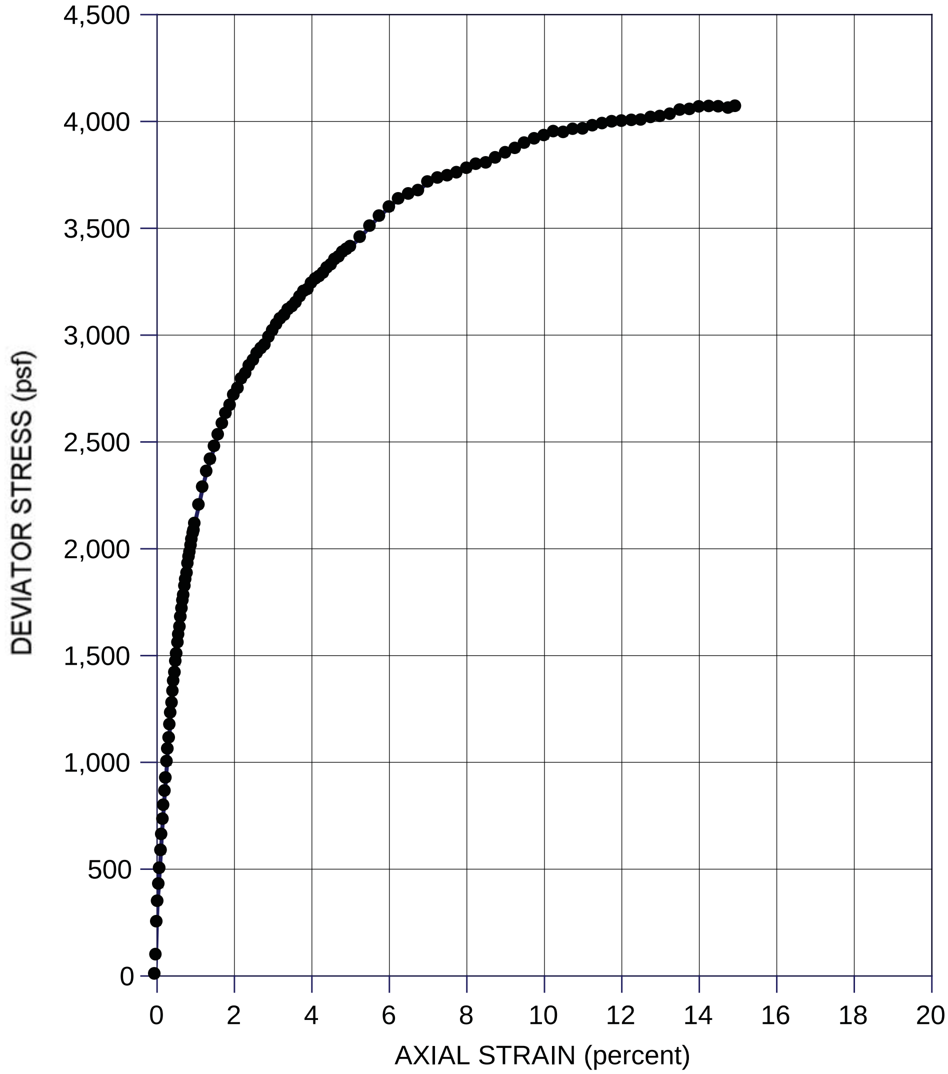
Date 11/18/11 | Project No. 750603902 | Figure C-28



DRAFT

SAMPLER TYPE Sprague & Henwood (S&H)		SHEAR STRENGTH	2,030 psf
DIAMETER (in) 2.43	HEIGHT (in) 5.00	STRAIN AT FAILURE	14.1 %
MOISTURE CONTENT	25.5 %	CONFINING PRESSURE	2,200 psf
DRY DENSITY	100 pcf	STRAIN RATE	1.0 % /min.
DESCRIPTION CLAY (CL), olive with orange-brown mottling		SOURCE B30-2 at 59.5 feet	

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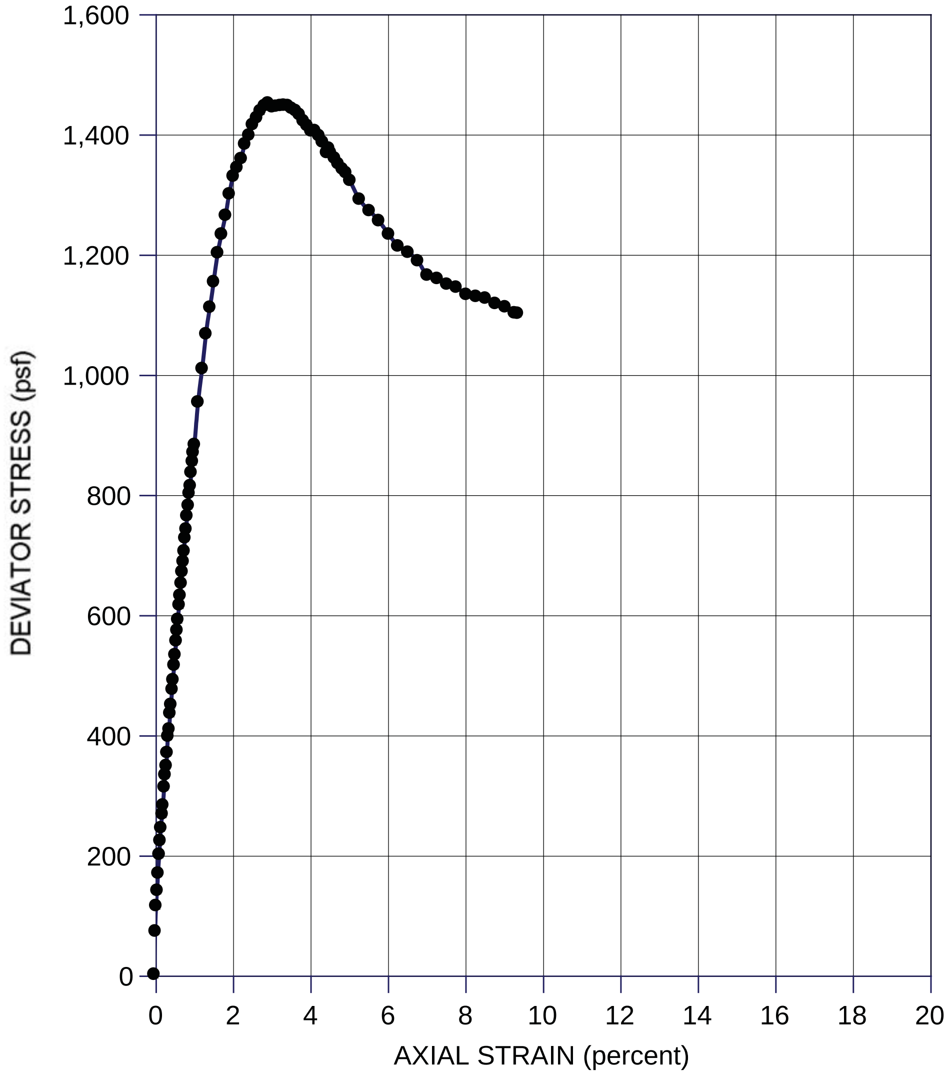
Date 11/18/11 | Project No. 750603902 | Figure C-29



DRAFT

SAMPLER TYPE Shelby Tube (ST)		SHEAR STRENGTH	725 psf
DIAMETER (in) 2.86	HEIGHT (in) 6.01	STRAIN AT FAILURE	2.8 %
MOISTURE CONTENT	63.8 %	CONFINING PRESSURE	1,500 psf
DRY DENSITY	62 pcf	STRAIN RATE	1.0 % /min.
DESCRIPTION CLAY (CH), gray		SOURCE B30-4 at 39 feet	

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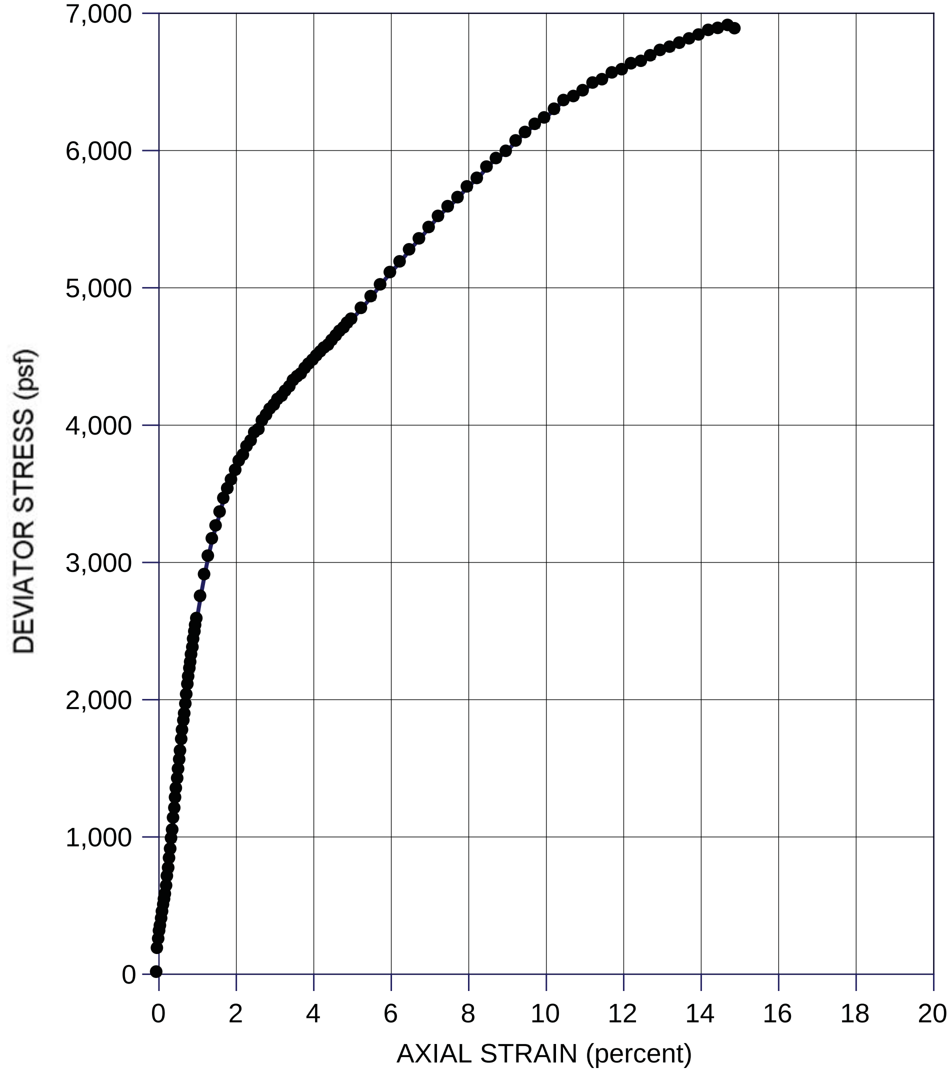
Date 11/18/11 | Project No. 750603902 | Figure C-30



DRAFT

SAMPLER TYPE	Sprague & Henwood (S&H)	SHEAR STRENGTH	3,450	psf
DIAMETER (in) 2.40	HEIGHT (in) 5.01	STRAIN AT FAILURE	14.7	%
MOISTURE CONTENT	22.3 %	CONFINING PRESSURE	1,700	psf
DRY DENSITY	105 pcf	STRAIN RATE	1.0	% /min.
DESCRIPTION	CLAY with SAND (CL), olive with orange-brown mottling	SOURCE B30-4 at 54 feet		

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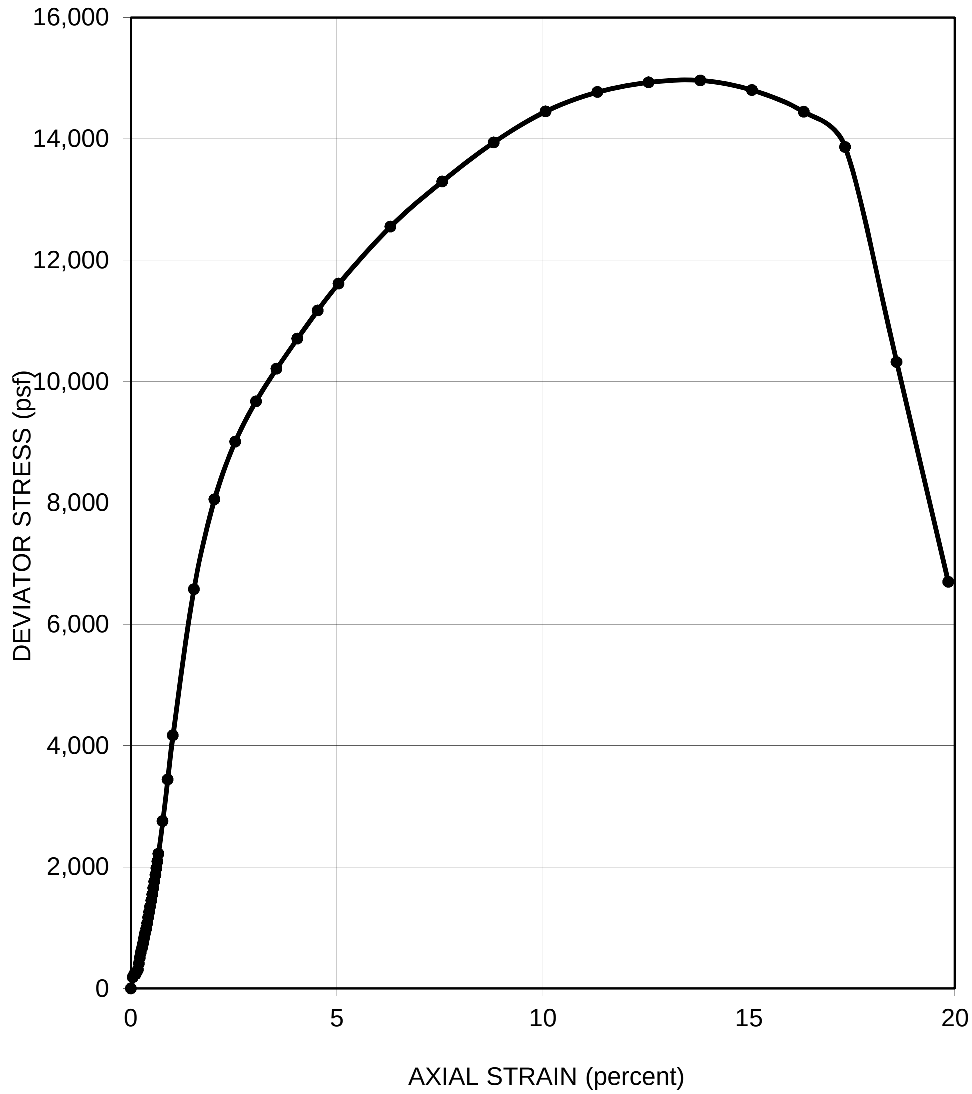
Date 11/18/11 | Project No. 750603902 | Figure C-31



DRAFT

SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	7,480	psf
DIAMETER (in.) 2.43	HEIGHT (in.) 6.00		STRAIN AT FAILURE	13.8	%
MOISTURE CONTENT	17.1	%	CONFINING PRESSURE	3,450	psf
DRY DENSITY	116	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown			SOURCE	B31-2 at 46 feet

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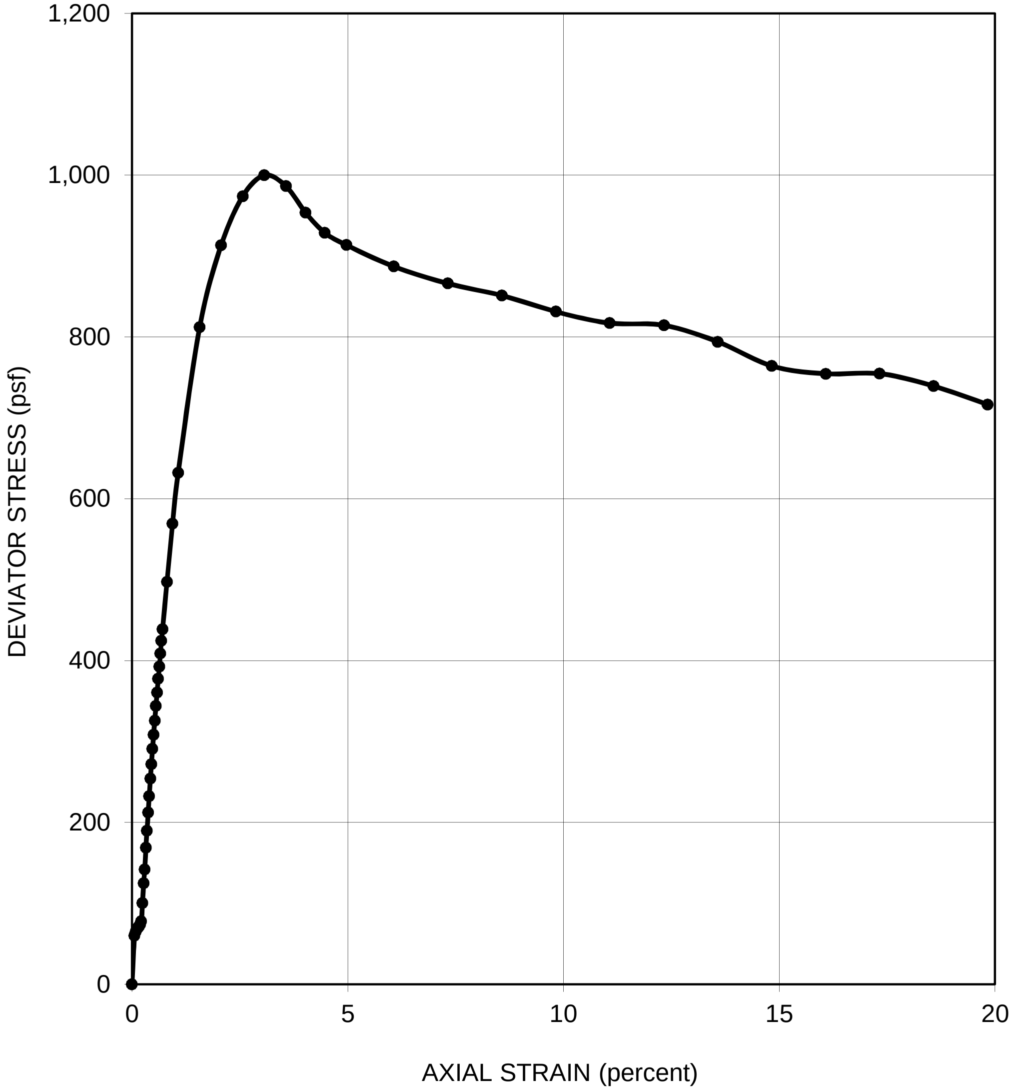
Date 11/11/11 | Project No. 750603902 | Figure C-32



DRAFT

SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	500	psf
DIAMETER (in.)	2.42	HEIGHT (in.) 5.16		STRAIN AT FAILURE	3.1	%
MOISTURE CONTENT		59.5	%	CONFINING PRESSURE	1,450	psf
DRY DENSITY		63	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CH), gray				SOURCE	B31-7 at 11.5 feet

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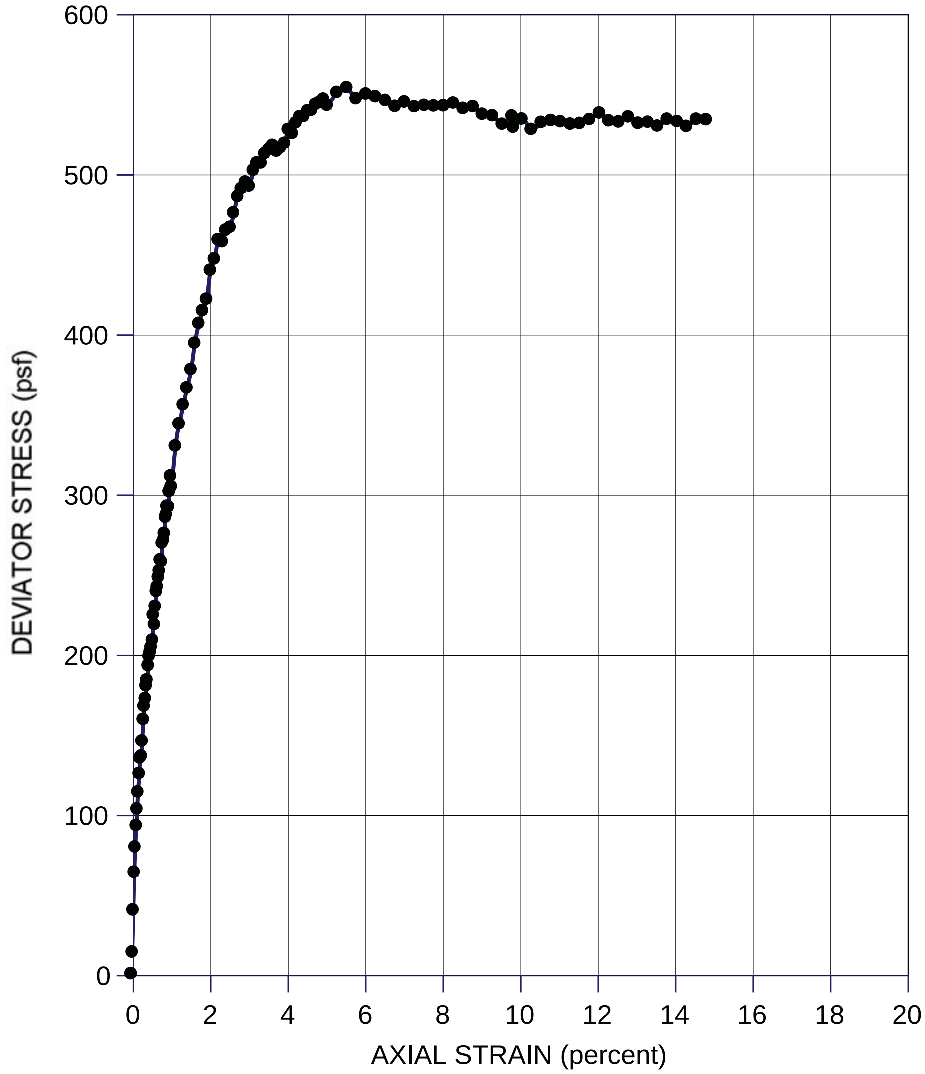
Date	11/11/11	Project No.	750603902	Figure	C-33



DRAFT

SAMPLER TYPE Shelby Tube (ST)		SHEAR STRENGTH	275 psf
DIAMETER (in) 2.87	HEIGHT (in) 6.00	STRAIN AT FAILURE	5.0 %
MOISTURE CONTENT	59.1 %	CONFINING PRESSURE	1,050 psf
DRY DENSITY	64 pcf	STRAIN RATE	1.0 % /min.
DESCRIPTION CLAY (CH), gray		SOURCE B32-1 at 16.5 feet	

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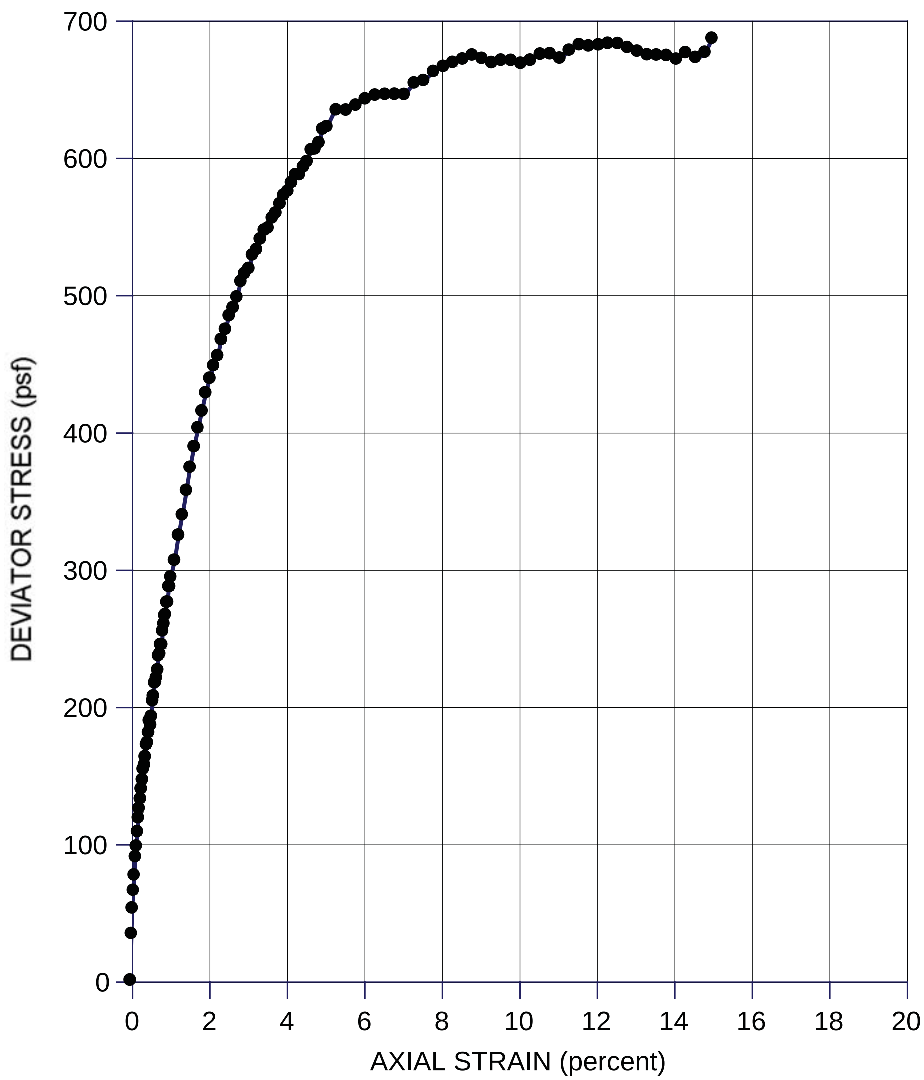
Date 11/18/11 | Project No. 750603902 | Figure C-34



DRAFT

SAMPLER TYPE Shelby Tube (ST)		SHEAR STRENGTH	340 psf
DIAMETER (in) 2.87	HEIGHT (in) 6.00	STRAIN AT FAILURE	15.0 %
MOISTURE CONTENT	57.2 %	CONFINING PRESSURE	850 psf
DRY DENSITY	65 pcf	STRAIN RATE	1.0 % /min.
DESCRIPTION CLAY (CH), gray		SOURCE B32-2 at 19 feet	

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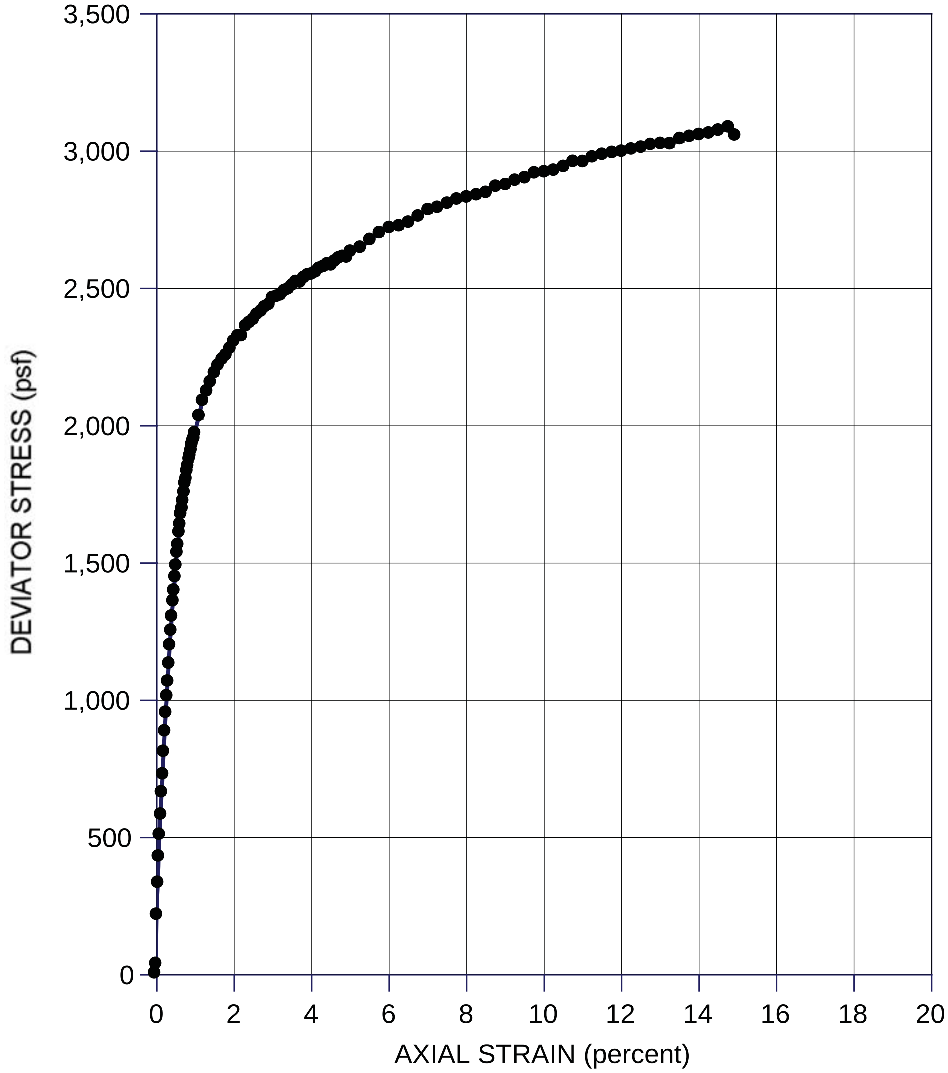
Date 11/18/11 | Project No. 750603902 | Figure C-35



DRAFT

SAMPLER TYPE Sprague & Henwood (S&H)		SHEAR STRENGTH	1,540 psf
DIAMETER (in) 2.43	HEIGHT (in) 4.99	STRAIN AT FAILURE	14.8 %
MOISTURE CONTENT	29.6 %	CONFINING PRESSURE	1,660 psf
DRY DENSITY	94 pcf	STRAIN RATE	1.0 % /min.
DESCRIPTION CLAY (CL), olive		SOURCE B32-2 at 45 feet	

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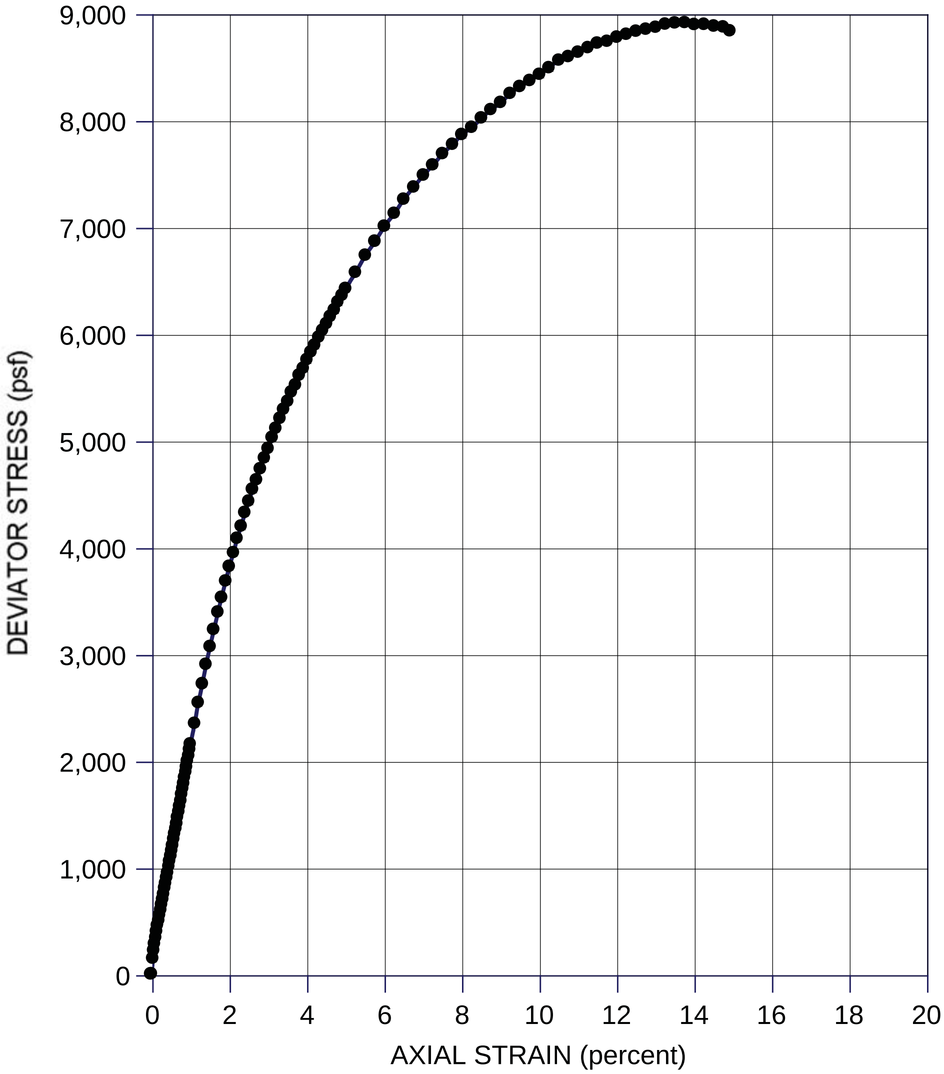
Date 11/18/11 | Project No. 750603902 | Figure C-36



DRAFT

SAMPLER TYPE Sprague & Henwood (S&H)		SHEAR STRENGTH	4,450 psf
DIAMETER (in) 2.44	HEIGHT (in) 5.05	STRAIN AT FAILURE	13.5 %
MOISTURE CONTENT	15.7 %	CONFINING PRESSURE	850 psf
DRY DENSITY	118 pcf	STRAIN RATE	0.99 % /min.
DESCRIPTION CLAY with SAND (CL), olive with red-brown mottling			SOURCE B32-5 at 24.5 feet

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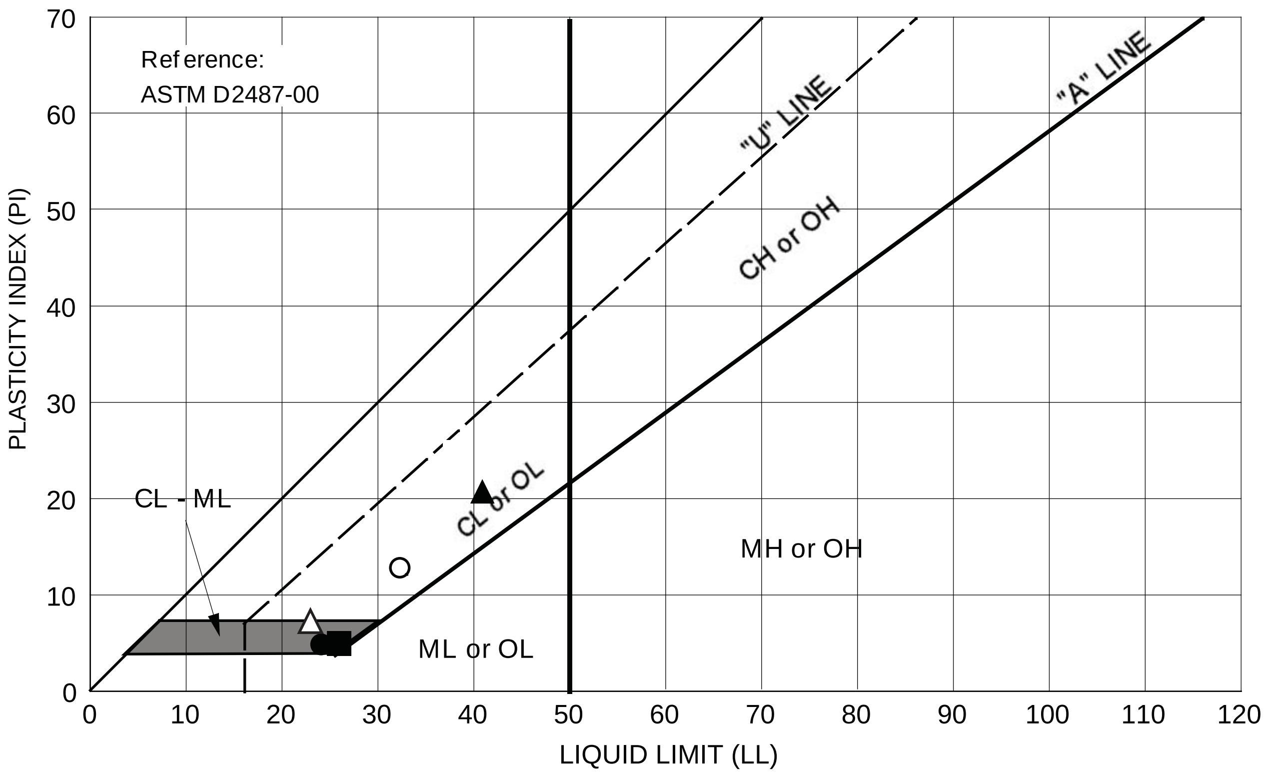
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 11/18/11 | Project No. 750603902 | Figure C-37



DRAFT

Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B29-2 at 5 feet	SILTY SAND with GRAVEL (SM), brown to olive-brown	17.4	24	5	23.6
▲	B29-6 at 10.5 ft.	SAND with CLAY and GRAVEL (SP-SC), gray-brown	17.4	41	21	11.1
■	B30-1 at 3 feet	SANDY SILTY CLAY with GRAVEL (CL-ML), olive-gray	--	26	5	--
○	B30-2 at 10 feet	CLAYEY SAND with GRAVEL (SC), green-gray	--	32	13	--
△	B30-5 at 11.5 ft.	SANDY SILTY CLAY (CL-ML), gray	--	23	7	--

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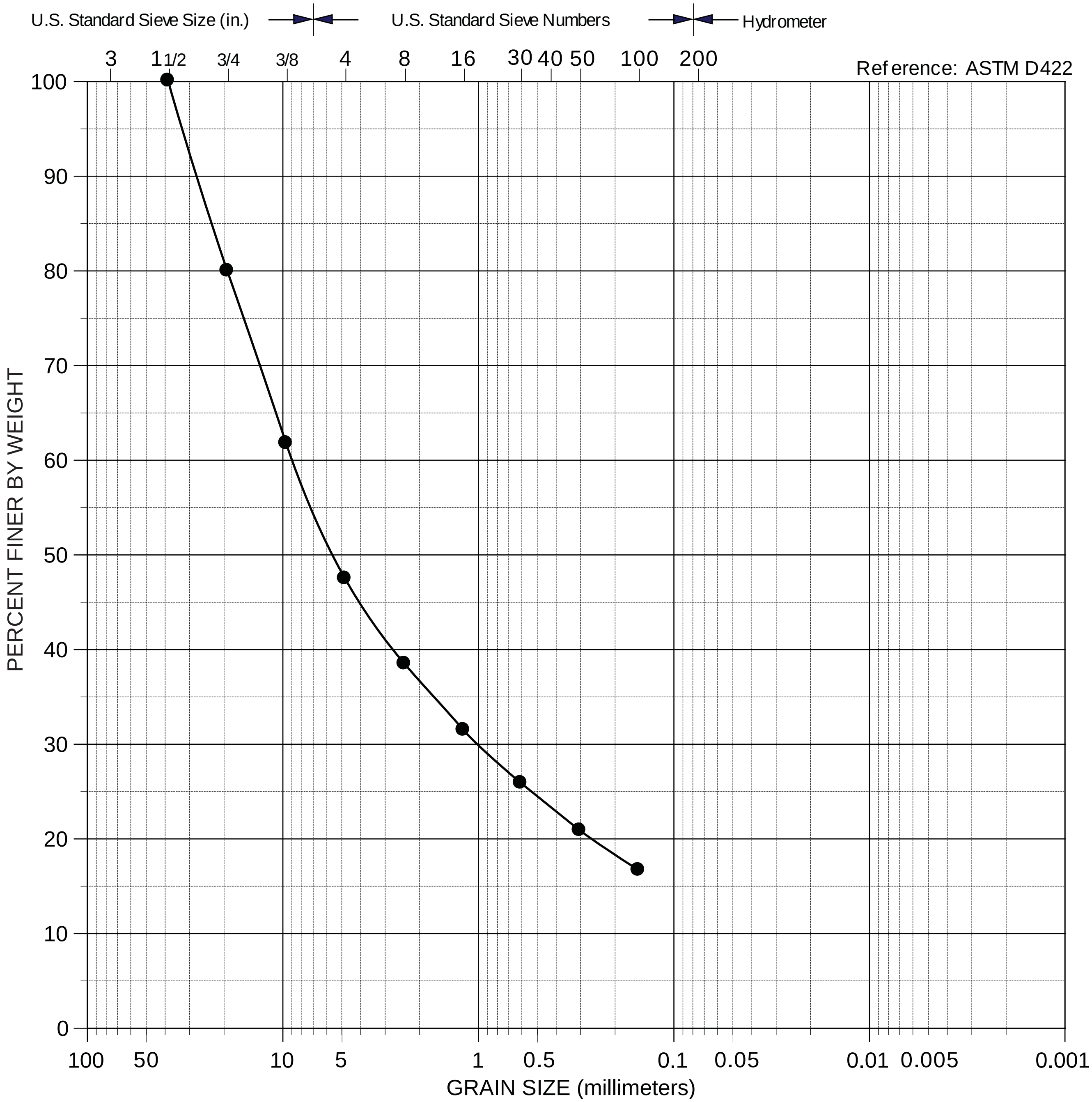
PLASTICITY CHART

Date 11/17/11 | Project No. 750603902 | Figure C-38



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Symbol	Sample Source	Classification
●	B30-3 at 16.5 feet	CLAYEY GRAVEL with SAND (GC), olive-gray

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PARTICLE SIZE ANLYSIS

Date 11/17/11	Project No. 750603902	Figure C-39

APPENDIX D

Soil Corrosivity Evaluation & Recommendations for Corrosion Control

December 12, 2011

Treadwell & Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111

Attention: **Ms. Kristen Lease**
Senior Staff Engineer

Subject: **Soil Corrosivity Evaluation & Recommendations for Corrosion Control**
Blocks 29-34, Mission Bay
San Francisco, CA

Dear Ms. Lease,

Pursuant to your request, **JDH Corrosion Consultants, Inc**. has conducted a site corrosivity evaluation for the above referenced project site and we have provided herein recommendations for long-term corrosion control for the proposed materials of construction for the underground utilities at this site.

Purpose

The purpose for this evaluation is to determine the corrosion potential, resulting from the soils at the subject site and to provide recommendations for long-term corrosion control for the pipelines, steel or concrete piles, concrete foundations and buried metallic utilities.

Background

The proposed development will consist of multiple structures between six-to nine-stories above grade with below grade service areas. We understand that the finished floor for the below grade areas of some structures will be approximately 25 feet below finished floor elevation. The structures will be pile supported (steel or concrete piles).

Soil Testing and Analysis

We conducted in-situ resistivities and supplemented this data with the chemical analysis of the soil samples from the Mission Bay Blocks 29-34 location.

Soil Testing Results

Soil samples were collected from the site by Treadwell & Rollo field personnel and they were transported to a state certified testing laboratory, **CERCO Analytical, Inc.** (certificate no. 2153) located in Concord, CA for chemical analysis. Six (6) samples from Block 29-32 were received on September 27, 2011 and six (6) samples from Block 33-34 were received on November 4, 2011. Each sample was analyzed for pH, chlorides, resistivity (@ 100% saturation), sulfates and Redox potential using ASTM test methods as detailed in the table below. The preparation of the soil samples for chemical analysis was in accordance with the applicable specifications.

Soil Analysis Test Methods

Chemical Analysis	ASTM Method
Chlorides	D4327
pH	D4972
Resistivity (100% Saturation)	G57
Sulfate	D4327
Redox Potential	D1498

The results of the chemical analysis are provided in the CERCO Analytical, Inc. reports dated October 5, 2011 and November 14, 2011. The results are summarized as follows:

CERCO Analytical, Inc.
Blocks 29 – 32 Fill Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	N.D. – 300 (mg/kg)	Non-corrosive to Moderately Corrosive *
pH	8.0 – 8.9	Non corrosive*
Resistivity	740 – 4,300 ohms-cm	Corrosive to Moderately Corrosive*
Sulfate	33 – 360 (mg/kg)	Non-corrosive to Mildly Corrosive**
Redox Potential	410 - 450 mV	Non-corrosive*

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

CERCO Analytical, Inc.
Blocks 29 – 32 Bay Mud Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	3,500 (mg/kg)	Severely Corrosive*
pH	8.6	Non corrosive*
Resistivity	120 ohms-cm	Severely Corrosive *
Sulfate	140 (mg/kg)	Non-corrosive**
Redox Potential	390 mV	Mildly Corrosive*

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

CERCO Analytical, Inc.
Blocks 33 – 34 Fill Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	N.D. – 24 (mg/kg)	Non-corrosive*
pH	8.0 – 8.9	Non corrosive*
Resistivity	2,200 – 6,500 ohms-cm	Moderately Corrosive*
Sulfate	N.D. – 110 (mg/kg)	Non-corrosive**
Redox Potential	410 - 430 mV	Non-corrosive*

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

CERCO Analytical, Inc.
Blocks 33 – 34 Bay Mud Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	3,100 (mg/kg)	Severely Corrosive*
pH	8.4	Non corrosive*
Resistivity	170 ohms-cm	Severely Corrosive *
Sulfate	78 (mg/kg)	Non-corrosive**
Redox Potential	240 mV	Mildly Corrosive*

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Chemical Testing Analysis

The chemical analysis provided by **CERCO Analytical, Inc.** indicates that based on this soil data, the fill soils are generally classified as corrosive and the Bay Mud is generally classified as "severely corrosive" based on the resistivity measurements. The chloride levels indicate "non-corrosive to severely corrosive" conditions to steel and ductile iron, and the sulfate levels indicate "non-corrosive to mildly corrosive" conditions for concrete structures placed into these soils with regard to sulfate attack. The pH of the soils ranges from slightly acidic to alkaline which classifies them as "non-corrosive" to buried steel and concrete structures.

In-Situ Soil Resistivity Measurements

The in-situ resistivity of the soil was measured at two (2) locations at the project site by ***JDH Corrosion Consultants, Inc.*** field personnel. Resistance measurements were conducted with probe spacing of 2.5, 5, 7.5, 10 and 15-feet at each location. For analysis purposes we have calculated the resistivity of soil layers 0-2.5, 2.5-5, 5-10 and 10-15' using the Barnes Method as follows:

$$\rho_{b-a} = KR_{(b-a)}$$

Where;

ρ_{b-a}	=	soil resistivity of layer depth b-a (ohm-cm)
a	=	soil depth to top layer (ft)
b	=	soil depth to bottom layer (ft)
R_a	=	soil resistance read at depth a (ohms)
R_b	=	soil resistance read at depth b (ohms)
R_{b-a}	=	resistance of soil layer from a to b (ft)
K	=	layer constant = $60.96\pi(b-a)$ (cm)

and $$\frac{1}{R_{b-a}} = \frac{1}{R_a} - \frac{1}{R_b}$$

The visual diagrams below describe the Wenner 4-pin testing configuration.

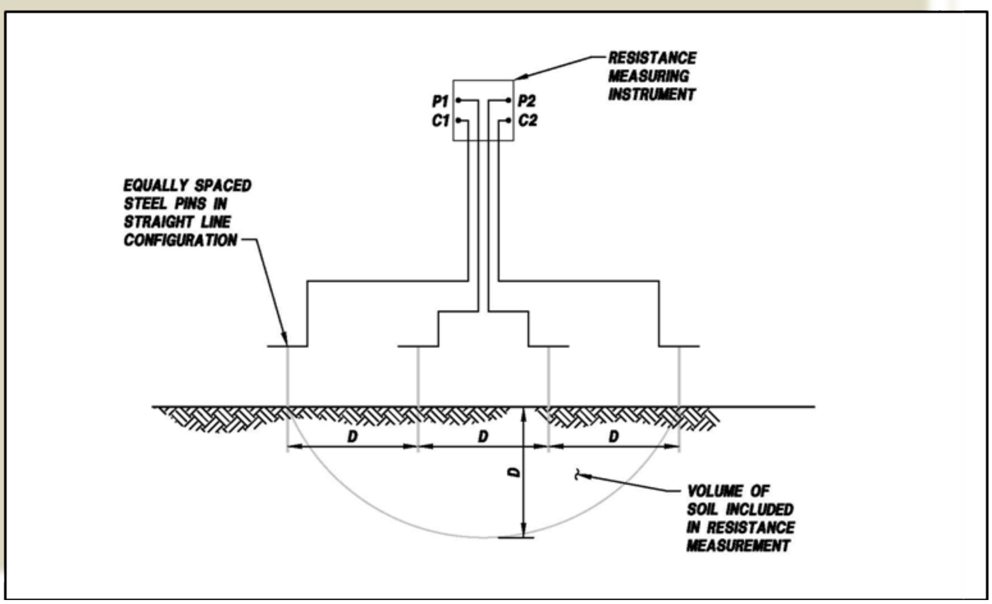


Fig 1: Wenner 4-Pin Resistivity Schematic No.1

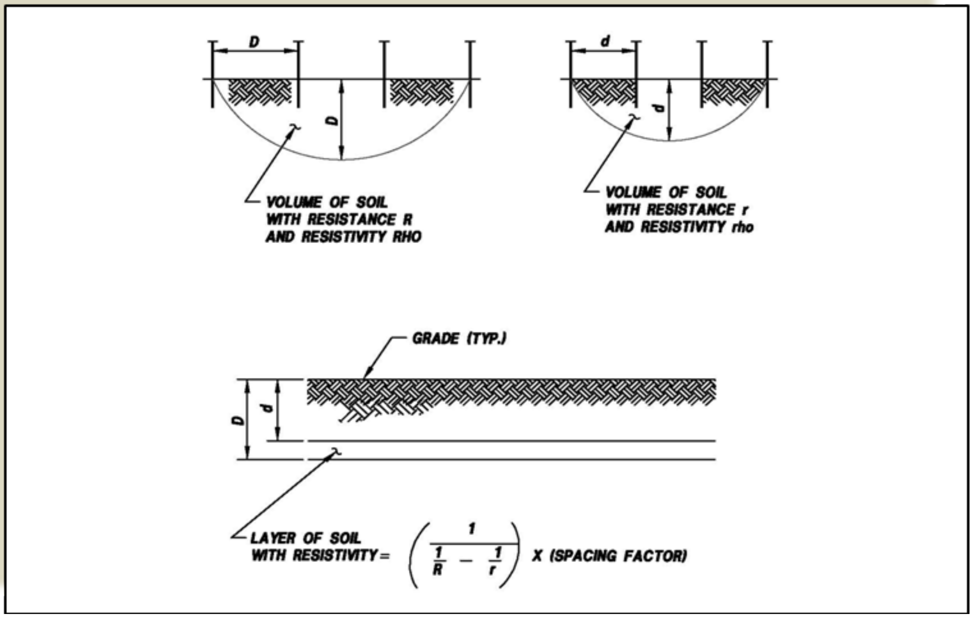


Fig 2: Illustration of Barnes Layer Calculations

In-Situ Soil Resistivity Analysis

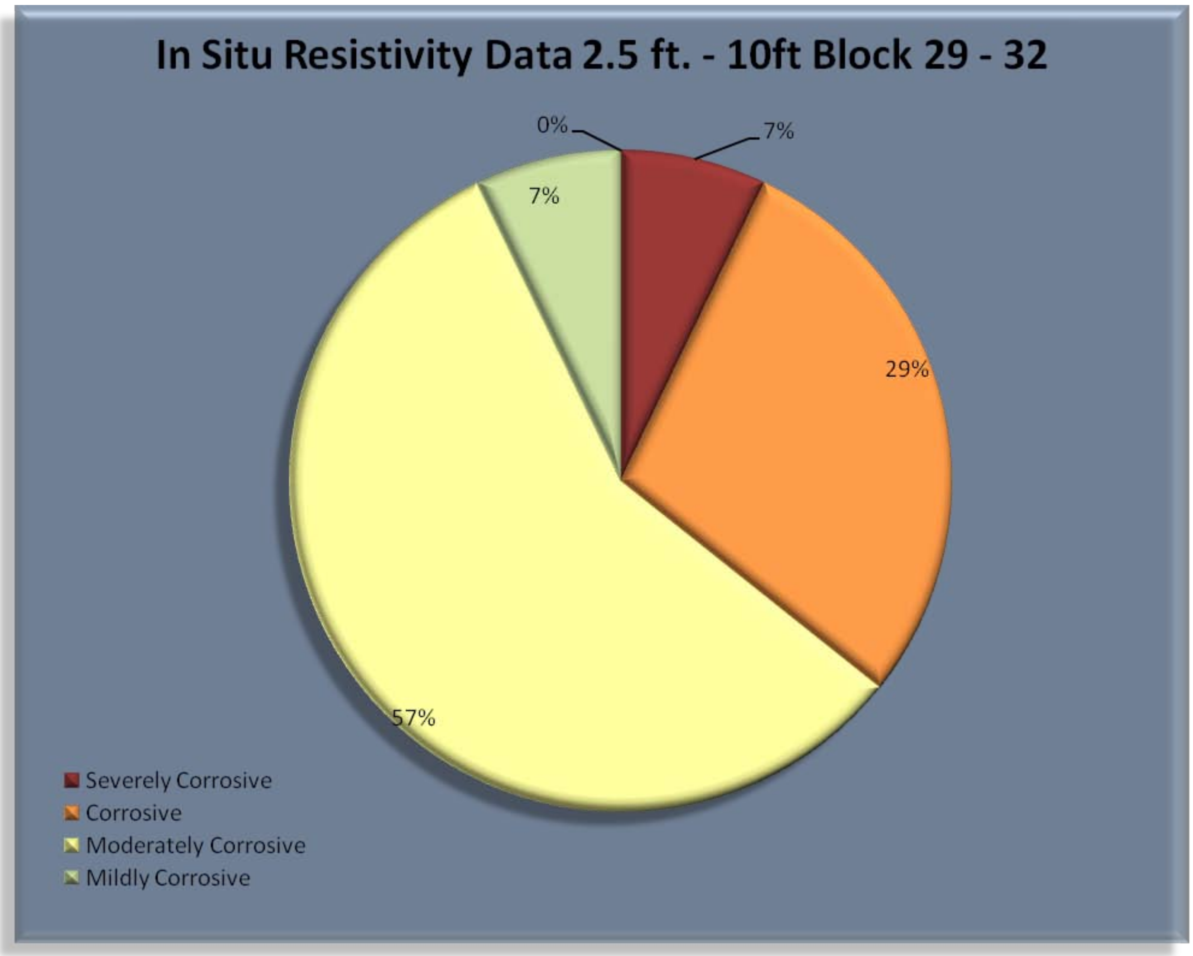
Corrosion of a metal is an electro-chemical process and is accompanied by the flow of electric current. Resistivity is a measure of the ability of a soil to conduct an electric current and is, therefore, an important parameter in consideration of corrosion data. Soil resistivity is primarily dependent upon the chemical content and moisture content of the soil mass.

The greater the amount of chemical constituents present in the soil, the lower the resistivity will be. As moisture content increases, resistivity decreases until maximum solubility of dissolved chemicals is attained. Beyond this point, an increase in moisture content results in dilution of the chemical concentration and resistivity increases. The corrosion rate of steel in soil normally increases as resistivity decreases. Therefore, in any particular group of soils, maximum corrosion will generally occur in the lowest resistivity areas. The following classification of soil corrosivity, developed by William J. Ellis[1], is used for the analysis of the soil data for the project site.

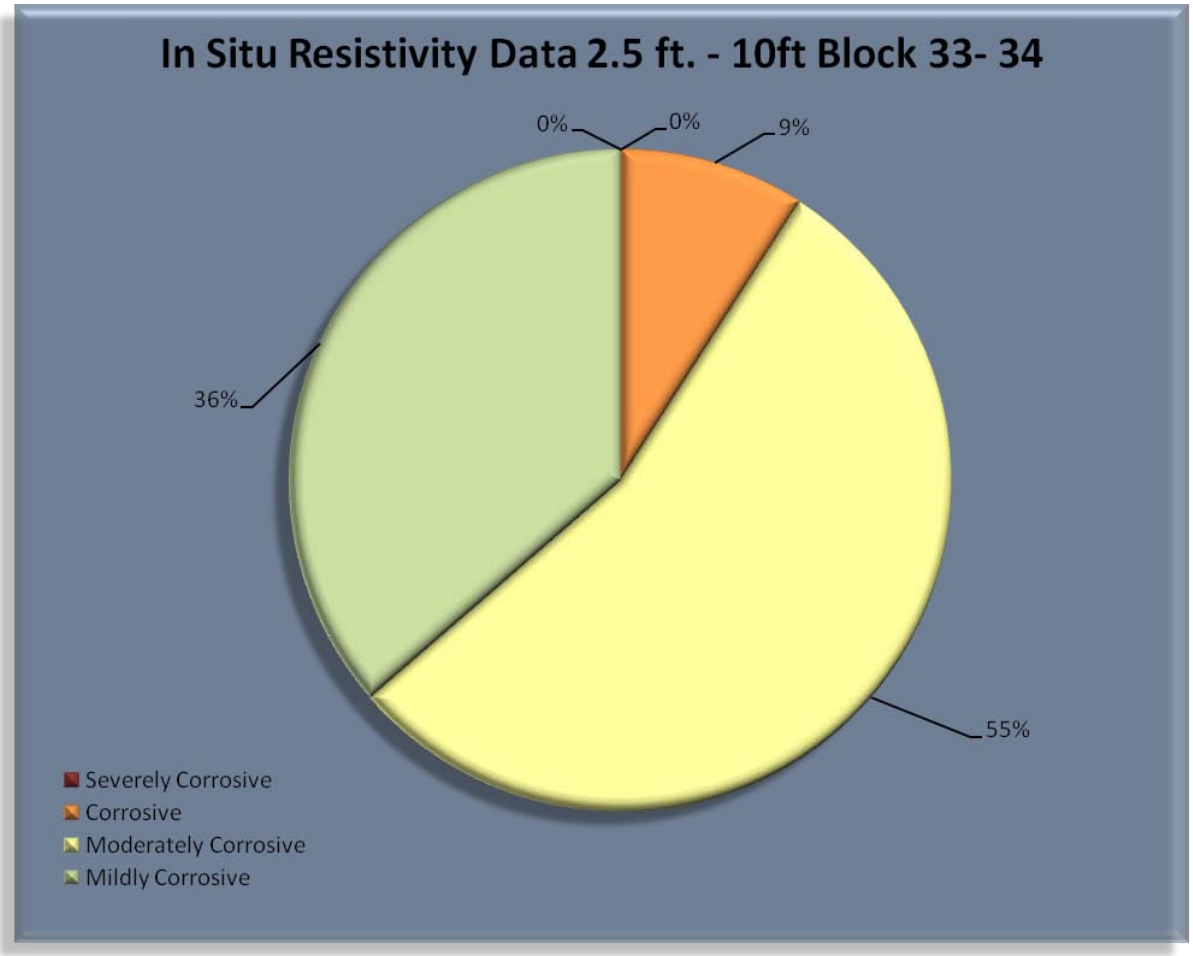
Resistivity (Ohm-cm)	Corrosivity Classification
0 – 500	Very Corrosive
501 – 2,000	Corrosive
2,001 – 8,000	Moderately Corrosive
8,001 – 32,000	Mildly Corrosive
> 32,000	Progressively Less Corrosive

The above classifications are appropriate for the project site and the results are presented in the graphs below. In general, the soils are classified as "corrosive" with respect to corrosion of buried cast/ductile iron and steel structures throughout the top 2.5 to 10 feet of the site.

The chart of the in-situ soil resistivity data for the soil layers 2.5 to 10 feet (i.e. fill soils) of Block 29-32 indicate that 7% of the soils are classified as "severely corrosive", 29% of the soils are classified as "corrosive", 57% of the soils are classified as "moderately corrosive" and 7% of the soils are classified as mildly corrosive.



The chart of the in-situ soil resistivity data for the soil layers 2.5 to 10 feet (i.e. fill soils) of Block 33-34 indicate that 9% of the soils are classified as "corrosive" and 55% of the soils are classified as "moderately corrosive" and 36% of the soils are classified as "mildly corrosive".



Discussion

Sub-grade Reinforced Concrete Walls and Floors

The presence of water-soluble sulfate ions in the soils tested in the fill zone of the soil at the site was at a relatively low level. As such, Type II cement can be utilized for the concrete foundations. However the soils are corrosive and the chloride levels are moderately high. In order to slow the ingress of aggressive ions, it is recommended that the water/cement ratio should not exceed 0.40 in order to achieve a dense concrete, with a minimum depth of cover of 3" over the reinforcing bars, especially in the areas where the foundation is more than a few feet deep.

Piles

Pre-stressed Pre-cast Reinforced Concrete Piles

The pre-stressed, pre-cast concrete piles will pass through the aggressive Bay Mud. It is therefore recommended that Type II cement should be utilized. The water/cement ratio should not exceed 0.35 in order to achieve a dense concrete, with a minimum depth of cover of 2" over the pre-stressing wires. Also, a mineral admixture shall be added to the concrete mix.

Bare Steel Piles

Due to the corrosive soils being encountered, the piles are expected to experience corrosion, especially in the top 30 feet. It is therefore recommended to use a corrosion allowance on all exposed surfaces of the piles. In addition use of coatings and cathodic protection may be required, depending upon the specific design of the structure.

Underground Metallic Pipelines

The fill soils at the project site are considered to be "corrosive" to ductile/cast iron, steel and dielectric coated steel. Therefore, we recommend the use of coatings, and/or polyethylene encasement, supplemented with cathodic protection for direct buried metallic pressure piping such as domestic and fire water pipelines. All underground pipelines should also be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to minimize potential galvanic corrosion problems.

Recommendations

Sub-grade Reinforced Concrete Walls and Floors

For application in reinforced concrete slab foundations, we recommend using a Type II modified cement mix with a maximum water-to-cement ratio of 0.40 and a minimum depth of cover for the reinforcing steel of 3-inches. Also, a mineral admixture shall be added to the concrete mix. The amount of mineral admixture shall be 25% of the total amount of the cementitious material used in the concrete mix and shall be comprised of 80% by mass mineral admixture conforming to ASTM Designation: C618 type F or N and 20% by mass mineral admixture meeting ASTM Designation: C 1240.

Also, a calcium nitrite corrosion inhibitor shall be added to the concrete mix. The amount of inhibitor added to the concrete mix will be determined by whether a vapor guard is installed between the soil and concrete. 4 gallons per cubic yard of calcium nitrite inhibitor shall be added to the concrete mix, if the vapor guard is **not** installed. If the vapor guard is installed, 2 gallons per cubic yard shall be added.

Piles

Pre-stressed Pre-cast Reinforced Concrete Piles

It is recommended that Type II cement should be utilized. The water/cement ratio should not exceed 0.35 in order to achieve a dense concrete, with a minimum depth of cover of 2" over the pre-stressing wires. Also, a mineral admixture shall be added to the concrete mix. The amount of mineral admixture shall be 25% of the total amount of the cementitious material used in the concrete mix and shall be comprised of 80% by mass mineral admixture conforming to ASTM Designation: C618 type F or N and 20% by mass mineral admixture meeting ASTM Designation: C 1240.

Bare Steel Piles

It is recommended to use a corrosion allowance on all exposed surfaces of the piles for the top 30 feet of the piles at a minimum. The exact length of the pile requiring the corrosion allowance will vary depending upon the design of the structure and the specific soils conditions for the subject piles. The amount of corrosion allowance (i.e. thickness) to be added to the piles is dependent upon the type of pile being used and the desired design life for the subject piles as provided in the following table:

Total Added Thickness for Corrosion Allowance

Pile Type	50-yr Design Life	75-yr Design Life	100-yr Design Life
Pipe Type Pile	(1/16") .0625-in.	(3/32") .09375-in.	(1/8") .125-in.
H-piles	(1/8") .125-in.	(3/16") .1875-in.	(1/4") .25-in.

A dielectric barrier such as a 10-mil thick polyethylene sheet should also be installed between the pile cap or reinforced concrete foundation and the soil underneath to minimize the effects of the galvanic cell between steel in soil and steel in concrete. In addition the possible use of coatings and cathodic protection should be considered, depending upon the specific design of the steel supports.

Ductile Iron Pipe (Pressure Piping such as Domestic Water and Fire)

1. Direct buried ductile iron pipe should be encased in 8-mil polyethylene as specified in AWWA specification C-105. Epoxy coatings are also an acceptable alternative type of coating system for the pipe and/or fittings such as valves.

2. All rubber gasket joints, fusion-bonded epoxy coated flanges and flexible couplings on ductile iron pipelines should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

3. Insulating flanges and/or couplings should be installed to electrically isolate the buried portion of pipeline from other metallic pipelines, reinforced concrete structures and above grade buildings or structures.

4. Test stations shall be installed on all ductile iron pipelines at a spacing of 800 to 1,000 feet. Bonding and test stations shall comply with NACE Standards.

5. A sacrificial type of cathodic protection utilizing ***high-potential magnesium*** anodes should be installed to protect the entire length of buried metallic pipeline. Cathodic protection should be designed in accordance with NACE Standard SP0169-07 and applicable local standards and included with the contract documents to permit installation along with the pipeline.

6. As an alternate, non-metallic piping may be used in lieu of ductile iron piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures. However, all metallic valves, fittings and appurtenances on non-metallic piping will require protection as specified below.

Ductile Iron Fittings & Metallic Valves (On Plastic Pressure Piping)

1. All direct buried ductile iron fittings installed on non-metallic piping shall be provided with a bituminous coating from the factory and encased in an 8-mil polyethylene bag in the field in accordance with AWWA Specification C-105. All bolts, restraining rods, etc. shall be coated with bitumastic prior to encasement in the polyethylene bag.

2. All metallic valves shall be coated from the factory (i.e. using powdered epoxy or equivalent type of coating system) and all bolts shall be coated with bitumastic in the field and the entire valve shall be encased in an 8-mil polyethylene bag in accordance with AWWA Specification C-105.

3. A sacrificial type of cathodic protection utilizing ***high-potential magnesium*** anodes should be installed to protect the valves and fittings. Cathodic protection should be designed in accordance with NACE Standard SP0169-07 and applicable local standards and included with the contract documents to permit installation along with the pipeline.

Cast Iron (Gravity Sewer and Storm Drain Lines)

1. Sewer and storm drain lines that will be routed underneath a concrete foundation should be encased in 8-mil polyethylene as specified in AWWA specification C-105. Any lines outside the footprint of the building do not require any special corrosion control measures.

Steel Pipelines (Natural Gas Pipelines & Risers)

1. A fusion-bonded epoxy coating system or a suitable tape coating should be applied to all buried steel pipelines in accordance with ANSI/AWWA C214-95, "AWWA Standard for Tape Coating Systems for the Exterior of Steel Water Pipelines." Also, a tape coating per AWWA Standard C209-95 is recommended for special sections, connections and fittings.

2. Insulating flanges and/or couplings should be installed to electrically isolate the buried portions of steel pipelines from other metallic pipelines, reinforced concrete structures and above grade structures.

3. All rubber gasket joints, fusion epoxy coated flanges and flexible couplings should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

4. A sacrificial type of cathodic protection using ***high-potential magnesium*** anodes should be installed to protect the buried portions of steel pipelines used for the natural gas piping systems. Cathodic protection should be designed in accordance with NACE Standard SP0169-07 and applicable local standards and included with the contract documents to permit installation along with the subject pipeline.

5. As an alternate, non-metallic piping may be used in lieu of steel piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures.

Copper Water Pipelines (Service Lines)

1. All copper water laterals shall be provided with a polyethylene sleeve to effectively isolate the copper piping from the earth.

2. All copper water laterals shall be electrically isolated from metallic water mains via the use of insulating type corporation stops installed at the water main.

LIMITATIONS

The conclusions and recommendations contained in this report reflect the opinion of the author of this report and are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees either expressed or implied are provided.

We thank you for the opportunity to be of assistance on this important project. If you have any questions concerning this report or the recommendations provided herein, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

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CORROSION
STATE OF CALIFORNIA

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Brendon Hurley

Brendon Hurley
JDH Corrosion Consultants, Inc.
Field Technician

CC: File 11109

REFERENCES

1. *Ellis, William J., Corrosion of Concrete Pipelines, Western States Corrosion Seminar, 1978*

2. *AWWA Manual of Water Supply Practices - M27, First Edition, External Corrosion - Introduction to Chemistry and Control (Denver, CO: 1987)*

3. *National association of Corrosion Engineers, Standard Recommended Practice, SP 01-69-07, Control of External Corrosion on underground or Submerged Pipeline*

California State Certified Laboratory No. 2153

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Client: JDH Corrosion Consultants, Inc.
Client's Project No.: 11109 (TR 750603902)
Client's Project Name: Blocks 29-32, Mission Bay
Date Sampled: Not Indicated
Date Received: 27-Sep-11
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 5-Oct-2011

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)*
1109187-001	B29-3, #3 @ 6'	450	8.0	-	1,400	-	49	200
1109187-002	B29-4, #2A @ 6'	440	8.2	-	1,400	-	38	200
1109187-003	B29-6, #3A @ 6'	450	8.3	-	4,300	-	N.D.	33
1109187-004	B31-4, #4 @ 5'	410	8.9	-	740	-'	300	360
1109187-005	B31-7, #3 @ 3.5'	420	8.4	-	1,900	-	37	200
1109187-006	B31-7, #4 @ 10'	390	8.6	-	120	-	3,500(1)	140

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	4-Oct-2011	4-Oct-2011	-	29-Sep-2011	-	3-Oct-2011 & 4-Oct-2011	3-Oct-2011

* Results Reported on "As Received" Basis

N.D. - None Detected

(1) Detection limit is elevated to 150 mg/kg due to dilution

Cheryl McMillen
Laboratory Director

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

California State Certified Laboratory No. 2153

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Client: JDH Corrosion Consultants, Inc.
Client's Project No.: 11109
Client's Project Name: Blocks 33-34 TR#750603902
Date Sampled: Not Indicated
Date Received: 4-Nov-11
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 14-Nov-2011

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)*
1111038-001	B34-5 #4 @ 9'	410	8.0	-	6,500	-	24	N.D.
1111038-002	B33-5 #4 @ 10-11'	420	8.0	-	3,800	-	16	30
1111038-003	B34-4 #3 @ 4.5-5.4'	420	8.6	-	2,400	-	20	71
1111038-004	B33-2 #1 @ 2-3'	430	8.9	-	2,200	-	N.D.	110
1111038-005	B34-7 #2 @ 4-4.5'	430	8.5	-	3,800	-	N.D.	69
1111038-006	B34-5 #8 @ 25' (Bay Mud)	240	8.4	-	170	-	3,100	78

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	9-Nov-2011	10-Nov-2011	-	8-Nov-2011	-	11-Nov-2011 & 10-Nov-2011	10-Nov-2011

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

Client:	*Treadwell & Rollo*
Project:	*Mission Bay Blocks 29-34*
Location:	*San Francisco, CA*
Date:	*8/30/2011*
Subject:	*In-Situ Soil Resistivity Data*

Legend	
Severely Corrosive	**Mildly Corrosive**
Corrosive	**Progressively Less Corrosive**
Moderately Corrosive	

*Test	Location	Resistance Data From AEMC Meter					Soil Resistivities (ohm-cm)					Barnes Layer Analysis (ohm-cm)				
#	Description	2.5	5	7.5	10	15	2.5	5	7.5	10	15	0-2.5'	2.5-5'	5-7.5'	7.5-10"	10-15'
1	Position 1 (29-32)	5.04	2.61	1.50	1.17	1.00	2413	2499	2154	2241	2873	2413	2592	1689	2546	6590
2	Position 2 (29-32)	8.40	4.02	1.83	1.20	0.85	4022	3849	2628	2298	2442	4022	3691	1608	1669	2790
3	Position 3 (29-32)	10.43	5.01	2.53	1.81	1.40	4993	4797	3634	3466	4022	4993	4616	2447	3045	5918
4	Position 4 (29-32)	12.25	6.96	2.93	1.68	0.95	5865	6664	4208	3217	2729	5865	7716	2423	1885	2093
5	Position 5 (29-32)	12.23	2.61	0.62	0.32	0.15	5855	2499	890	613	431	5855	1589	389	317	270
6	Position 6 (29-32)	9.14	2.19	0.85	0.59	0.46	4376	2097	1221	1130	1321	4376	1379	665	923	1999
7	Position 7 (29-32)	25.3	12.05	4.12	2.48	0.87	12112	11538	5917	4749	2499	12112	11015	2997	2983	1283
8	Position 8 (33-34)	9.84	3.15	2.23	1.02	1.82	4711	3016	3203	1953	5228	4711	2218	3655	900	NA
9	Position 9 (33-34)	14.64	5.6	11.6	3.1	1.8	7009	5362	16661	5937	5171	7009	4342	NA	2025	4110
10	Position 10 (33-34)	23.7	11.81	7.31	5.17	2.05	11346	11308	10499	9901	5889	11346	11270	9185	8455	3253

APPENDIX E

Development of Site-Specific Response Spectra

APPENDIX E
DEVELOPMENT OF SITE-SPECIFIC RESPONSE SPECTRA

This appendix presents the results of our site-specific earthquake studies and our estimation of the level of ground shaking at the site during future earthquakes. We performed a Probabilistic Seismic Hazard Analysis (PSHA) and deterministic analysis to develop site-specific horizontal response spectra for the levels of shaking corresponding to the Maximum Considered Earthquake (MCE) and Design Earthquake (DE) per the 2010 San Francisco Building Code (SFBC) and ASCE 7-05. The MCE spectrum is defined as the lesser of the probabilistic spectrum having 2 percent probability of exceedance in 50 years or 150 percent of median deterministic event on the governing fault. The DE is defined as 2/3 of the MCE spectrum.

To develop site-specific design response spectra for Blocks 29, 30 and 31, we:

- performed PSHA to develop uniform hazard response spectrum for rock for a hazard level corresponding to a 2 percent probability of exceedance in 50 years (2,475-year return period);
- performed a deterministic analysis to develop response spectra for rock corresponding to 150 percent of the median deterministic analysis;
- developed the MCE rock spectrum as the lesser of the probabilistic and deterministic spectra described above;
- developed the Design Earthquake (DE) level of shaking response spectrum, consistent with the definition of 2010 SFBC which is 2/3 of the MCE spectrum;
- performed spectral matching of five recorded time-histories to the DE rock spectrum. The DE time histories were scaled by 1.5 to develop MCE time histories. Both the DE and MCE time histories were used as input motions in ground response analyses;
- performed equivalent linear ground response analyses to compute response spectra at the ground surface for the MCE and DE levels of shaking;
- developed recommended, smooth, horizontal spectra for the MCE and DE levels of shaking.

To develop site-specific design response spectra for Block 32, we:

- performed PSHA to develop uniform hazard response spectrum for soil for a hazard level corresponding to a 2 percent probability of exceedance in 50 years (2,475-year return period)

- performed a deterministic analysis to develop a response spectrum for soil corresponding to 150 percent of the median deterministic analysis;
- developed the MCE ground surface spectrum as the lesser of the probabilistic and deterministic spectra described above;
- developed the Design Earthquake (DE) level of shaking response spectrum, consistent with the definition of 2010 SFBC which is 2/3 of the MCE spectrum;
- developed recommended, smooth, horizontal spectra for the MCE and DE levels of shaking

Details regarding our study are presented in the remainder of this Appendix.

E1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a Probabilistic Seismic Hazard Analysis (PSHA), which systematically should account for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are that:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data
- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake
- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

As part of the development of the MCE rock spectrum for Blocks 29, 30 and 31 and the ground surface spectrum for Block 32, we performed a PSHA to develop site-specific response spectra for a 2 percent probability of exceedance in 50 years. The spectra were developed using the computer code EZFRISK 7.62 (Risk Engineering 2011). The approach used in EZFRISK is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using attenuation relationships that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault, as well as the average shear wave velocity of the upper 100 feet of the rock or soil surface.

E1.1 Probabilistic Model

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

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

where:

ν_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i
P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z
$f_{Mi}(m)$ and $f_{Ri|Mi}(r;m)$ = probability density functions for magnitude and distance

Z represents peak rock or soil acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

E1.2 Source Modeling and Characterization

The segmentation of faults, maximum magnitudes, and recurrence rates were modeled using the data presented in the WGCEP (2008) and Cao et al. (2003) reports. Also, we included the combination of fault segments and their associated magnitudes and recurrence rates as described in the WGCEP (2008) in our seismic hazard model. Table E-1 presents the distance and direction from the site to the fault, mean characteristic magnitude, mean slip rate, and fault length for individual fault segments. We used the California fault database identified as "USGS08" in EZFRISK 7.62. We understand this database was obtained directly from USGS (McGuire 2005) and models the faults with multiple segments and includes background sources. Each segment is characterized with multiple magnitudes, occurrence or slip rates and weights. This approach takes into account the epistemic uncertainty associated with the various seismic sources in our model. Also, we included the USGS 2008 background source (gridded) for California included as part of the EZFRISK 7.62 source models.

TABLE E-1
Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Fault Length (km)
N. San Andreas; SAN+SAP	12.7	West	7.73		274
N. San Andreas; SAN+SAP+SAS	12.7	West	7.87		336
N. San Andreas; SAO+SAN+SAP	12.7	West	7.95		410
N. San Andreas; SAO+SAN+SAP+SAS	12.7	West	8.05		472
N. San Andreas; SAP	12.7	West	7.23	17	85
N. San Andreas; SAP+SAS	12.7	West	7.48	17	147
N. San Andreas; SAN	16	West	7.51	24	189
N. San Andreas; SAO+SAN	16	West	8.00	24	326
Hayward-Rodgers Creek; HN	16	Northeast	6.60	9	35
Hayward-Rodgers Creek; HN+HS	16	Northeast	7.00	9	87
Hayward-Rodgers Creek; RC+HN	16	Northeast	7.19	9	97
Hayward-Rodgers Creek; RC+HN+HS	16	Northeast	7.33	9	150
Hayward-Rodgers Creek; HS	17	East	6.78	9	52
San Gregorio Connected	19	West	7.50	5.5	176
Mount Diablo Thrust	33	East	6.70	2	25
Calaveras; CN	34	East	6.87	6	45
Calaveras; CN+CC	34	East	7.00		104
Calaveras; CN+CC+CS	34	East	7.03		123
Hayward-Rodgers Creek; RC	35	North	7.07	9	62
Green Valley Connected	38	East	6.80	4.7	56
Monte Vista-Shannon	39	Southeast	6.50	0.4	45
Point Reyes	43	West	6.90	0.3	47
West Napa	45	Northeast	6.70	1	30
Greenville Connected	51	East	7.00	2	50
Great Valley 5, Pittsburg Kirby Hills	55	East	6.70	1	32
Calaveras; CC	63	Southeast	6.39	15	59
Calaveras; CC+CS	63	Southeast	6.50	15	78
Great Valley 4b, Gordon Valley	69	Northeast	6.80	1.3	28
N. San Andreas; SAS	75	Southeast	7.12	17	62
Great Valley 7	76	East	6.90	1.5	45
Hunting Creek-Berryessa	77	North	7.10	6	60
Zayante-Vergeles	85	Southeast	7.00	0.1	58
Great Valley 4a, Trout Creek	91	Northeast	6.60	1.3	19
Maacama-Garberville	93	North	7.40	9	221
Monterey Bay-Tularcitos	98	Southeast	7.30	0.5	83

E1.3 Attenuation Relationships

Recently, Pacific Earthquake Engineering Research Center (PEER) embarked on the Next Generation Attenuation (NGA) project to update the previously developed attenuation relationships which were mostly published in 1997. We used the relationships by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008). These attenuation relationships include the average shear wave velocity in the upper 100 feet. Furthermore, these relationships were developed using different earthquake databases, therefore, the average of the relationships was used to develop the recommended spectra.

The NGA relationships database includes the most up to date recorded and processed data. They were developed for the orientation-independent geometric mean of the data. Geometric mean is defined as the square root of the product of the two recorded components.

E1.3.1 Blocks 29, 30 and 31

Subsurface information from our geotechnical investigation indicates Blocks 29, 30 and 31 are underlain by fill and Bay Mud. Stiff Old Bay Clay with interbedded sand lenses underlies the Bay Mud with bedrock at depths ranging from about 40 to 130 feet below existing ground surface. Subsurface conditions are discussed in more detail in Section E3.3. For the purpose of developing input motions for the ground response analyses, we developed the rock spectrum using an average shear wave velocity, V_{S30} of 760 m/sec (2,500 ft/sec) in the top 30 meters (100 feet) of the rock surface.

E1.3.2 Block 32

Subsurface information from our geotechnical investigation indicates Block 32 is underlain by fill and generally less than 10 feet of Bay Mud. We anticipate the fill will be improved through Rapid Impact Compaction. Stiff to hard clay with interbedded dense to very dense sand layers underlie the Bay Mud with bedrock at depths ranging from about 35 to 50 feet below existing ground surface. To develop the ground surface spectra, we used an average shear wave velocity, V_{S30} of 340 m/sec (1,120 ft/sec) in the top 30 meters (100 feet) of the soil profile. The site is categorized as a site class S_D.

E1.4 PSHA RESULTS

The results of the rock PSHA for Blocks 29, 30 and 31 and for soil for Block 32 are discussed in the following subsections.

E1.4.1 Blocks 29, 30 and 31

Figure E-1 present the results of the rock PSHA for the 2 percent probability of exceedance in 50 years hazard level for Blocks 29, 30 and 31. The average of the four attenuation relationships is also shown on this figure. These results are for the geometric mean of the recorded orthogonal components.

E1.4.2 Block 32

Figure E-2 present the results of the soil PSHA for the 2 percent probability of exceedance in 50 years hazard level for Block 32. The average of the four attenuation relationships is also shown on this figure. These results are for the geometric mean of the recorded orthogonal components.

E1.4.3 Deaggregation Results

Figure E-3 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, we conclude that the PSHA for the periods of interest at this site is dominated by an earthquake with a moment magnitude of 8.0 occurring on the San Andreas fault at about 12.7 kilometers.

E2.0 DETERMINISTIC ANALYSIS

We performed a deterministic analysis to develop the MCE rock spectrum for Blocks 29, 30 and 31 and MCE soil spectrum for Block 32. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. The MCE was defined as an event having a Moment Magnitude of 8.0 consistent with the mean magnitude assigned by WGCEP (2008) for a repeat of the 1906 earthquake on the San Andreas fault at a distance of about 12.7 kilometers from the site. This is consistent with the deaggregation results discussed in Section E1.4.3.

E2.1 Blocks 29, 30 and 31

The same attenuation relationships and shear wave velocities as discussed in Section E1.3.1 were used in the deterministic analysis to develop the rock spectrum for Blocks 29, 30 and 31. The median deterministic results of the four attenuation relationships, the average of these attenuation relationships as well as 150 percent of the average of the median deterministic are presented on Figures E-4.

E2.2 Block 32

The same attenuation relationships and shear wave velocities as discussed in Section E1.3.2 were used in the deterministic analysis to develop the soil spectrum for Blocks 32. The median deterministic results of the four attenuation relationships, the average of these attenuation relationships as well as 150 percent of the average of the median deterministic are presented on Figures E-5.

E3.0 DEVELOPMENT OF RECOMMENDED SPECTRA FOR BLOCKS 29, 30 AND 31

To develop the recommended ground surface spectra for Blocks 29, 30 and 31, we developed the recommended MCE and DE rock spectra, spectrally matched recorded time histories to the recommended spectrum, developed an idealized profile to model the subsurface conditions and performed equivalent linear ground response analyses to compute response spectra at the ground surface. The following subsections present details regarding the development of the recommended MCE and DE ground surface spectra for Blocks 29, 30 and 31

E3.1 DEVELOPMENT OF RECOMMENDED ROCK SPECTRA

The MCE as defined in the 2010 SFBC is the lesser of the PSHA spectrum having a 2 percent probability of exceedance in 50 years or 150 percent of the median deterministic spectrum for the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE spectrum. Figure E-6 presents a comparison between PSHA results for a 2 percent probability of exceedance in 50 years and

150 percent of the deterministic results for rock. The comparison indicates that 150 percent of the median deterministic results are less than the PSHA results for a 2 percent probability of exceedance in 50 years.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-05 to develop the site-specific spectra for the MCE and DE. Chapter 21 of ASCE 7-05 requires the checks listed below:

1. the deterministic spectrum used to develop the MCE shall not fall below the Deterministic Lower Limit spectrum as shown on Figure 21.2-1 of ASCE 7-05;
2. the DE spectrum shall not fall below 80 percent of general design spectrum (Section 21.3 of Chapter 21 ASCE 7-05).

Figure E-6 and Table E-2 present a comparison of the site-specific spectra for a 2 percent probability of exceedance in 50 years, 150 percent of the median deterministic, and the Deterministic Lower Limit spectra for Site Class B per ASCE 7-05. For all periods less than four seconds the spectrum for the 150 percent of the median deterministic is less than the Deterministic Lower Limit. Therefore, we recommend that the lesser of the Deterministic Lower Limit and 2 percent probability of exceedance in 50 years be used to develop the MCE. The recommended MCE spectrum is presented on Figure E-7 and in Table E-2.

TABLE E-2
Comparison of Site-specific and Code Spectra for Development of MCE Rock Spectrum
S_a (g) for 5 percent damping

Period (seconds)	PSHA – 2% probability of exceedance in 50 years	150% of the Median Deterministic	ASCE 7-05 Deterministic Lower Limit for S_B Site Class	Recommended MCE
0.01	0.628	0.430	1.500	0.628
0.10	1.300	0.795	1.500	1.300
0.20	1.564	0.959	1.500	1.500
0.30	1.339	0.853	1.500	1.339
0.40	1.153	0.740	1.500	1.153
0.50	1.000	0.644	1.200	1.000
0.60	0.873	0.566	1.000	0.873
0.75	0.739	0.484	0.800	0.739
1.00	0.581	0.387	0.600	0.581
2.00	0.300	0.205	0.300	0.300
3.00	0.194	0.139	0.200	0.194
4.00	0.138	0.100	0.150	0.138

Table E-3 presents the development of recommended DE rock spectrum following the procedures outlined in Chapter 21 of ASCE 7-05. DE is defined as 2/3 of the MCE per the 2010 SFBC; however, the recommended DE may not be below 80 percent of the general spectrum at any period (ASCE 7-05 Section 21.3). Figure E-6 and Table E-3 presents a comparison of 2/3 of the MCE spectrum and 80 percent of the general spectrum for site class B. With the exception of spectral value at a period of 0.4 second, 80 percent of the general spectrum is lower than 2/3 of the MCE spectrum and therefore we recommend 80 percent of general spectrum for this period and 2/3 of the MCE spectrum for all other periods to develop the DE spectrum. The recommended DE spectrum is shown on Figure E-7 and Table E-3.

TABLE E-3
Comparison of Site-specific and Code Spectra for Development of DE Rock Spectrum
S_a (g) for 5 percent damping

Period (seconds)	Recommended MCE	2/3 times MCE	80% of General Design Spectrum for S_B Soil Class	Recommended DE
0.01	0.628	0.419	0.320	0.419
0.10	1.300	0.866	0.800	0.866
0.20	1.500	1.000	0.800	1.000
0.30	1.339	0.893	0.800	0.893
0.40	1.153	0.769	0.800	0.800
0.50	1.000	0.667	0.660	0.667
0.60	0.873	0.582	0.550	0.582
0.75	0.739	0.493	0.440	0.493
1.00	0.581	0.387	0.330	0.387
2.00	0.300	0.200	0.174	0.200
3.00	0.194	0.129	0.110	0.129
4.00	0.138	0.092	0.083	0.092

The recommended MCE and DE rock spectra as well as a comparison with the MCE and DE 2010 SFBC S_B spectra are presented on Figure E-7. Digitized values of the recommended MCE and DE rock spectra are presented in Tables E-2 and E-3, respectively, for a damping ratio of 5 percent.

E3.2 TIME HISTORY MATCHING FOR ROCK SPECTRA

To develop time histories that are compatible with the recommended DE rock spectrum shown on Figure E-7, we performed spectral matching of the rock spectrum with actual recorded ground motions.

The selection of a recorded time history is an important step in developing the ground motion. The intent in this selection process is to choose time histories that have a similar magnitude and distance to the design ground motion. In addition, the use of different earthquakes captures the unique and different character of each particular earthquake. Table E-4 presents the six earthquake time histories used in the spectral matching for rock.

TABLE E-4
Earthquake Time Histories Used
For Matching Rock Spectra

Earthquake, Year	Recording	Magnitude	Closest Distance to Rupture (km)	Peak Acceleration (g)
Imperial Valley, 1940	El Centro – 270 deg.	7.0	6	0.215
Loma Prieta, 1989	Corralitos - 90 deg.	6.9	5	0.644
Loma Prieta, 1989	Bran - 0 deg.	6.9	11	0.453
Kocaeli, 1999	Gebze - 0 deg.	7.4	17	0.244
Denali, 1999	PS10 – 47 deg.	7.9	3	0.319
Landers, 1992	Joshua Tree - 90 deg.	7.4	12	0.274

The tabulated reference time histories were modified such that their response spectrum matched the target spectrum. The computer program EZFRISK 7.62 was used to perform the spectral matching. The spectral matching was performed in the time domain. Figures E-8 through E-13 present the acceleration, velocity and displacement of the matched time histories along with the comparison between the target and the matched DE spectrum. The spectrally matched time histories were scaled by a factor of 1.5 to develop time histories for the MCE for use in the ground response analysis.

E3.3 DESIGN RESPONSE SPECTRA

To develop site-specific response spectra, rock motions are modified to take into account the soil conditions at the site. We developed idealized profiles to model the subsurface conditions. The soil profile is based on data from current and previous investigations at the site. The profiles generally consist of fill over Bay Mud which was in turn underlain by interbedded layers of silty sand, clayey sand and/or sandy clay. Below these layers are dense to very dense sand with varying amounts of fines of the Colma formation. The Colma formation overlies stiff to very stiff clay locally referred to as Old Bay Clay. We developed two idealized profiles designated as "shallow" and deep for each of the blocks to account for the variability in the thicknesses of the layers and depth to bedrock. Table E-5 presents the thickness of the various layers as well as the depth to bedrock for each of the two profiles for Blocks 29, 30 and 31.

TABLE E-5
Summary of Layer Thickness
For the Idealized Subsurface Profiles

Soil Description	Block 29		Block 30		Block 31	
	Shallow Profile – Layer Thickness (ft)	Deep Profile – Layer Thickness (ft)	Shallow Profile – Layer Thickness (ft)	Deep Profile – Layer Thickness (ft)	Shallow Profile – Layer Thickness (ft)	Deep Profile – Layer Thickness (ft)
Fill	27	20	11	9	10	15
Bay Mud	20	40	28	45	10	29
Clayey or Silty Sand/Sandy Clay	22	10	12	13	10	12
Colma Sand	26	35	9	30	10	34
Old Bay Clay	6	25	-	9	-	-
Depth to top of rock (feet)	101	130	60	106	40	90

Our analyses assume the upper 15 feet of potentially liquefiable fill will be mitigated.

E3.4 SHAKE Results

Response spectra at the ground surface for Blocks 29 and 31 and at the basement level for Block 30 were computed using the computer program SHAKE-91, a one-dimensional, site response analysis based on vertically propagating horizontal shear waves. The program mathematically transmits input bedrock motions vertically through an idealized soil column to the ground surface. To account for the non-linear characteristics of soil, this program uses equivalent-linear procedures with strain compatible shear moduli and damping ratios. The six matched time histories discussed in section E3.2 were used as input rock outcrop motions for the DE level of shaking; the DE time histories were scaled by a factor of 1.5 for input as MCE time histories. Portions of the proposed buildings at Blocks 29, 30 and 31 have basement levels. Because the proposed basement levels do not extend beneath most of the building footprint, the SHAKE results for Blocks 29 and 31 were calculated at the ground surface level. However, plans indicate that the entire Block 30 building footprint will be underlain by basement levels that extend to depths on the order of 25 feet. Therefore the SHAKE results were calculated at the depth of the bottom of the proposed basement level for Block 30.

E3.4.1 Block 29

The results of the SHAKE analyses for the DE level of shaking are presented on Figures E-14 and E-15 for the shallow and deep profile, respectively. Similar plots are presented on Figure E-16 and E-17 for the MCE level of shaking. The average of the results is also presented on each figure. Figure E-18 presents the average SHAKE results for each profile for both the DE and MCE as well as 80 percent of the 2010 SFBC DE and MCE spectra for site class E. The recommended spectra may not be less than 80 percent of code spectra. The recommended smooth DE and MCE spectra are also shown on Figure E-18.

E3.4.2 Block 30

The results of the SHAKE analyses for the DE level of shaking at the basement level are presented on Figures E-19 and E-20 for the shallow and deep profile, respectively. Similar plots are presented on Figure E-21 and E-22 for the MCE level of shaking. The average of the results is also presented on each figure. Figure E-23 presents the average SHAKE results for each profile for both the DE and MCE as well as 80 percent of the 2010 SFBC DE and MCE spectra for site class E. The recommended spectra may not be less than 80 percent of code spectra. The recommended smooth DE and MCE spectra for use at the basement level are also shown on Figure E-23.

E3.4.3 Block 31

The results of the SHAKE analyses for the DE level of shaking are presented on Figures E-24 and E-25 for the shallow and deep profile, respectively. Similar plots are presented on Figure E-26 and E-27 for the MCE level of shaking. The average of the results is also presented on each figure. Figure E-28 presents the average SHAKE results for each profile for both the DE and MCE as well as 80 percent of the 2010 SFBC DE and MCE spectra for site class E. The recommended spectra may not be less than 80 percent of code spectra. The recommended smooth DE and MCE spectra are also shown on Figure E-28.

E4.0 DEVELOPMENT OF RECOMMENDED SPECTRA FOR BLOCK 32

To develop the ground surface spectra for Block 32 we used the same procedure as discussed in Section E3.1, which was used to develop the rock spectra. Figure E-29 and Table E-6 present a comparison of the site-specific spectra for a 2 percent probability of exceedance in 50 years, 150 percent of the median deterministic, and the Deterministic Lower Limit spectra for Site Class D per ASCE 7-05. For all periods less than four seconds the spectrum for the 150 percent of the median deterministic is less than the Deterministic Lower Limit. Therefore, we recommend that the lesser of the Deterministic Lower Limit and 2 percent probability of exceedance in 50 years be used to develop the MCE. The recommended MCE spectrum is presented on Figure E-30 and in Table E-6.

TABLE E-6
Comparison of Site-specific and Code Spectra for Development of MCE Ground Surface Spectrum, S_a (g) for 5 percent damping

Period (seconds)	PSHA – 2% probability of exceedance in 50 years	150% of the Median Deterministic	ASCE 7-05 Deterministic Lower Limit for S_D Site Class	Recommended MCE
0.01	0.758	0.492	1.500	0.758
0.05	0.940	0.584	1.500	0.940
0.10	1.319	0.793	1.500	1.319
0.20	1.706	1.008	1.500	1.500
0.30	1.725	1.037	1.500	1.500
0.40	1.617	0.988	1.500	1.500
0.50	1.503	0.932	1.500	1.500
0.60	1.384	0.865	1.500	1.384
0.75	1.252	0.789	1.200	1.200
1.00	1.058	0.660	0.900	0.900
1.50	0.783	0.495	0.600	0.600
2.00	0.603	0.384	0.450	0.450
3.00	0.406	0.267	0.300	0.300
4.00	0.294	0.194	0.225	0.225

Table E-7 presents the development of recommended DE spectrum at the ground surface following the procedures outlined in Chapter 21 of ASCE 7-05. The DE is defined as 2/3 of the MCE per the 2010 SFBC; however, the recommended DE may not be below 80 percent of the general spectrum at any period (ASCE 7-05 Section 21.3). Figure E-29 and Table E-7 presents a comparison of 2/3 of the MCE spectrum and 80 percent of the general spectrum for site class D. At all periods 2/3 of the MCE spectrum is greater than 80 percent of the general spectrum and hence govern. The recommended DE spectrum is shown on Figure E-30 and Table E-3.

TABLE E-7
Comparison of Site-specific and Code Spectra for Development of DE Ground Surface Spectrum, S_a (g) for 5 percent damping

Period (seconds)	Recommended MCE	2/3 times MCE	80% of General Design Spectrum for S_D Site Class	Recommended DE
0.01	0.758	0.505	0.320	0.505
0.05	0.940	0.627	0.512	0.627
0.10	1.319	0.880	0.704	0.880
0.20	1.500	1.000	0.800	1.000
0.30	1.500	1.000	0.800	1.000
0.40	1.500	1.000	0.800	1.000
0.50	1.500	1.000	0.800	1.000
0.60	1.384	0.923	0.800	0.923
0.75	1.200	0.800	0.664	0.800
1.00	0.900	0.600	0.498	0.600
1.50	0.600	0.400	0.332	0.400
2.00	0.450	0.300	0.249	0.300
3.00	0.300	0.200	0.166	0.200
4.00	0.225	0.150	0.125	0.150

The recommended MCE and DE ground surface spectra for Block 32 as well as a comparison with the MCE and DE 2010 SFBC S_D spectra are presented on Figure E-30.

E5.0 Recommended Spectra

The recommended horizontal spectra for Blocks 29, 30, 31 and 32 are shown on Figures E-31, E-32, E-33 and E-34, respectively. Digitized values of the recommended horizontal MCE and DE spectra for Blocks 29, 30, and 31 for a damping ratio of 5 percent are presented in Tables E-8, E-9, E-10 and E-11, respectively.

TABLE E-8
Recommended Spectra for Block 29
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.419	0.357
0.10	0.542	0.434
0.20	0.735	0.618
0.30	0.924	0.791
0.40	1.075	0.908
0.50	1.183	0.973
0.60	1.270	1.010
0.70	1.325	1.040
0.80	1.350	1.070
0.90	1.360	1.085
1.00	1.365	1.070
1.10	1.365	1.020
1.20	1.300	0.929
1.30	1.200	0.778
1.40	1.065	0.651
1.50	0.925	0.563
1.60	0.809	0.506
1.70	0.720	0.468
1.80	0.664	0.442
1.90	0.628	0.418
2.00	0.596	0.398
2.10	0.568	0.379
2.20	0.543	0.362
2.30	0.519	0.346
2.40	0.497	0.332
2.50	0.478	0.318
2.60	0.459	0.306
2.70	0.442	0.295
2.80	0.426	0.284
2.90	0.412	0.274
3.00	0.398	0.265
3.10	0.385	0.257
3.20	0.373	0.249
3.30	0.362	0.241
3.40	0.351	0.234
3.50	0.341	0.227
3.60	0.332	0.221
3.70	0.323	0.215
3.80	0.314	0.209
3.90	0.306	0.204
4.00	0.298	0.199

TABLE E-9
Recommended Spectra for Block 31
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.554	0.446
0.10	0.775	0.582
0.20	0.982	0.763
0.30	1.154	0.929
0.40	1.280	1.048
0.50	1.356	1.111
0.60	1.387	1.124
0.70	1.379	1.099
0.80	1.341	1.049
0.90	1.282	0.989
1.00	1.211	0.927
1.10	1.130	0.868
1.20	1.059	0.811
1.30	0.985	0.737
1.40	0.912	0.664
1.50	0.843	0.597
1.60	0.780	0.537
1.70	0.724	0.487
1.80	0.674	0.442
1.90	0.628	0.418
2.00	0.596	0.398
2.10	0.568	0.379
2.20	0.543	0.362
2.30	0.519	0.346
2.40	0.497	0.332
2.50	0.478	0.318
2.60	0.459	0.306
2.70	0.442	0.295
2.80	0.426	0.284
2.90	0.412	0.274
3.00	0.398	0.265
3.10	0.385	0.257
3.20	0.373	0.249
3.30	0.362	0.241
3.40	0.351	0.234
3.50	0.341	0.227
3.60	0.332	0.221
3.70	0.323	0.215
3.80	0.314	0.209
3.90	0.306	0.204
4.00	0.298	0.199

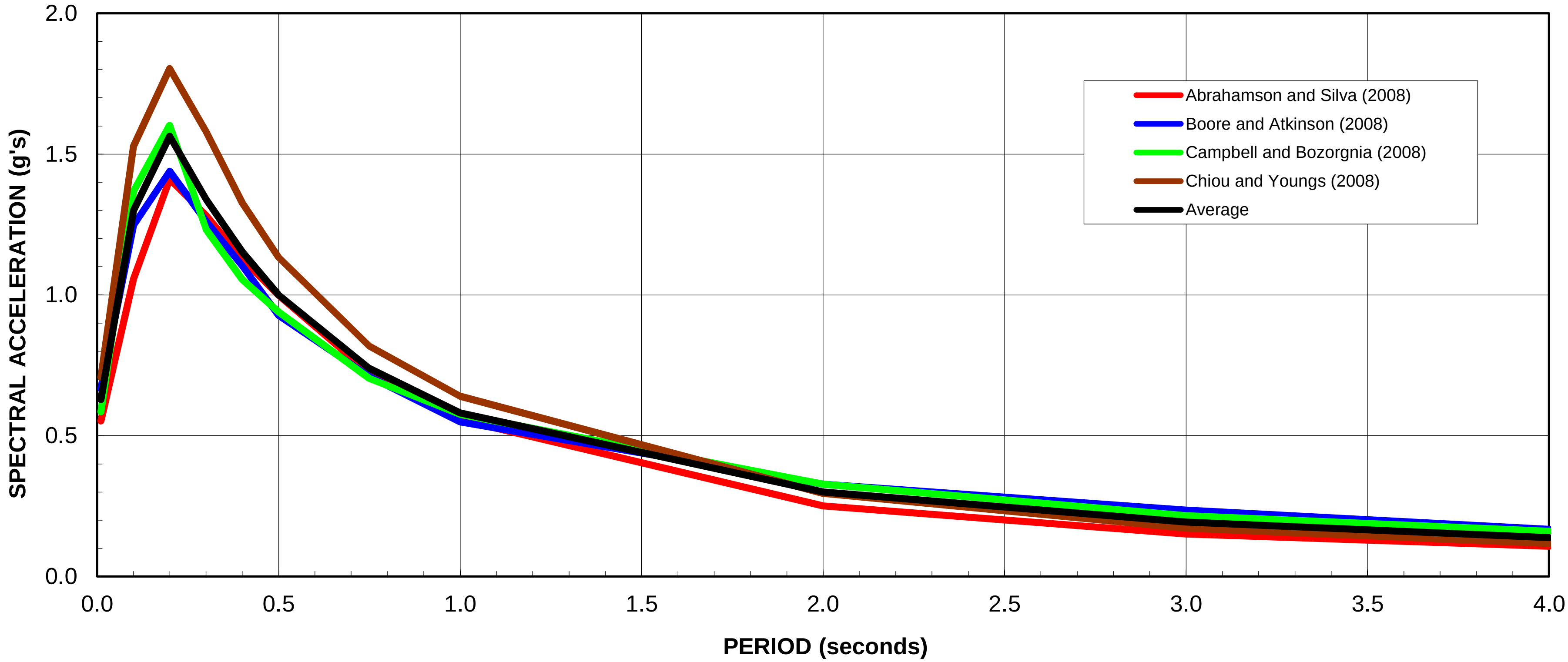
TABLE E-10
Recommended Spectra for Block 32
(5 percent damping)

Period (seconds)	MCE	DE
0.00	0.758	0.505
0.05	0.940	0.627
0.10	1.319	0.880
0.20	1.500	1.000
0.30	1.500	1.000
0.40	1.500	1.000
0.50	1.500	1.000
0.60	1.384	0.923
0.75	1.200	0.800
1.00	0.900	0.600
1.50	0.600	0.400
2.00	0.450	0.300
3.00	0.300	0.200
4.00	0.225	0.150

Because we developed site-specific response spectra, the site coefficients in accordance with Section 21.4 of ASCE 7-05 for S_{MS}, S_{M1}, S_{DS} and S_{D1} are presented in Table E-11.

TABLE E-11
Site Coefficients per Section 21.4 of ASCE 7-05
(5 percent damping)

Site Coefficient	Block 29	Block 30	Block 31	Block 32
S_{MS}	1.229	0.991	1.248	1.500
S_{M1}	1.365	1.192	1.211	0.900
S_{DS}	0.977	0.869	1.012	1.000
S_{D1}	1.070	0.896	0.927	0.600



Damping Ratio = 5%

Note: Estimated Bedrock V_{S30} = 760 m/s

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BLOCKS 29-32 MISSION BAY San Francisco, California		
RESULTS OF PSHA FOR ROCK, 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS BLOCKS 29, 30 AND 31		
Date 12/13/11	Project No. 750603902	Figure E-1

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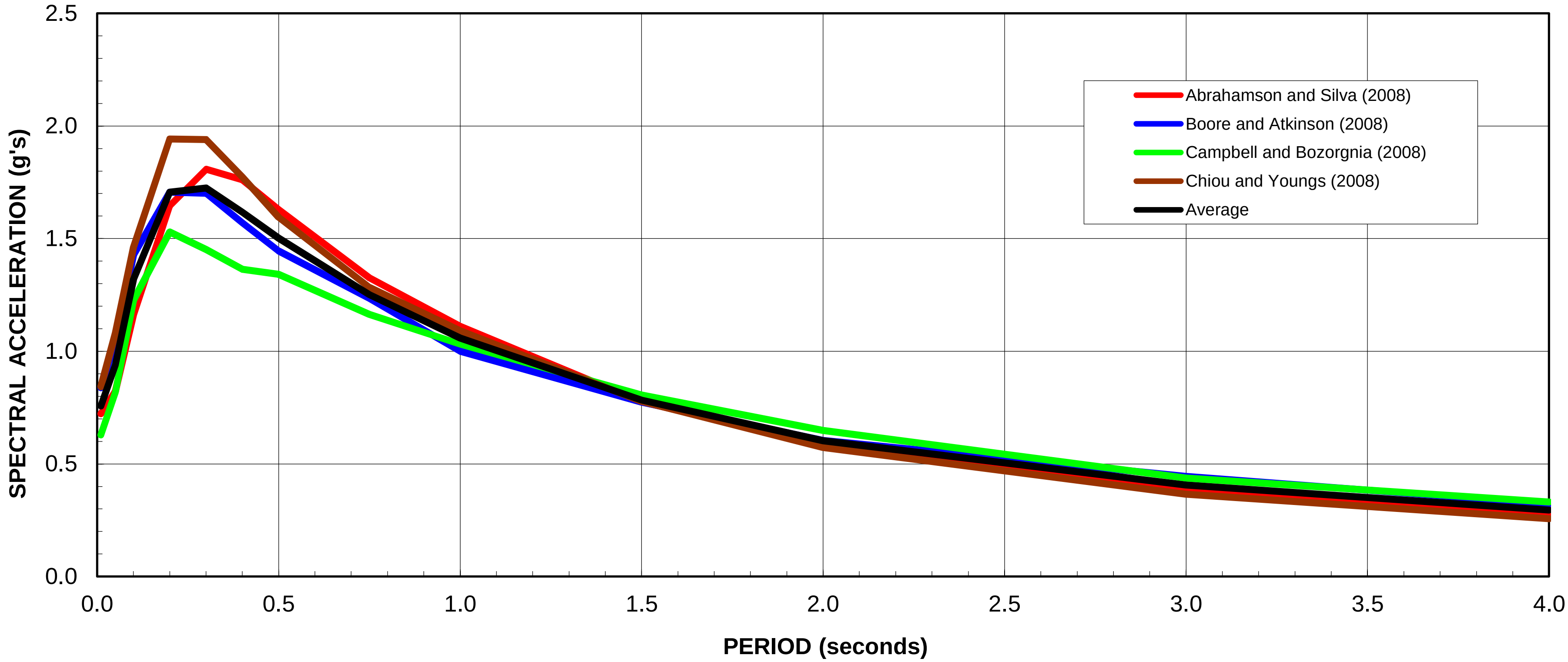

DRAFT

Damping Ratio = 5%

Note: Estimated V_{S30} = 340 m/s

BLOCKS 29-32
MISSION BAY
San Francisco, California

RESULTS OF PSHA, 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS FOR BLOCK 32

Date 12/13/11	Project No. 750603902	Figure E-2

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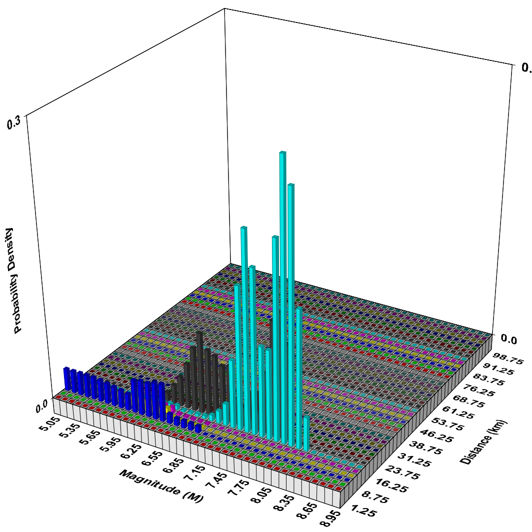
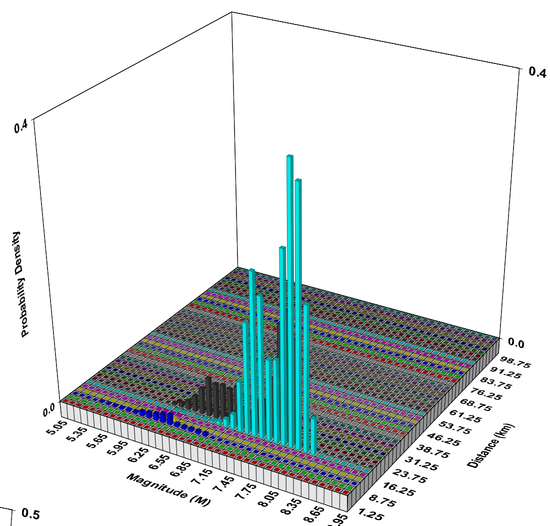
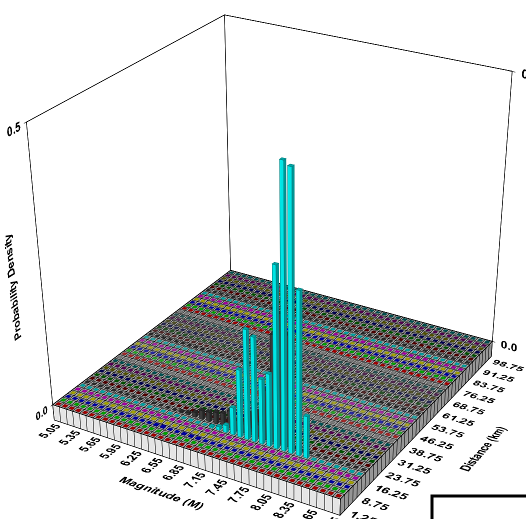


(a) PGA



(b) S_a, T = 1.0 seconds



(c) S_a, T = 4.0 seconds

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BLOCKS 29-32
MISSION BAY
San Francisco, California

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS FOR ROCK - MAGNITUDE DISTANCE DEAGGREGATION PLOTS FOR BLOCKS 29, 30 AND 31

Date 12/13/11	Project No. 750603902	Figure E-3a

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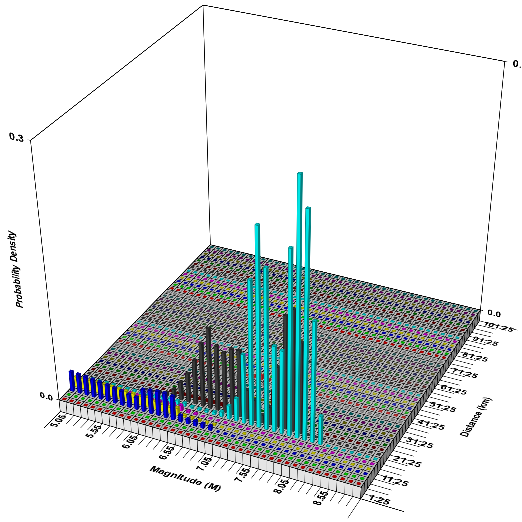
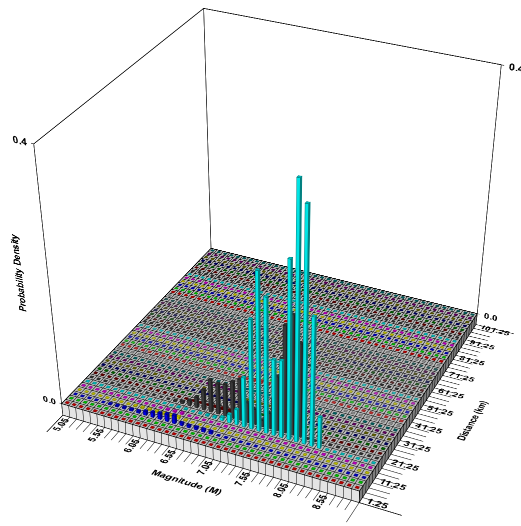
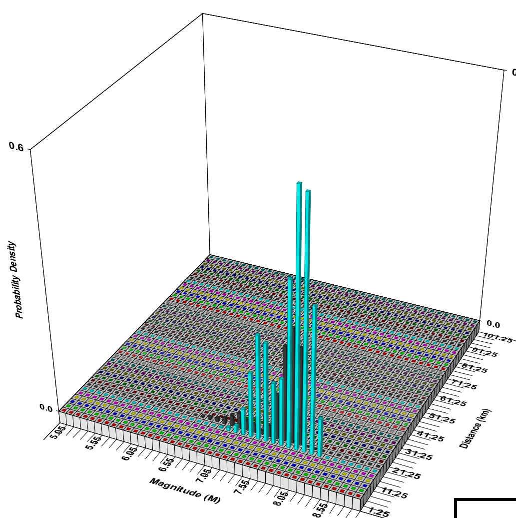


(a) PGA



(b) S_a, T = 1.0 seconds



(c) S_a, T = 4.0 seconds

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BLOCKS 29-32
MISSION BAY
San Francisco, California

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS FOR ROCK - MAGNITUDE DISTANCE DEAGGREGATION PLOTS FOR BLOCK 32

Date 12/13/11	Project No. 750603902	Figure E-3b

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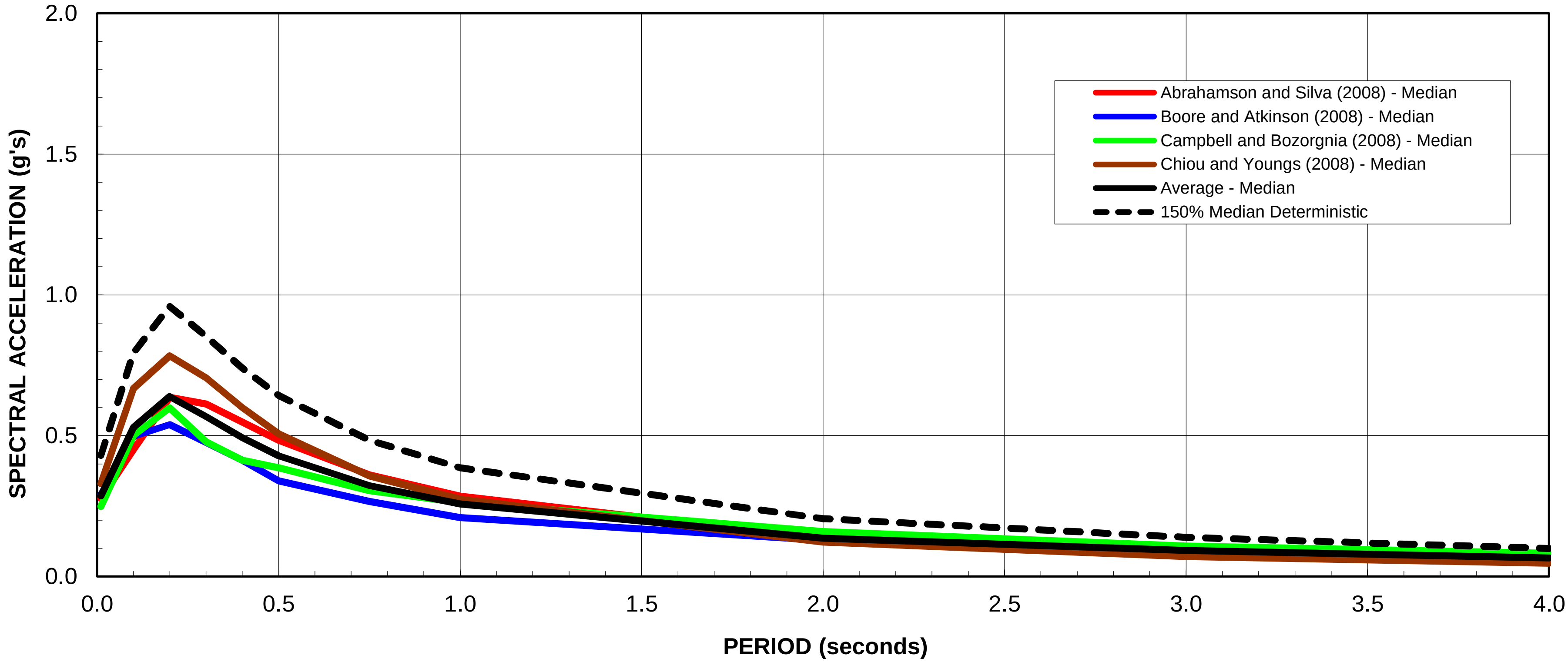

Damping Ratio = 5%

Note: Estimated Bedrock V_{S30} = 760 m/s

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BLOCKS 29-32 **MISSION BAY** San Francisco, California		
RESULTS OF DETERMINISTIC ANALYSIS FOR ROCK **M_w = 8.0, Dist. = 12.7 km FOR BLOCKS 29, 30 AND 31**		
Date 12/13/11	Project No. 750603902	Figure E-4

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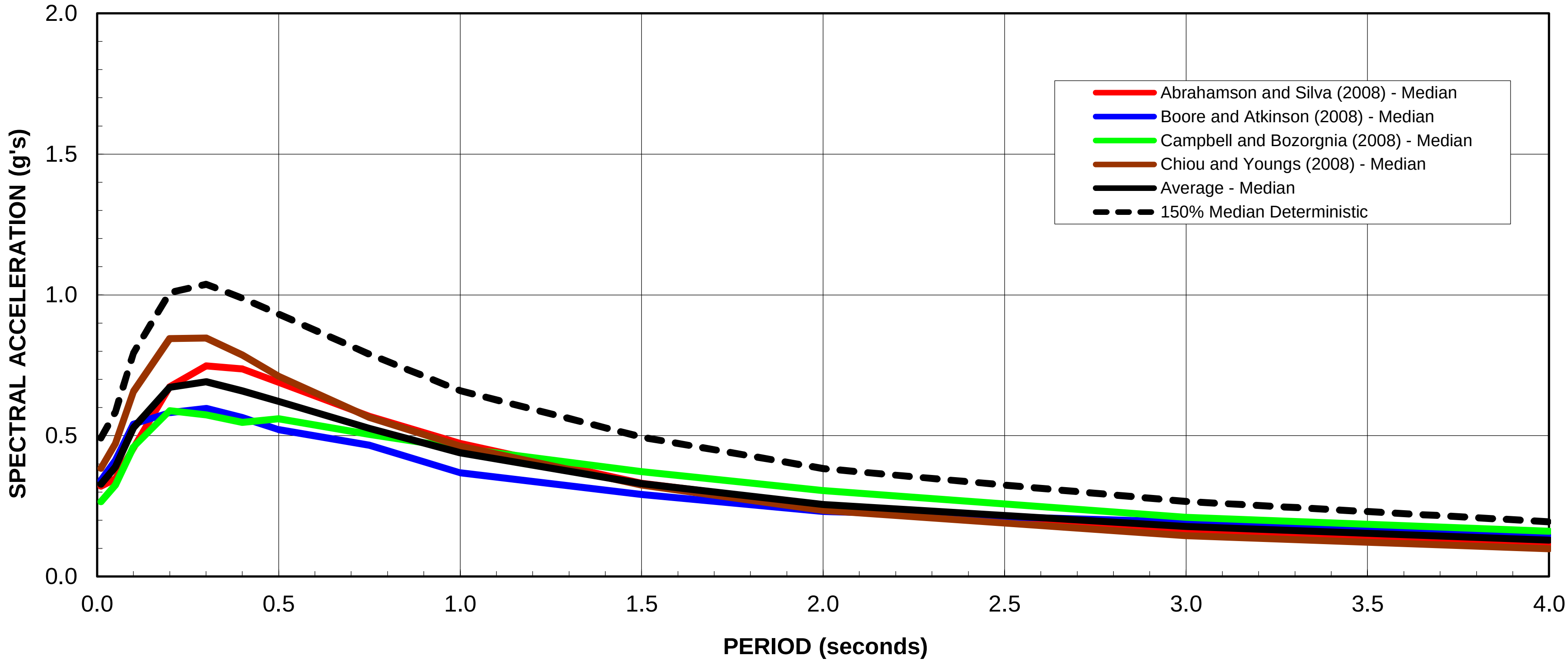

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Damping Ratio = 5%

Note: Estimated V_{S30} = 340 m/s

BLOCKS 29-32
MISSION BAY
San Francisco, California

RESULTS OF DETERMINISTIC ANALYSIS
M_w = 8.0, Dist. = 12.6 km FOR BLOCK 32

Date 12/13/11	Project No. 750603902	Figure E-5

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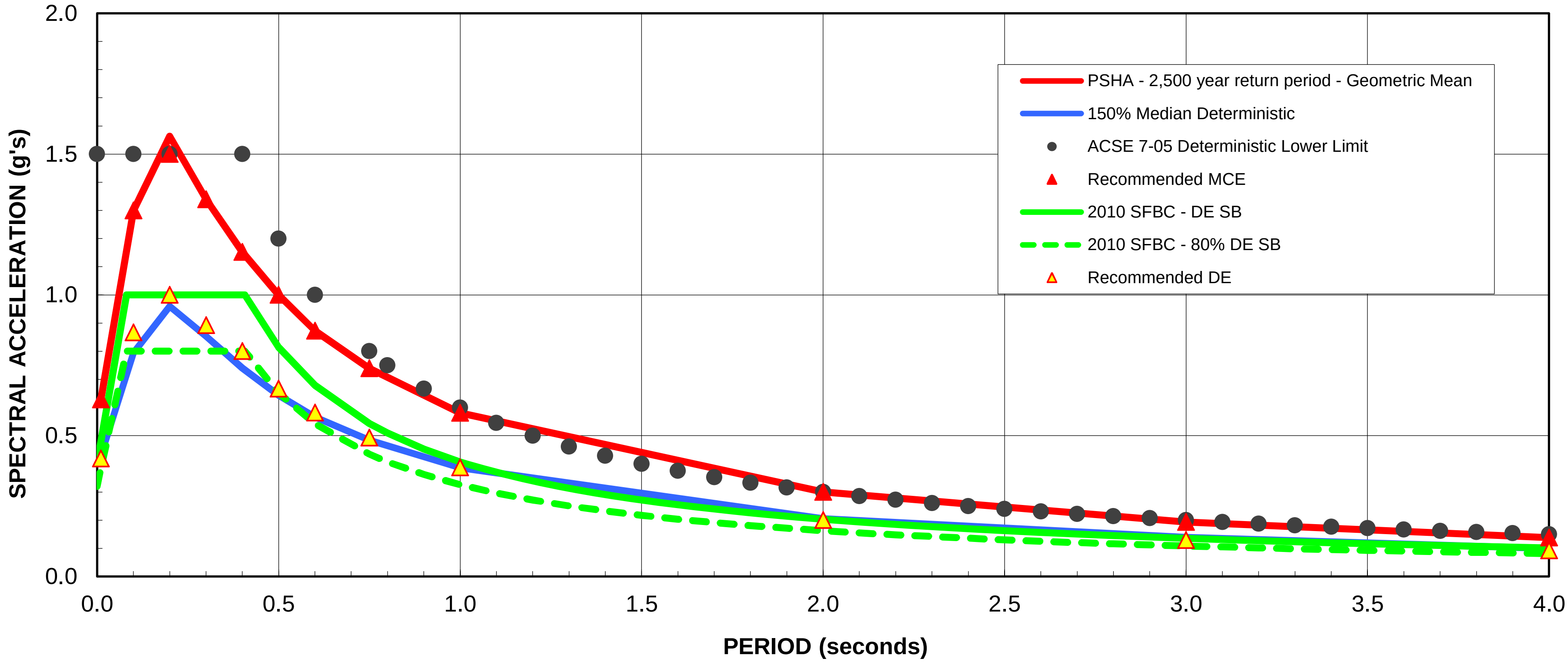

Damping Ratio = 5%

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Note: M_w = 8.0, Dist. 12.7 km, estimated Average V_{S30} = 760 m/s

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
COMPARISON OF ROCK DETERMINISTIC, PROBABILISTIC AND CODE SPECTRA BLOCKS 29, 30 AND 31		
Date 12/13/11	Project No. 750603902	Figure E-6

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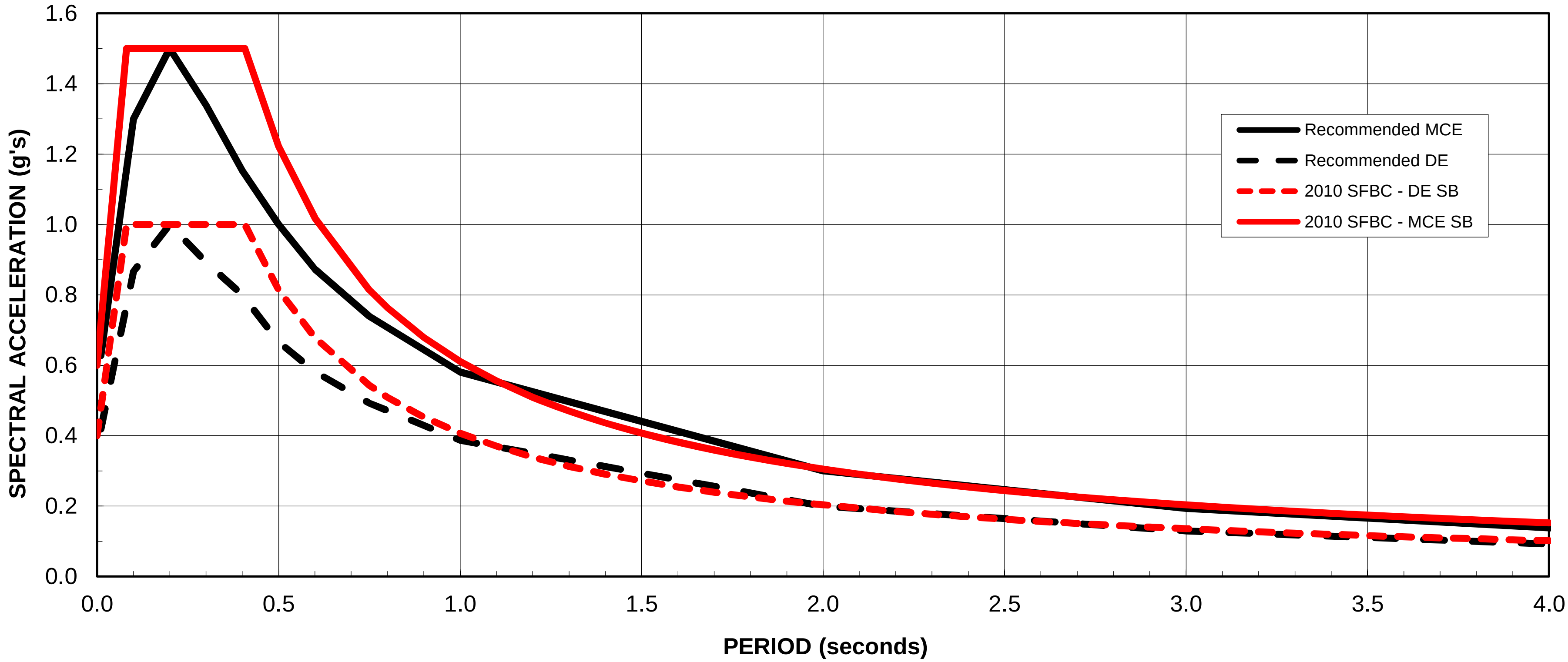

Damping Ratio = 5%

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BLOCKS 29-32 **MISSION BAY** San Francisco, California		
COMPARISON OF RECOMMENDED ROCK AND CODE SPECTRA FOR BLOCKS 29, 30 AND 31		
Date 12/13/11	Project No. 750603902	Figure E-7
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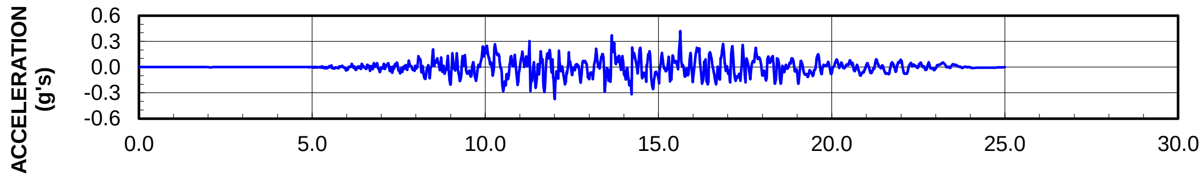
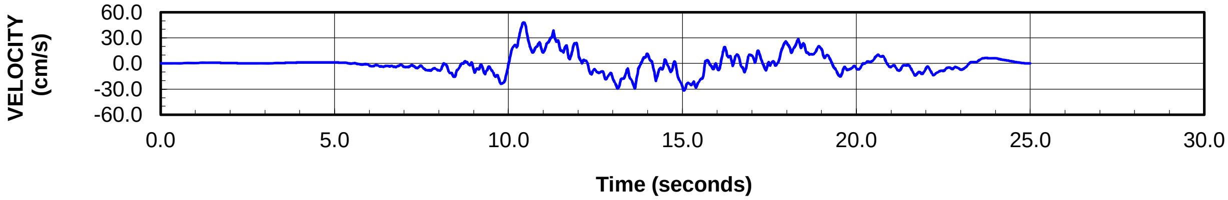
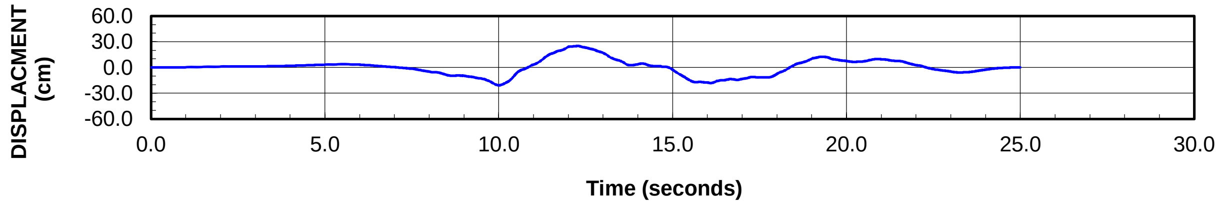
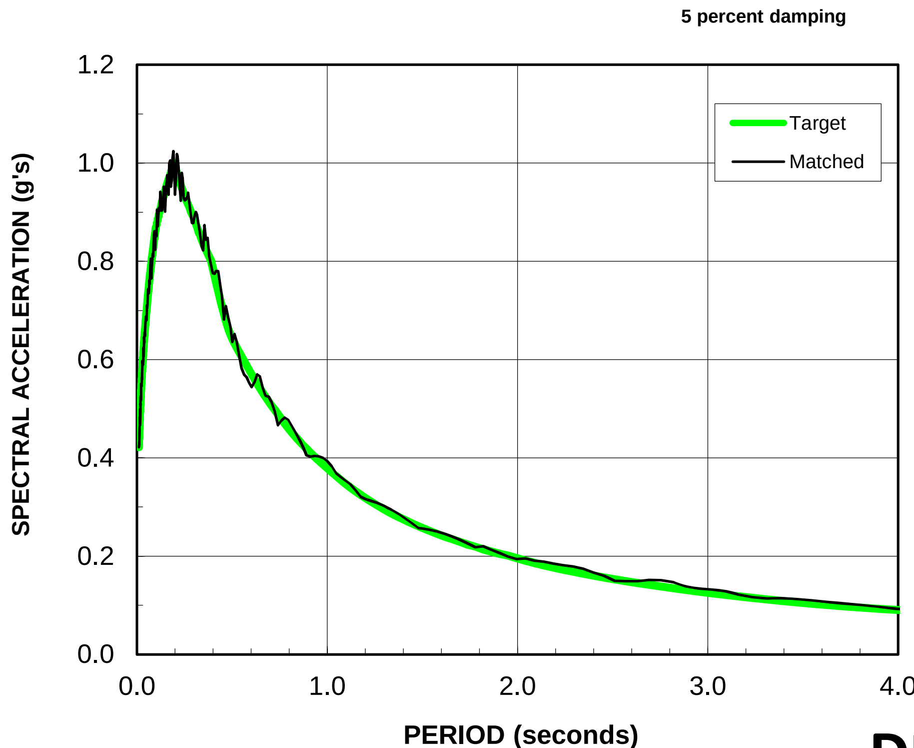


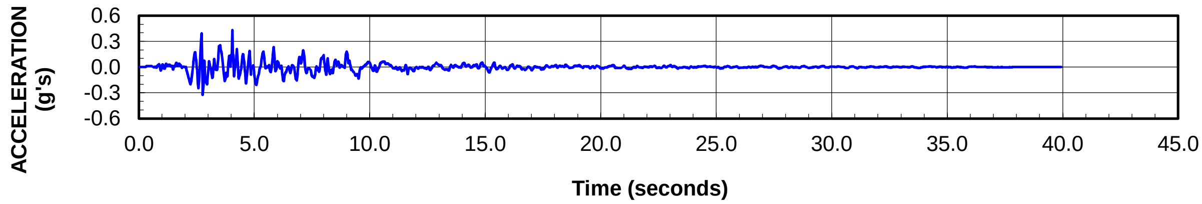


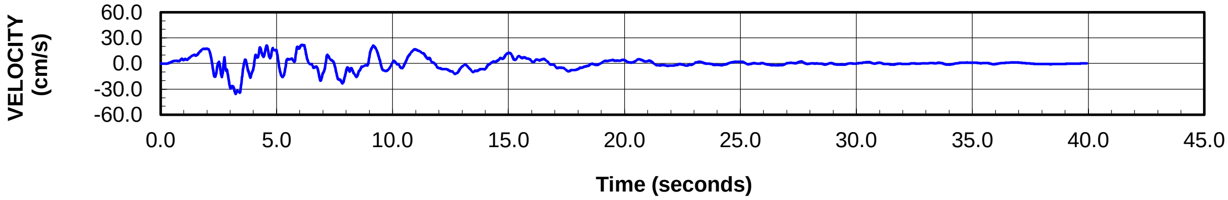


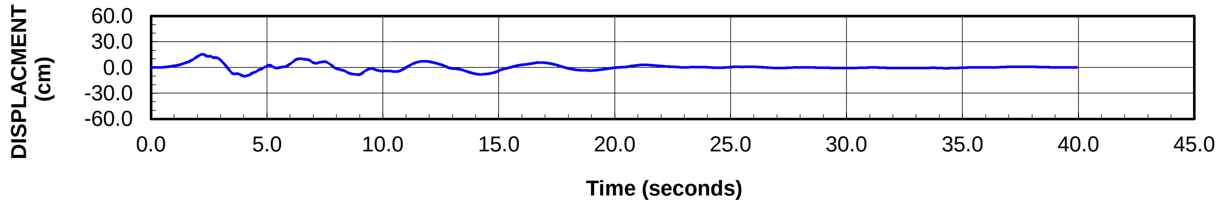


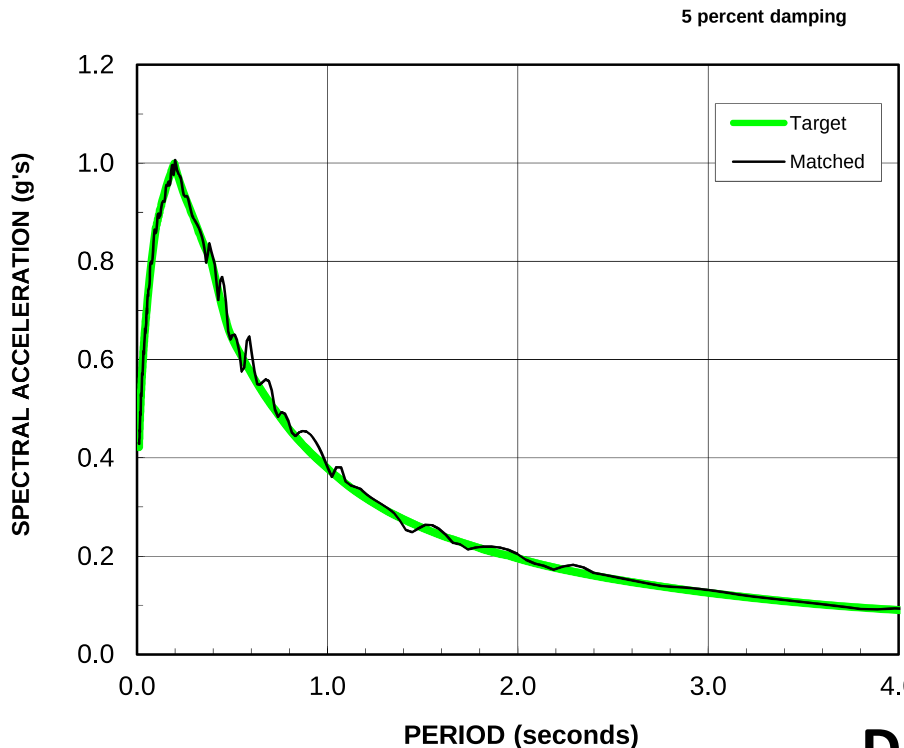
DRAFT

BLOCK 29-32 **MISSION BAY** San Francisco, California	**DE MATCHED TIME HISTORY AND RESPONSE SPECTRUM 1989 LOMA PRIETA EARTHQUAKE BRAN 0 Degs.**
Treadwell&Rollo A LANGAN COMPANY	Date 11/23/11 | Project No. 750603902 | Figure E-8









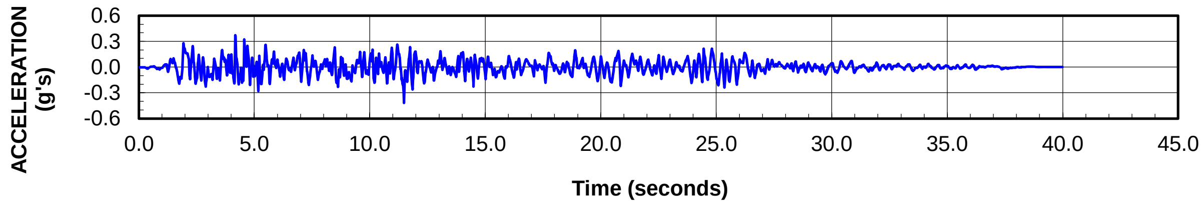
DRAFT

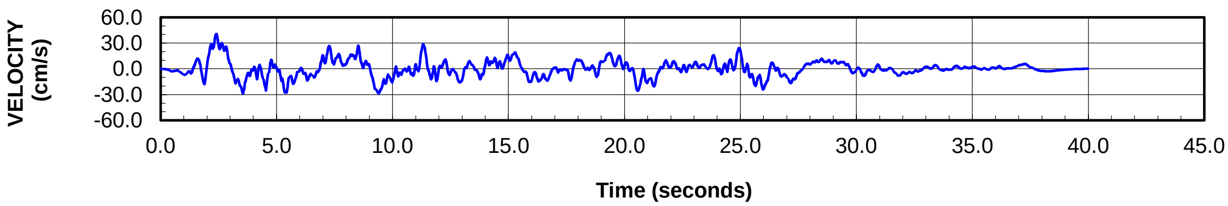
BLOCK 29-32
MISSION BAY
San Francisco, California

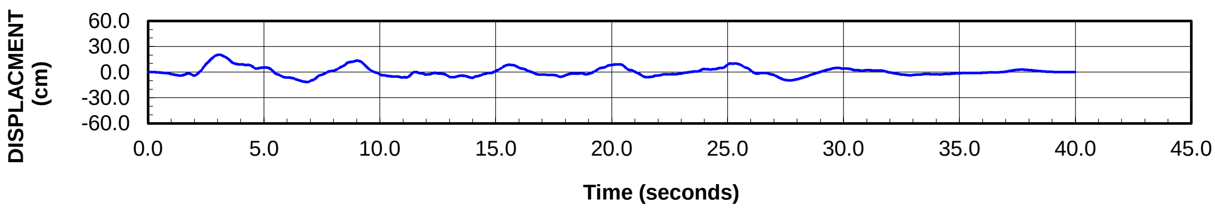
Treadwell&Rollo
A LANGAN COMPANY

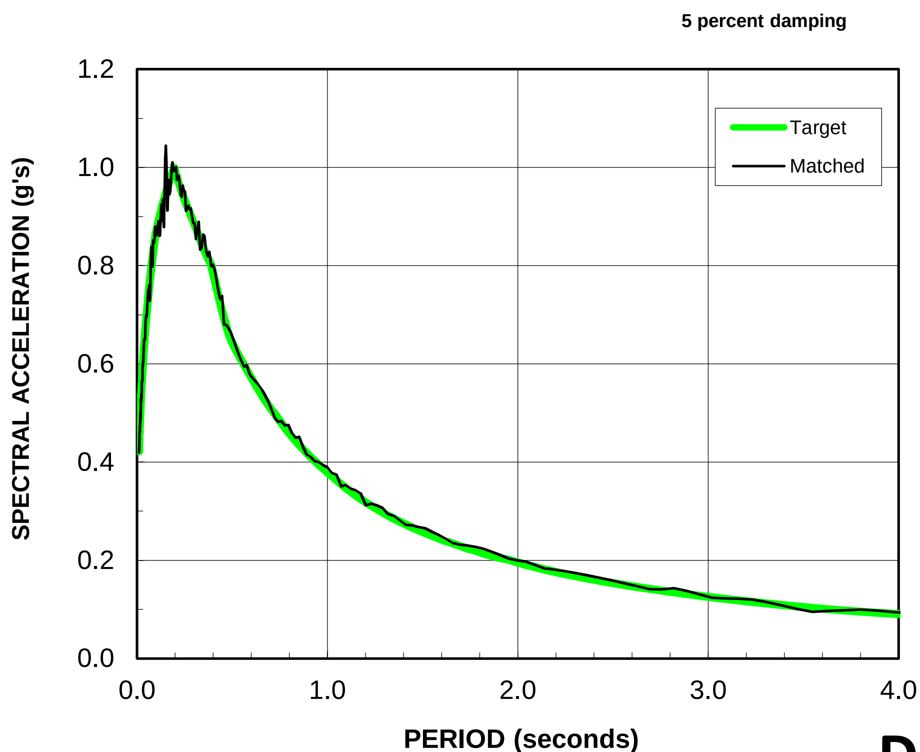
DE MATCHED TIME HISTORY AND RESPONSE SPECTRUM 1989 LOMA PRIETA EARTHQUAKE CORRALITOS 90 Degs.

Date 11/23/11	Project No. 750603902	Figure E-9



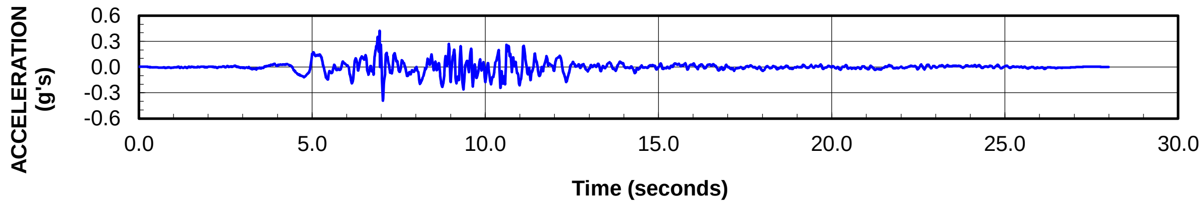


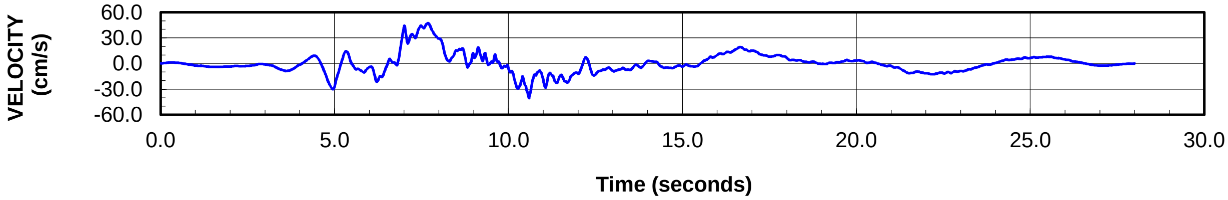


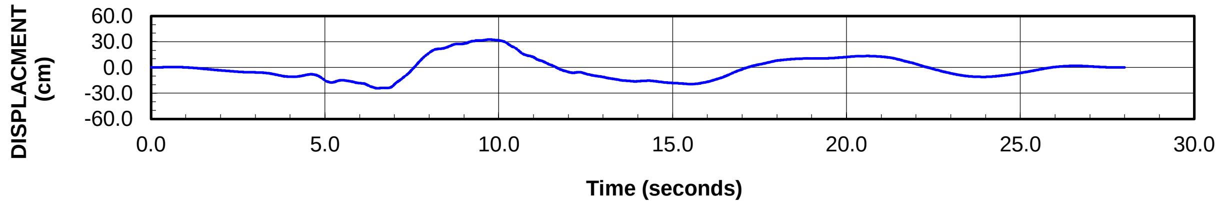


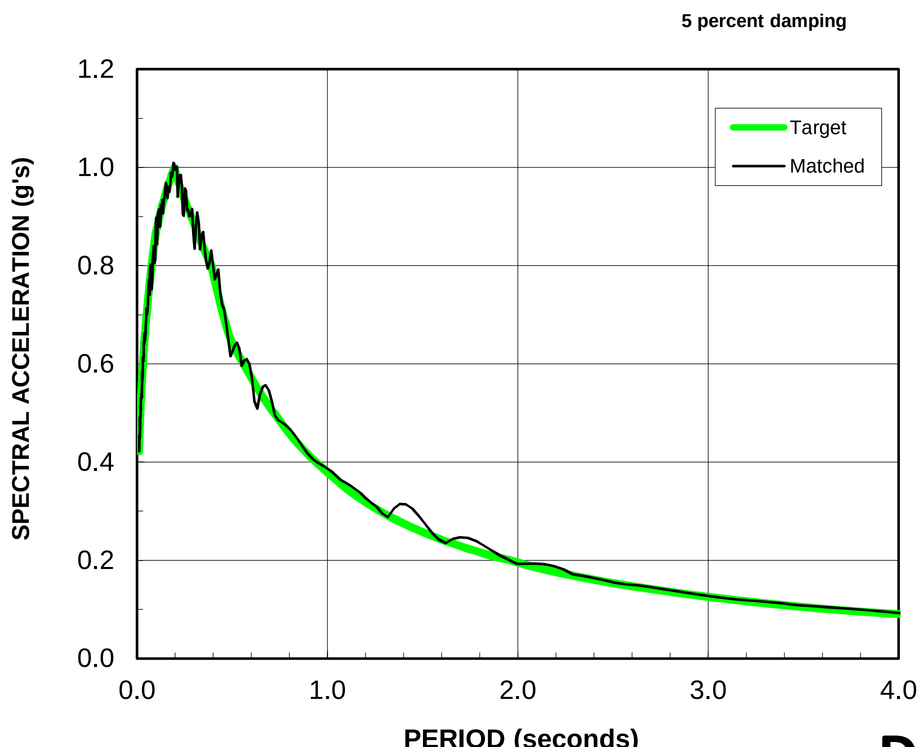
DRAFT

BLOCK 29-32 **MISSION BAY** San Francisco, California	**DE MATCHED TIME HISTORY AND RESPONSE SPECTRUM 1940 IMPERIAL VALLEY EARTHQUAKE EL CENTRO 270 Degs.**
Treadwell&Rollo A LANGAN COMPANY	Date 11/23/11 | Project No. 750603902 | Figure E-10









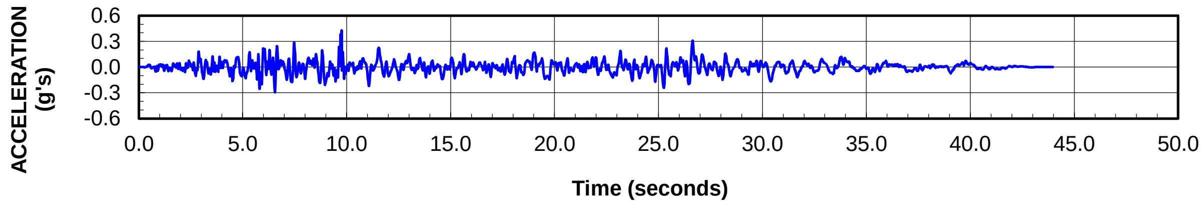
DRAFT

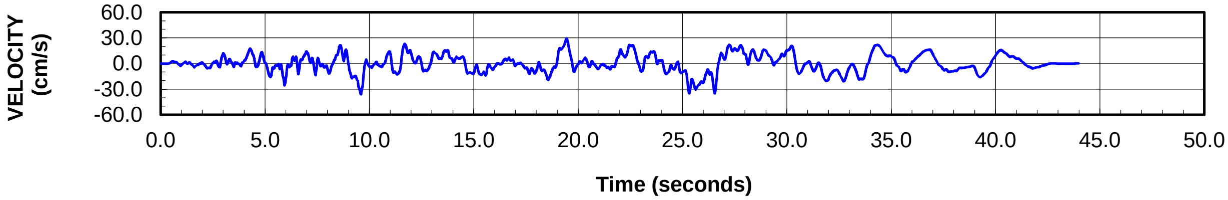
BLOCK 29-32
MISSION BAY
San Francisco, California

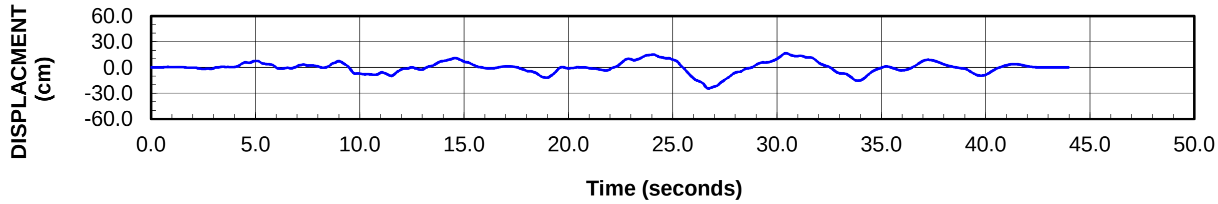
Treadwell&Rollo
A LANGAN COMPANY

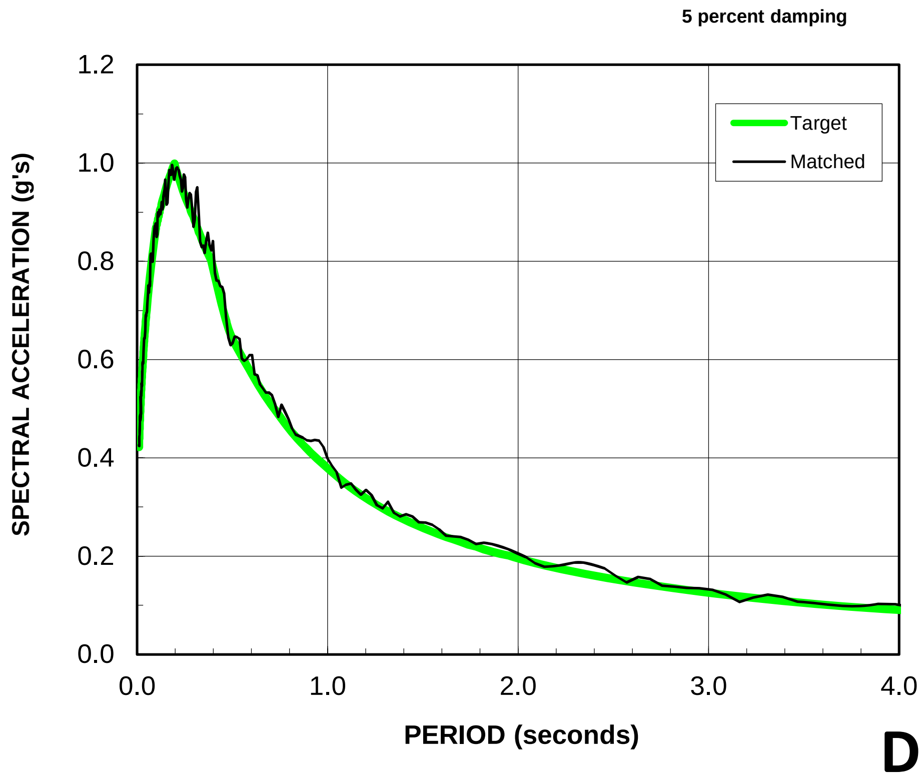
DE MATCHED TIME HISTORY AND RESPONSE SPECTRUM 1999 KOCAELI EARTHQUAKE GEBZE 0 Degs.

Date 11/23/11	Project No. 750603902	Figure E-11



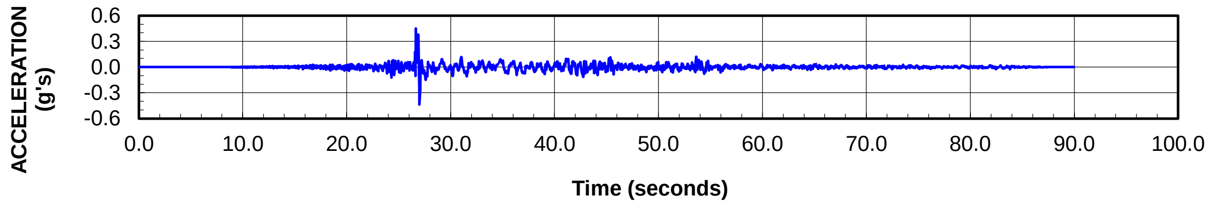


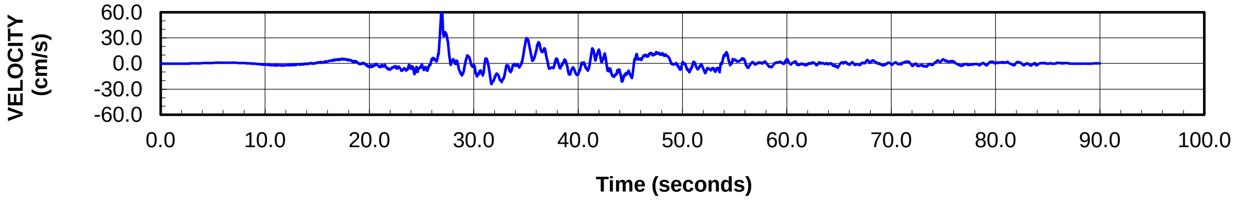


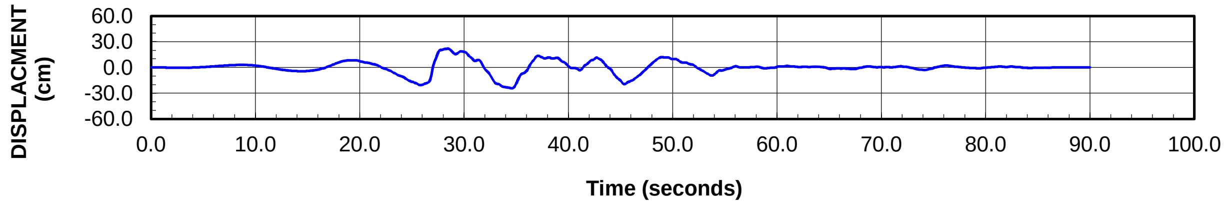


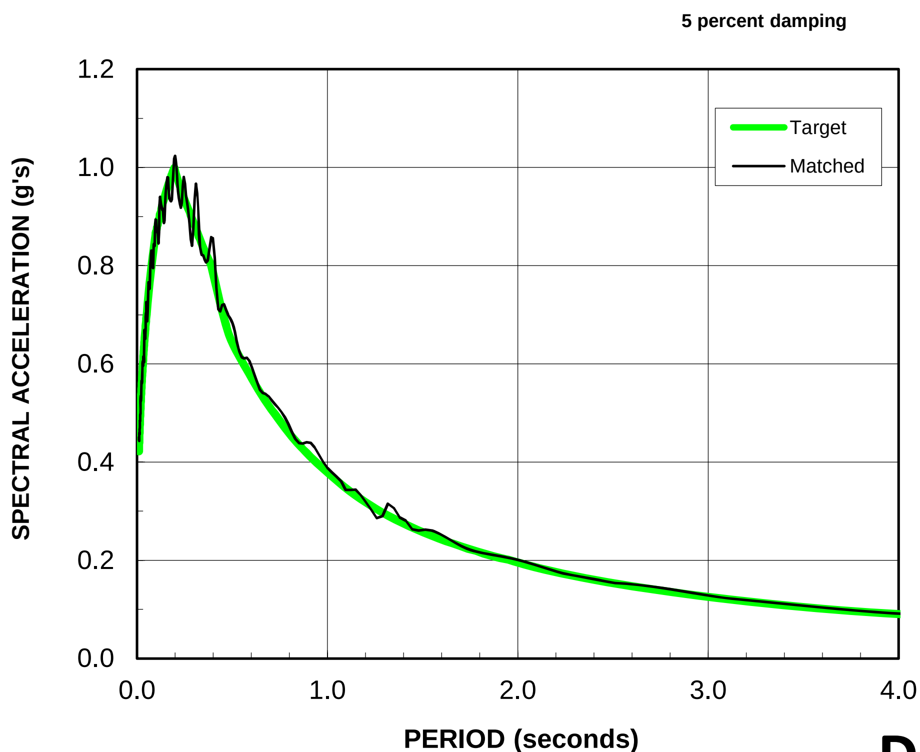
DRAFT

BLOCK 29-32 **MISSION BAY** San Francisco, California	**DE MATCHED TIME HISTORY AND RESPONSE SPECTRUM 1992 LANDERS EARTHQUAKE JOSHUA TREE 90 Degs.**
Treadwell&Rollo A LANGAN COMPANY	Date 11/23/11 | Project No. 750603902 | Figure E-12









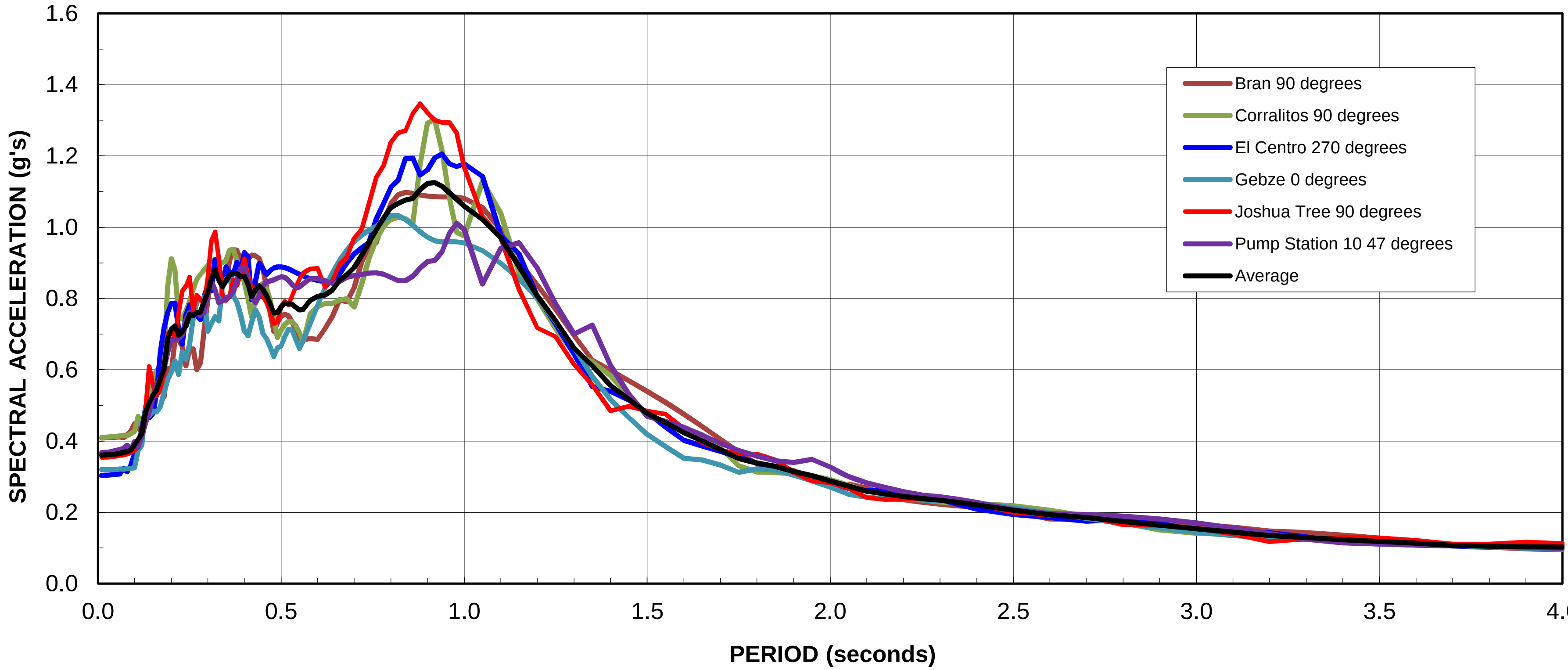
DRAFT

BLOCK 29-32
MISSION BAY
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DE MATCHED TIME HISTORY AND RESPONSE SPECTRUM 2002 DENALI EARTHQUAKE PS-10 47 Degs.

Date 11/23/11	Project No. 750603902	Figure E-13



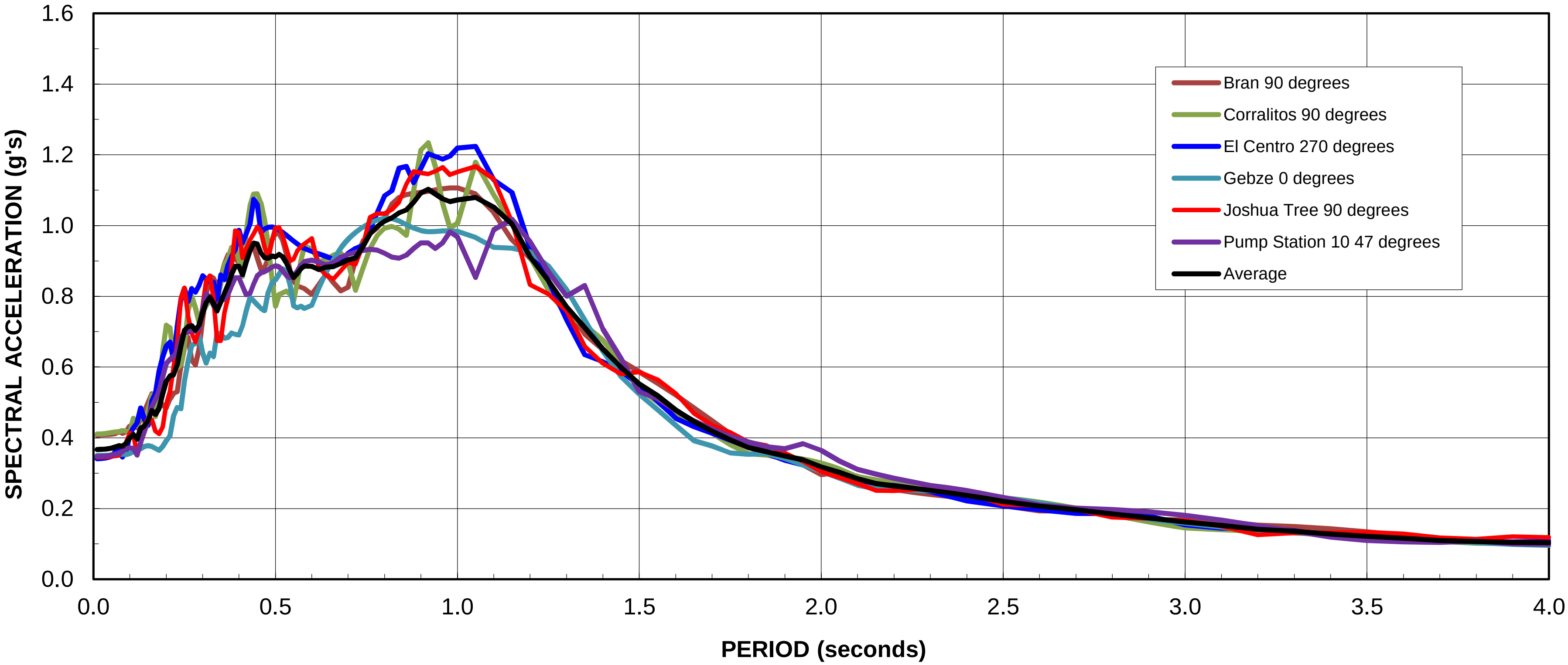
DRAFT

Damping Ratio = 5%

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake

BLOCKS 29-32 MISSION BAY San Francisco, California		
DE SHAKE RESULTS FOR SHALLOW PROFILE BLOCK 29		
Date 12/13/11	Project No. 750603902	Figure E-14

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

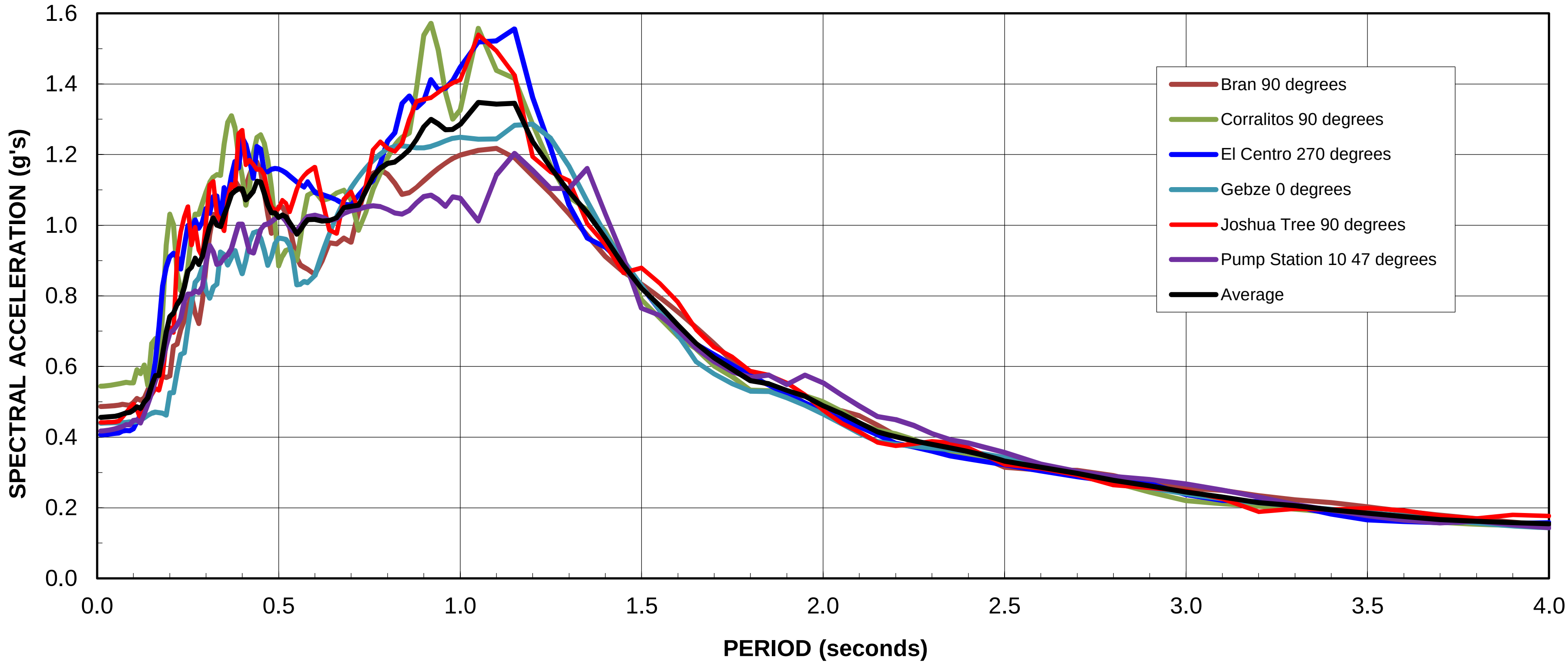
Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake

BLOCKS 29-32
MISSION BAY
San Francisco, California

DE SHAKE RESULTS FOR DEEP PROFILE
BLOCK 29

Date 12/13/11	Project No. 750603902	Figure E-15

Treadwell&Rollo
A LANGAN COMPANY



Damping Ratio = 5%

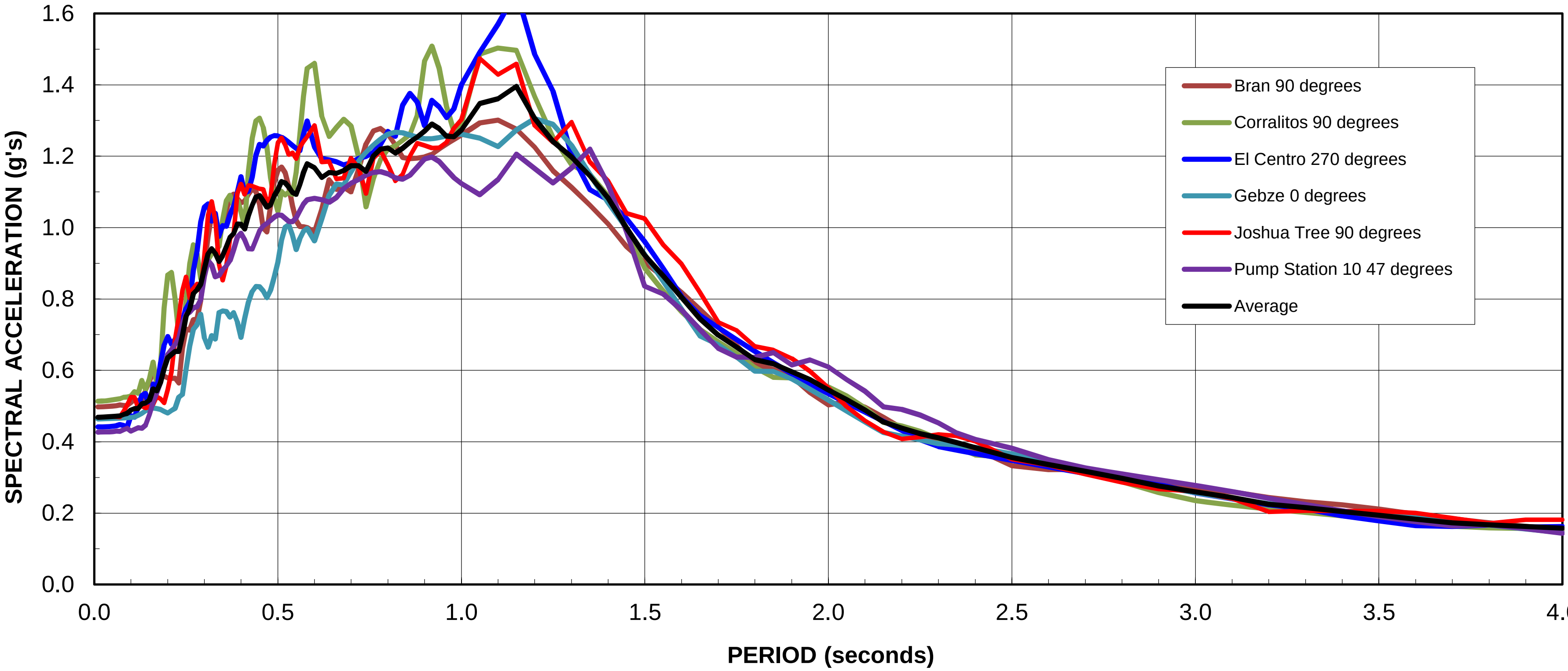
DRAFT

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) MCE denotes Maximum Considered Earthquake

BLOCKS 29-32 **MISSION BAY** San Francisco, California
MCE SHAKE RESULTS FOR SHALLOW PROFILE BLOCK 29

Date 12/13/11	Project No. 750603902	Figure E-16

Treadwell&Rollo
A LANGAN COMPANY



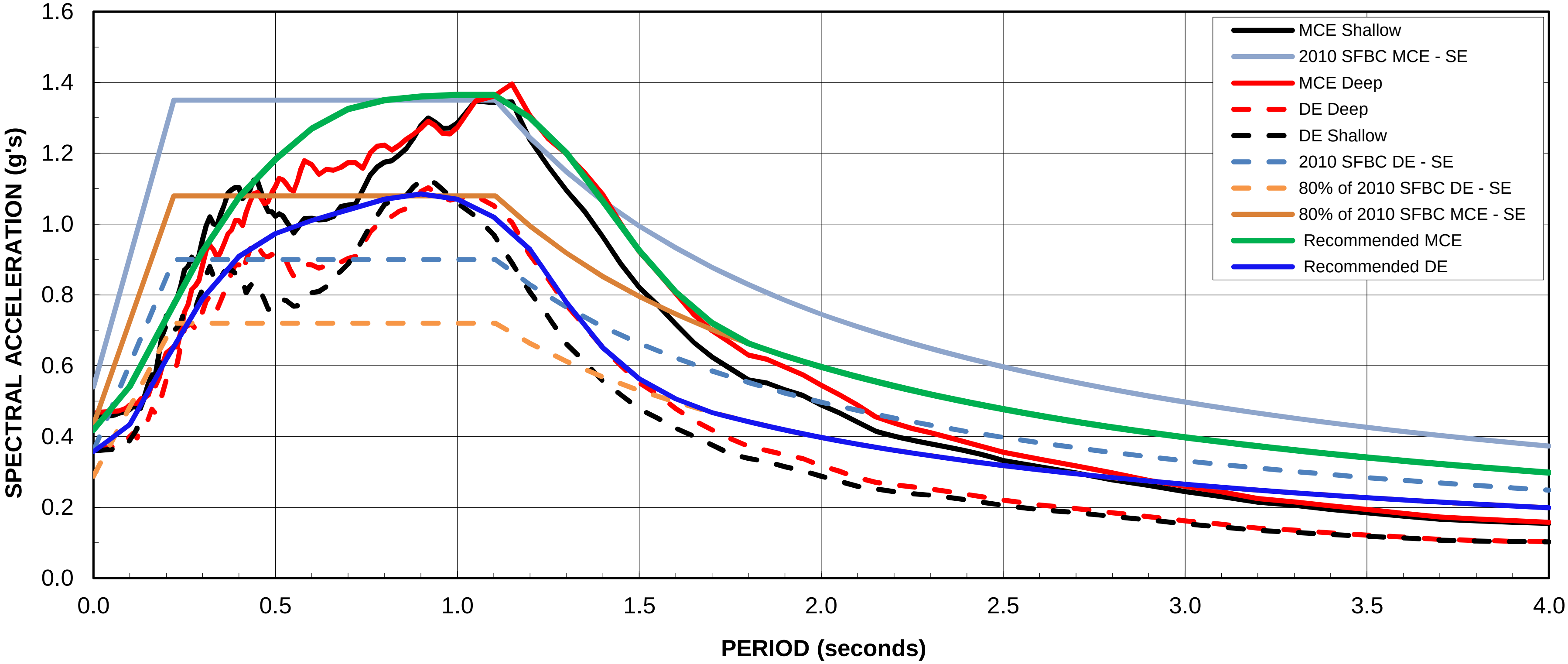
Damping Ratio = 5%

DRAFT

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) MCE denotes Maximum Considered Earthquake

BLOCKS 29-32 MISSION BAY San Francisco, California		
MCE SHAKE RESULTS FOR DEEP PROFILE BLOCK 29		
Date 12/13/11	Project No. 750603902	Figure E-17

Treadwell&Rollo
A LANGAN COMPANY



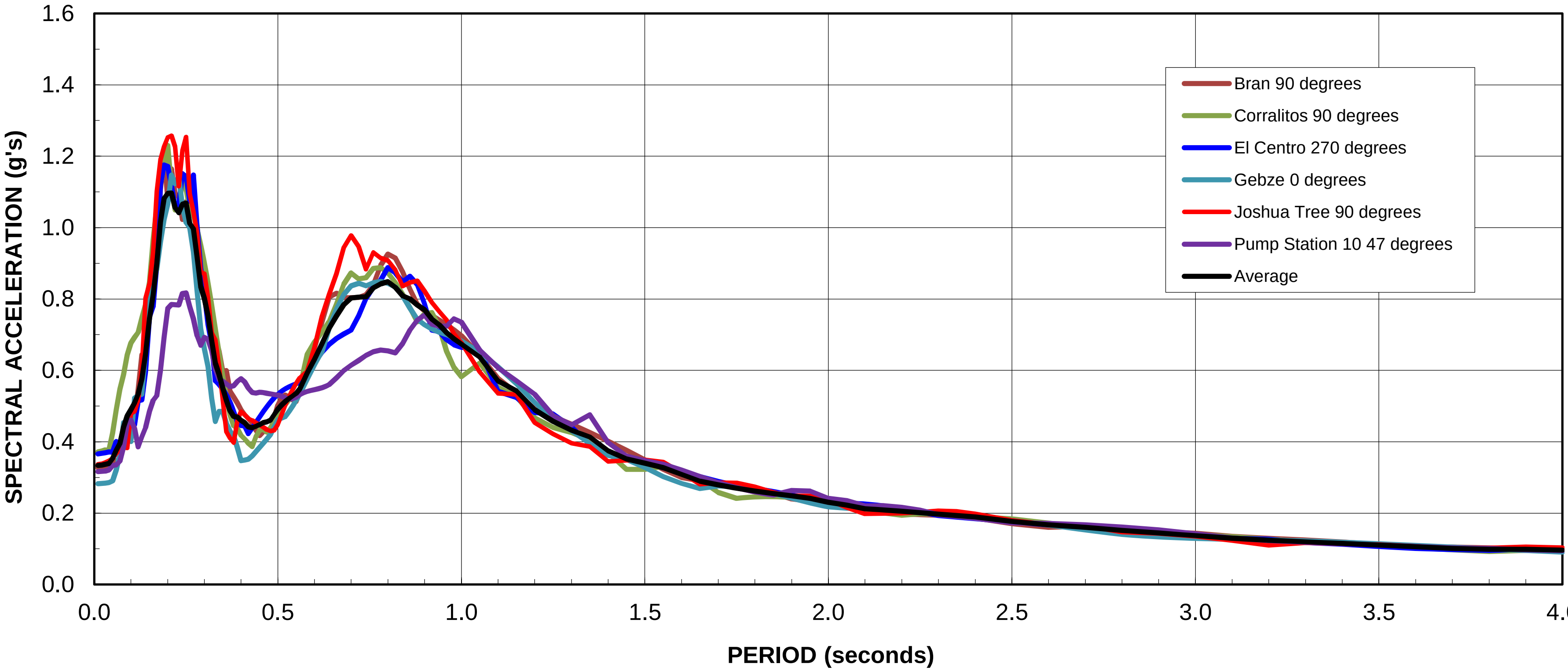
Damping Ratio = 5%

DRAFT

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake
(3) MCE denotes Maximum Considered Earthquake

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
AVERAGE SHAKE RESULTS BLOCK 29		
Date 12/13/11	Project No. 750603902	Figure E-18

Treadwell&Rollo
A LANGAN COMPANY



Damping Ratio = 5%

DRAFT

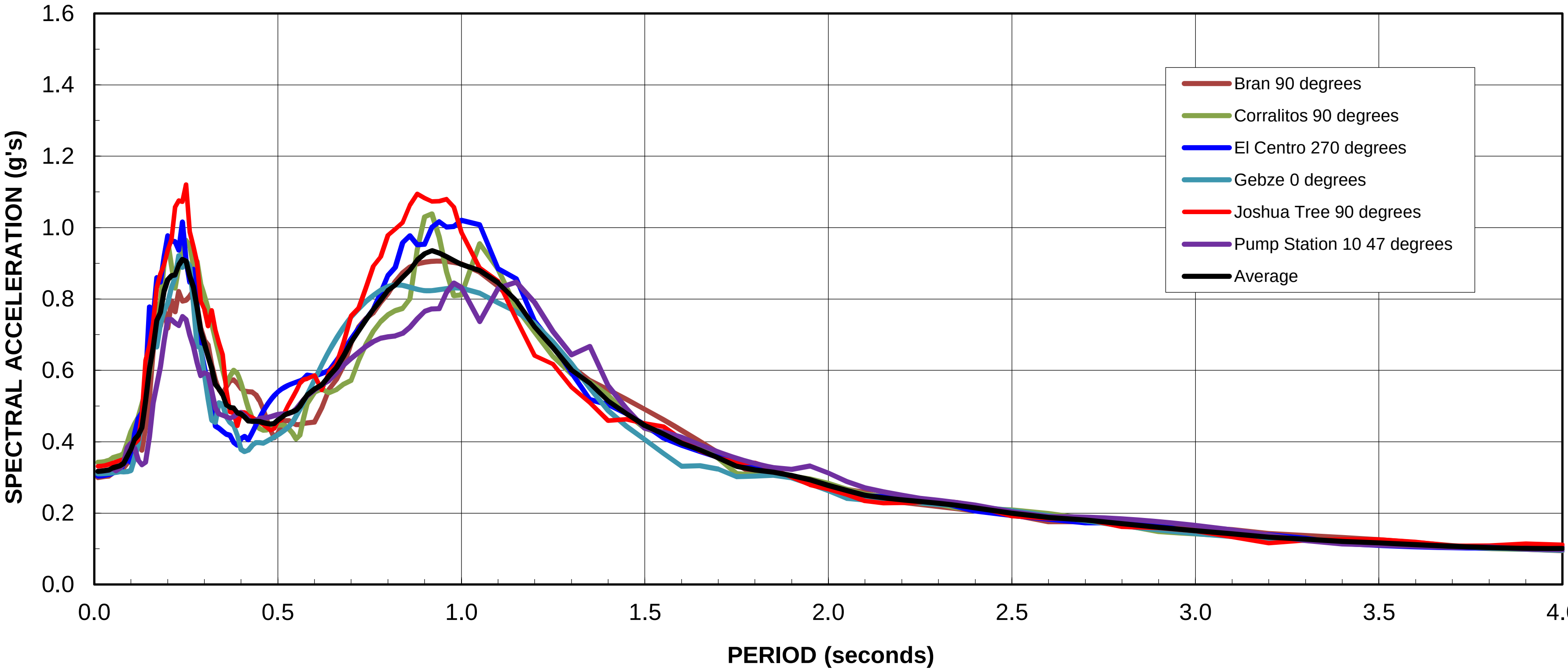
Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake
(3) The response spectra are calculated at proposed bottom of basement

BLOCKS 29-32
MISSION BAY
San Francisco, California

DE SHAKE RESULTS FOR SHALLOW PROFILE
BLOCK 30

Date 12/13/11	Project No. 750603902	Figure E-19

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

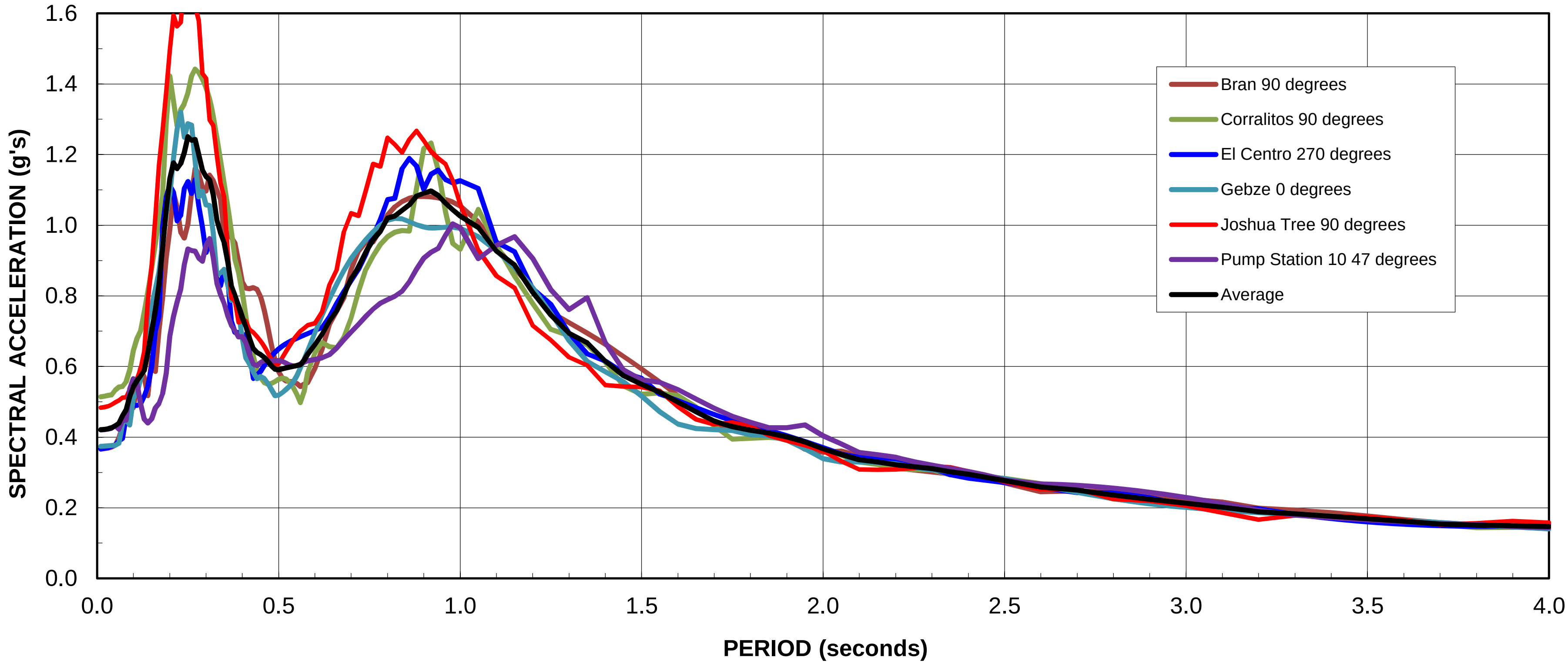
Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake
(3) The response spectra are calculated at proposed bottom of basement

BLOCKS 29-32
MISSION BAY
San Francisco, California

DE SHAKE RESULTS FOR DEEP PROFILE BLOCK 30

Date 12/13/11	Project No. 750603902	Figure E-20

Treadwell&Rollo
A LANGAN COMPANY



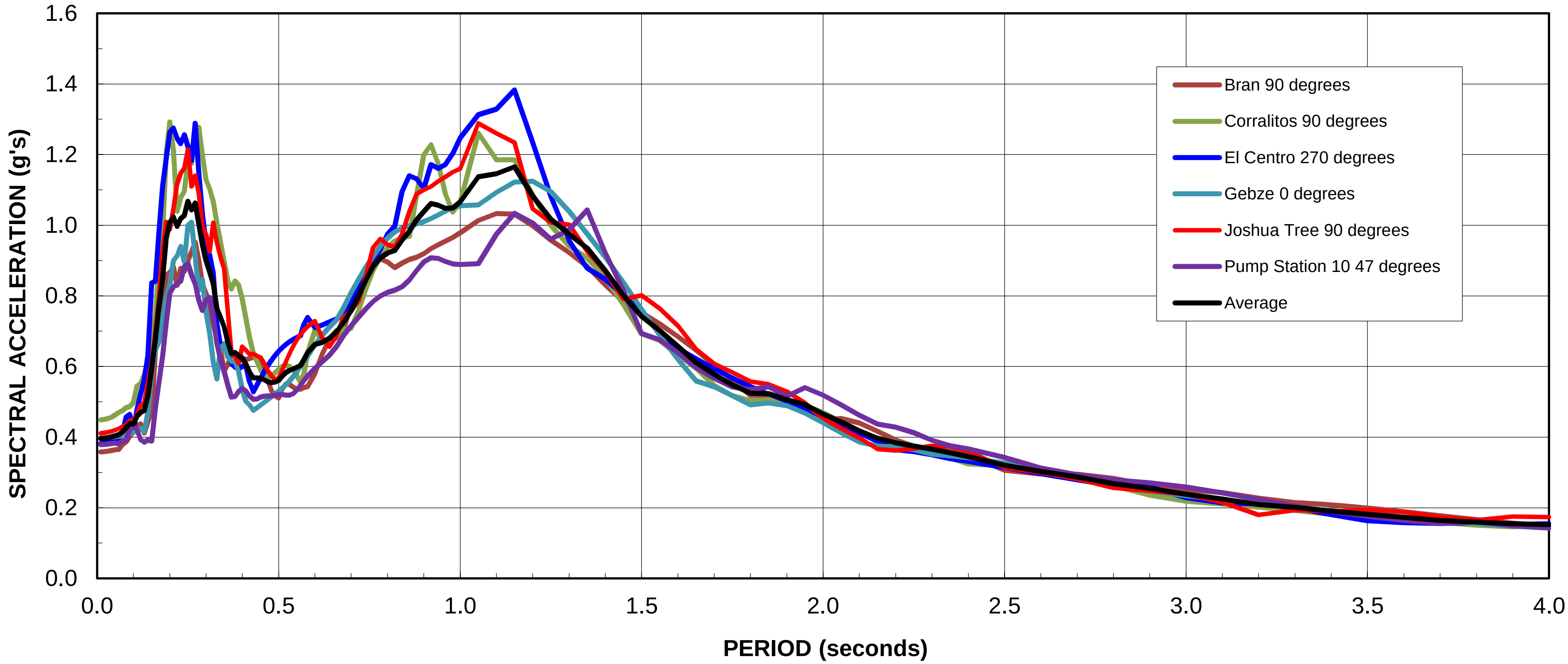
Damping Ratio = 5%

DRAFT

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) MCE denotes Maximum Considered Earthquake
(3) The response spectra are calculated at proposed bottom of basement

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
MCE SHAKE RESULTS FOR SHALLOW PROFILES BLOCK 30		
Date 12/13/11	Project No. 750603902	Figure E-21

Treadwell&Rollo
A LANGAN COMPANY

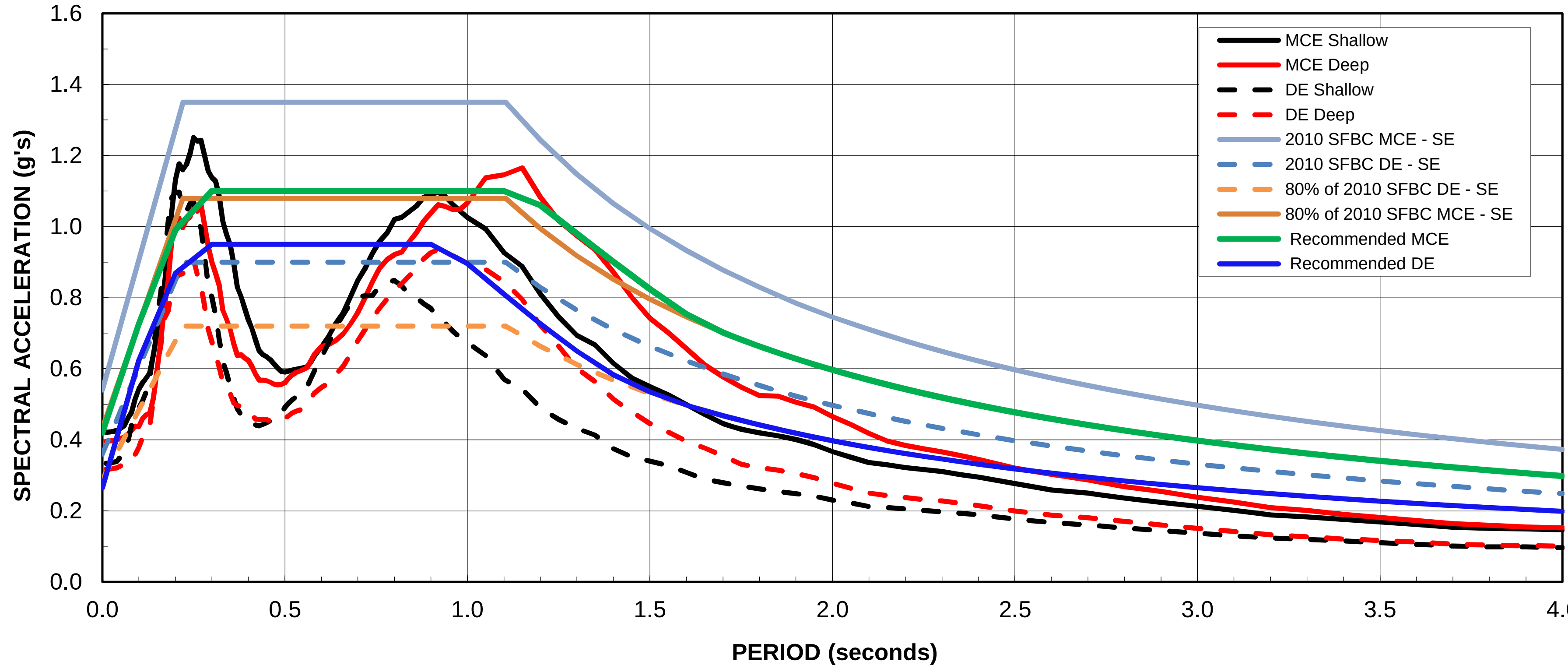


Damping Ratio = 5%

DRAFT

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) MCE denotes Maximum Considered Earthquake
(3) The response spectra are calculated at proposed bottom of basement

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
MCE SHAKE RESULTS FOR DEEP PROFILE **BLOCK 30**		
Date 12/13/11	Project No. 750603902	Figure E-22
Treadwell&Rollo A LANGAN COMPANY		



Damping Ratio = 5%

DRAFT

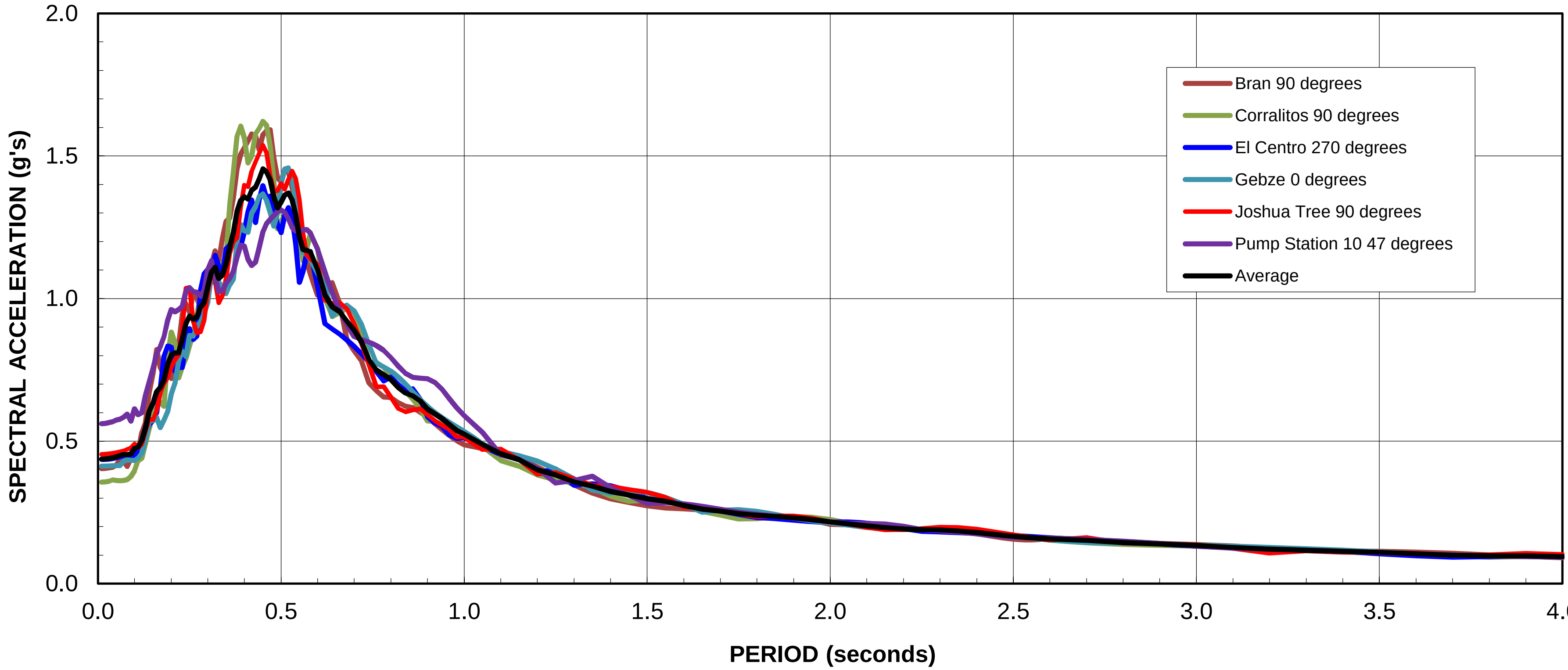
Notes: (1) Assumes upper 15 ft. of potentially liquefiacle fill has been mitigated.
(2) MCE and DE denote Maximum Considered Earthquake and Design Earthquake, respectively
(3) The response spectra are calculated at proposed bottom of basement

BLOCKS 29-32
MISSION BAY
San Francisco, California

AVERAGE SHAKE RESULTS AT BASEMENT LEVEL BLOCK 30

Date 12/13/11	Project No. 750603902	Figure E-23

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

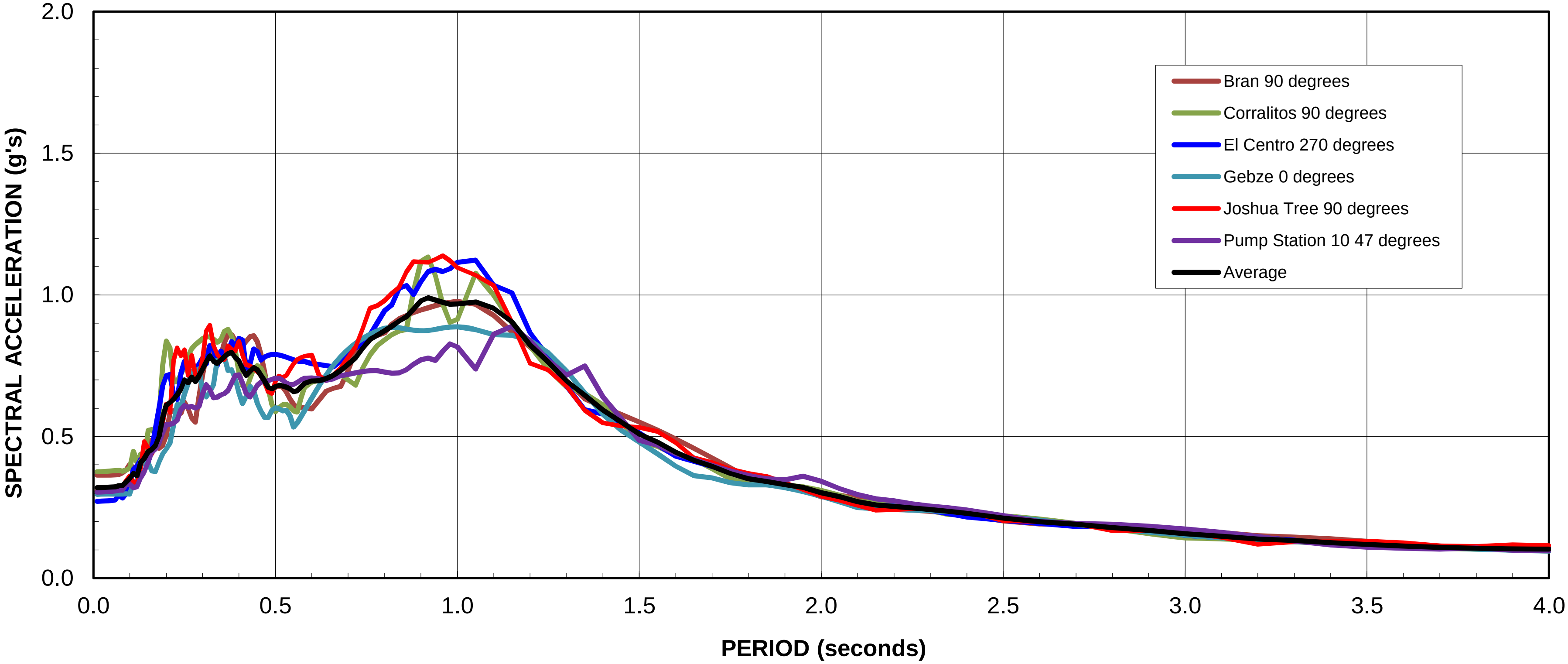
Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake

BLOCKS 29-32
MISSION BAY
San Francisco, California

DE SHAKE RESULTS FOR SHALLOW PROFILE
BLOCK 31

Date 12/13/11	Project No. 750603902	Figure E-24

Treadwell&Rollo
A LANGAN COMPANY



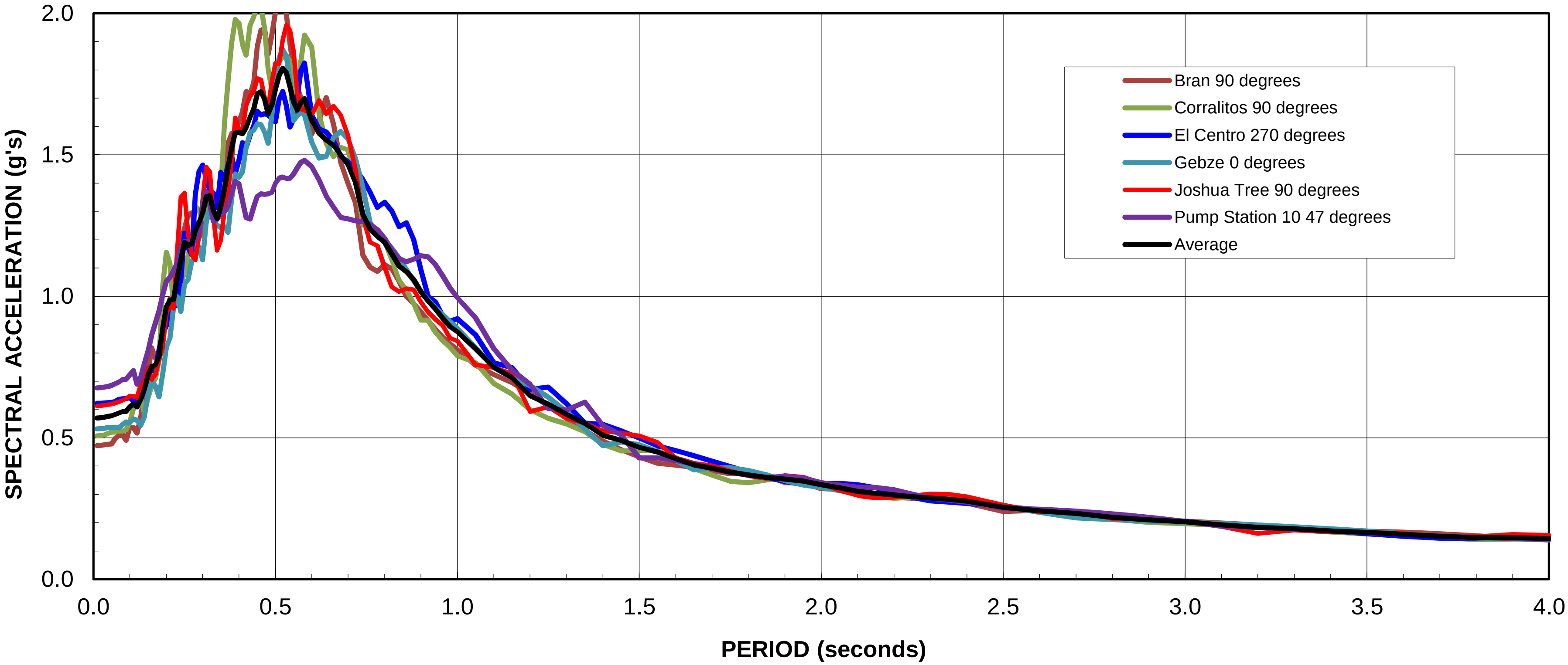
DRAFT

Damping Ratio = 5%

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
DE SHAKE RESULTS FOR DEEP PROFILE **BLOCK 31**		
Date 12/13/11	Project No. 750603902	Figure E-25

Treadwell&Rollo
A LANGAN COMPANY

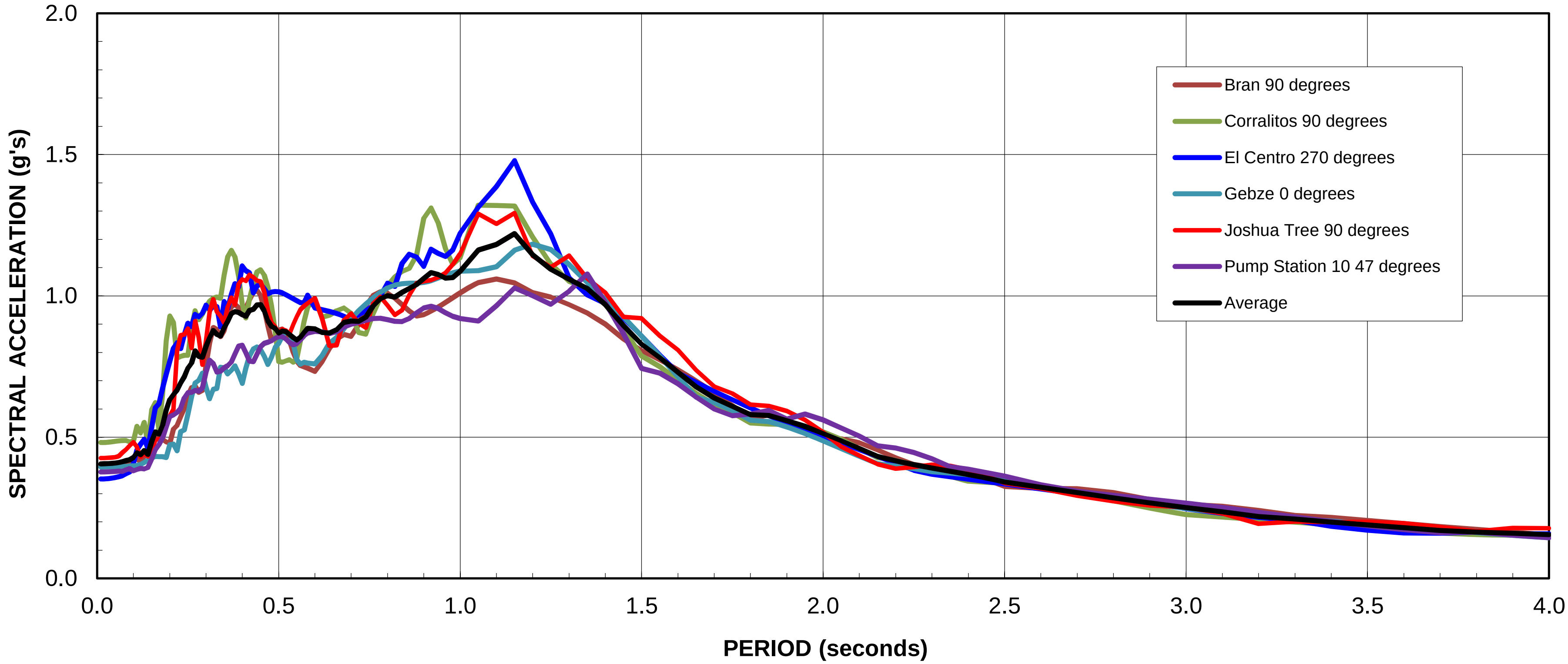


DRAFT

Damping Ratio = 5%

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) MCE denotes Maximum Considered Earthquake

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
MCE SHAKE RESULTS FOR SHALLOW PROFILE BLOCK 31		
Date 12/13/11	Project No. 750603902	Figure E-26
Treadwell&Rollo A LANGAN COMPANY		



DRAFT

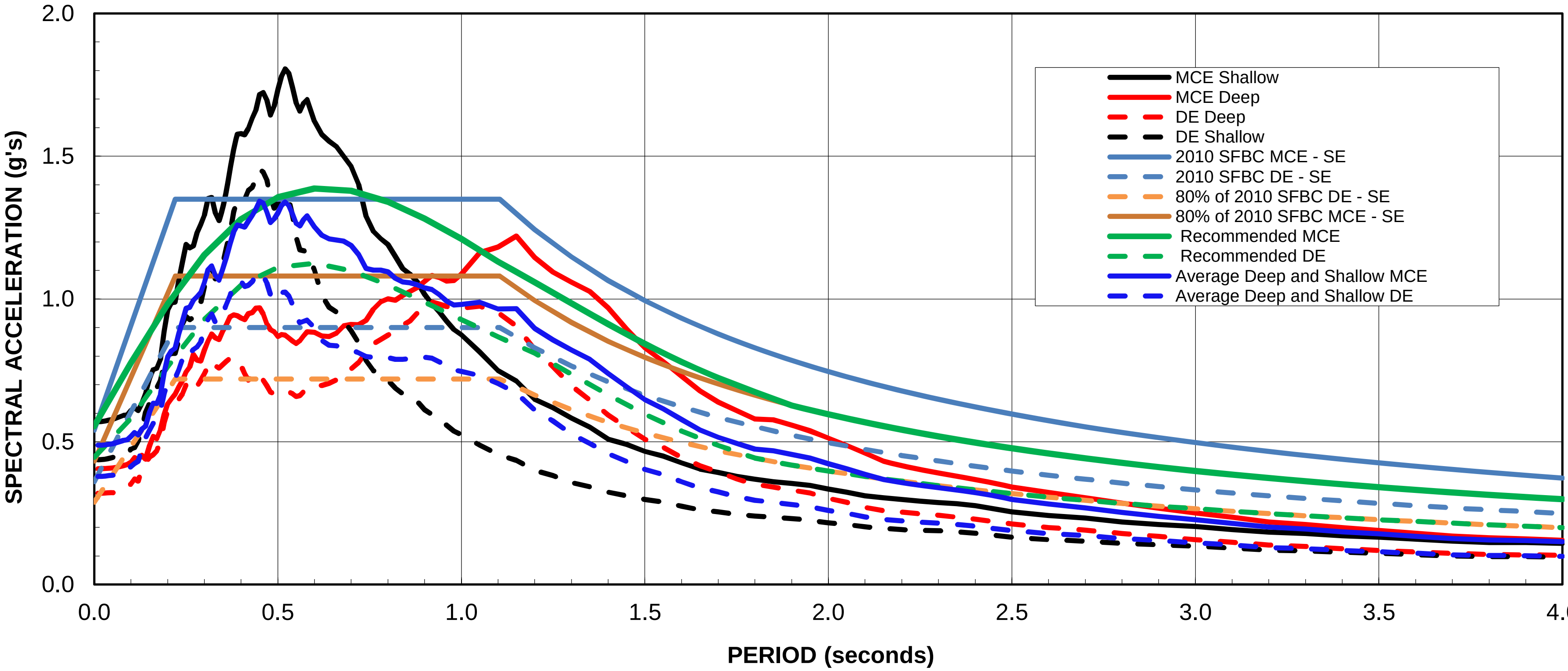
Damping Ratio = 5%

Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) MCE denotes Maximum Considered Earthquake

BLOCKS 29-32 **MISSION BAY** San Francisco, California
MCE SHAKE RESULTS FOR DEEP PROFILE **BLOCK 31**

Date 12/13/11	Project No. 750603902	Figure E-27

Treadwell&Rollo
A LANGAN COMPANY



Damping Ratio = 5%

DRAFT

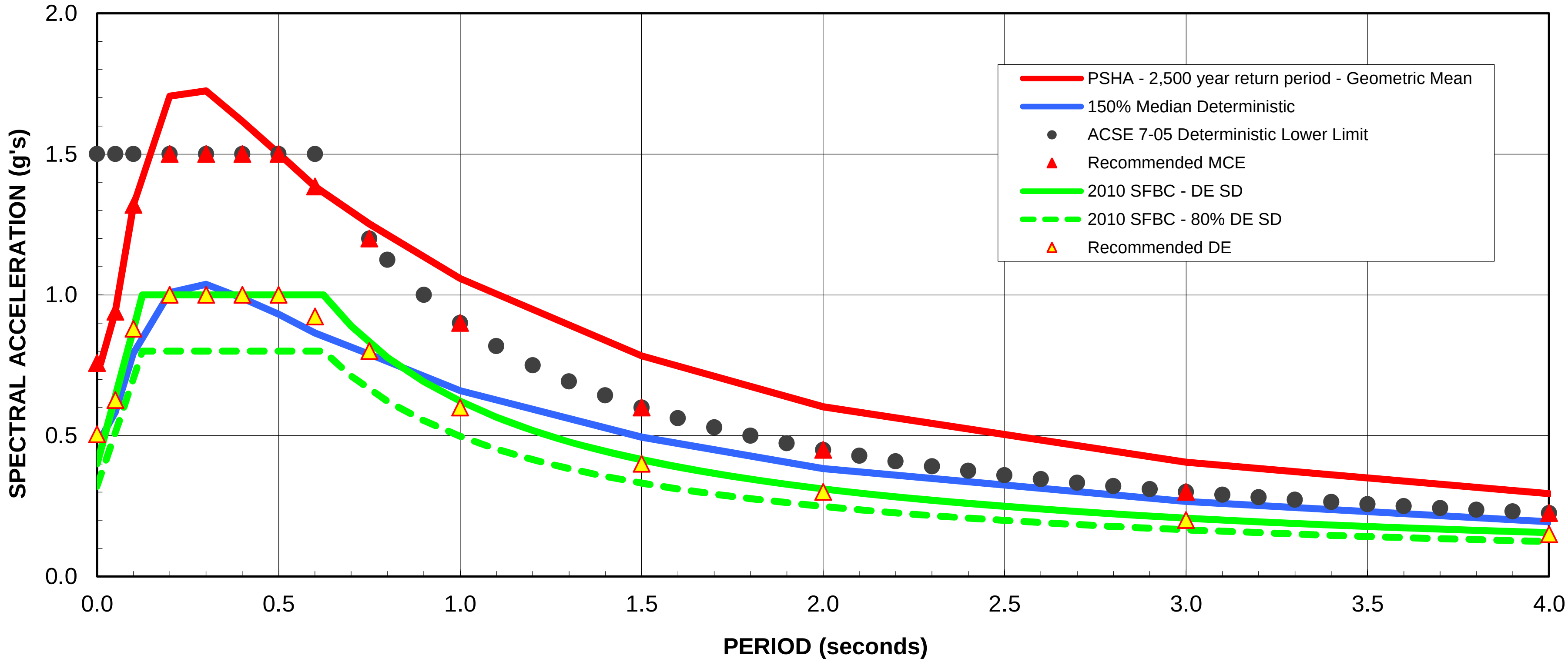
Notes: (1) Assumes upper 15 ft. of potentially liquefiable fill has been mitigated.
(2) DE denotes Design Earthquake
(3) MCE denotes Maximum Considered Earthquake

BLOCKS 29-32
MISSION BAY
San Francisco, California

AVERAGE SHAKE RESULTS BLOCK 31

Date 12/13/11	Project No. 750603902	Figure E-28

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

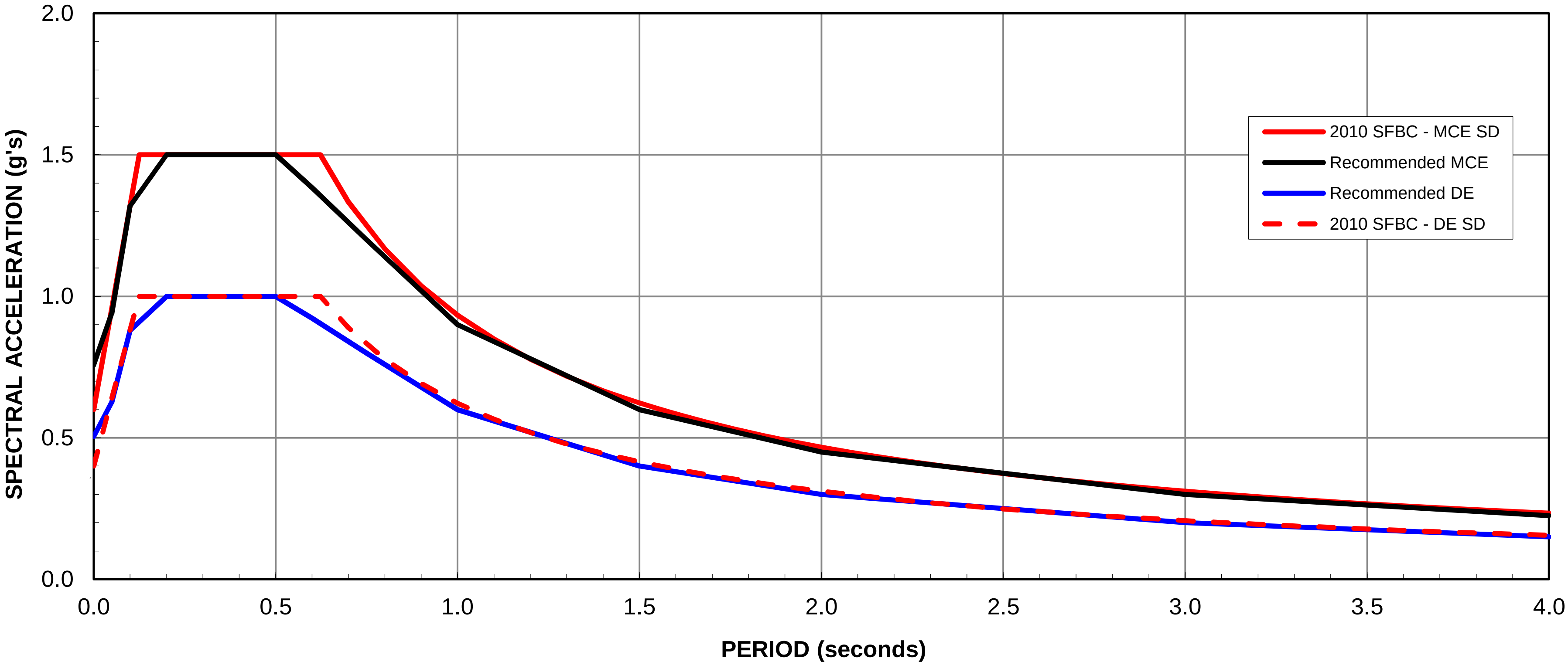
Note: M_w = 8.0, Dist. 12.7 km, estimated Average V_{S30} = 340 m/s

BLOCKS 29-32
MISSION BAY
San Francisco, California

COMPARISON OF DETERMINISTIC, PROBABILISTIC AND CODE SPECTRA FOR BLOCK 32

Date 12/13/11	Project No. 750603902	Figure E-29

Treadwell&Rollo
A LANGAN COMPANY



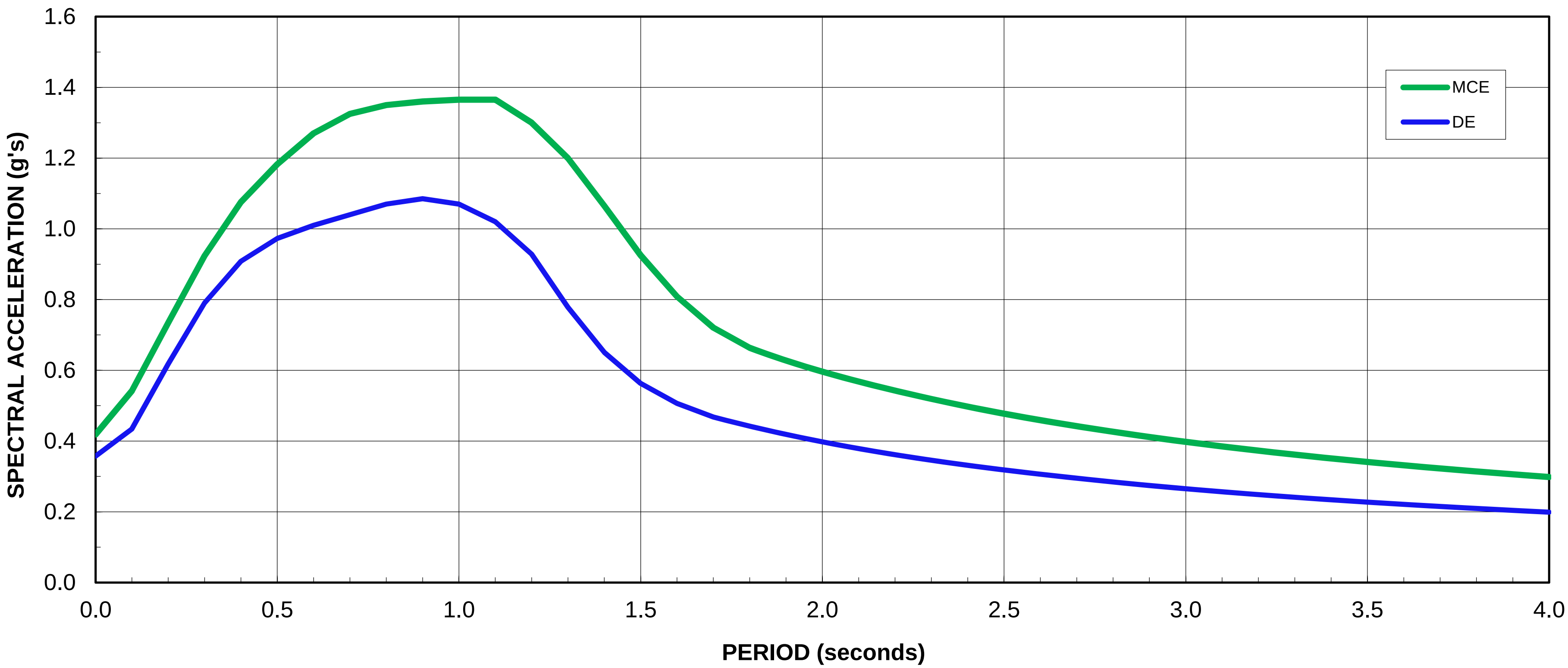
DRAFT

Damping Ratio = 5%

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively.

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
COMPARISON OF RECOMMENDED **SPECTRA WITH CODE** **BLOCK 32**		
Date 12/13/11	Project No. 750603902	Figure E-30

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

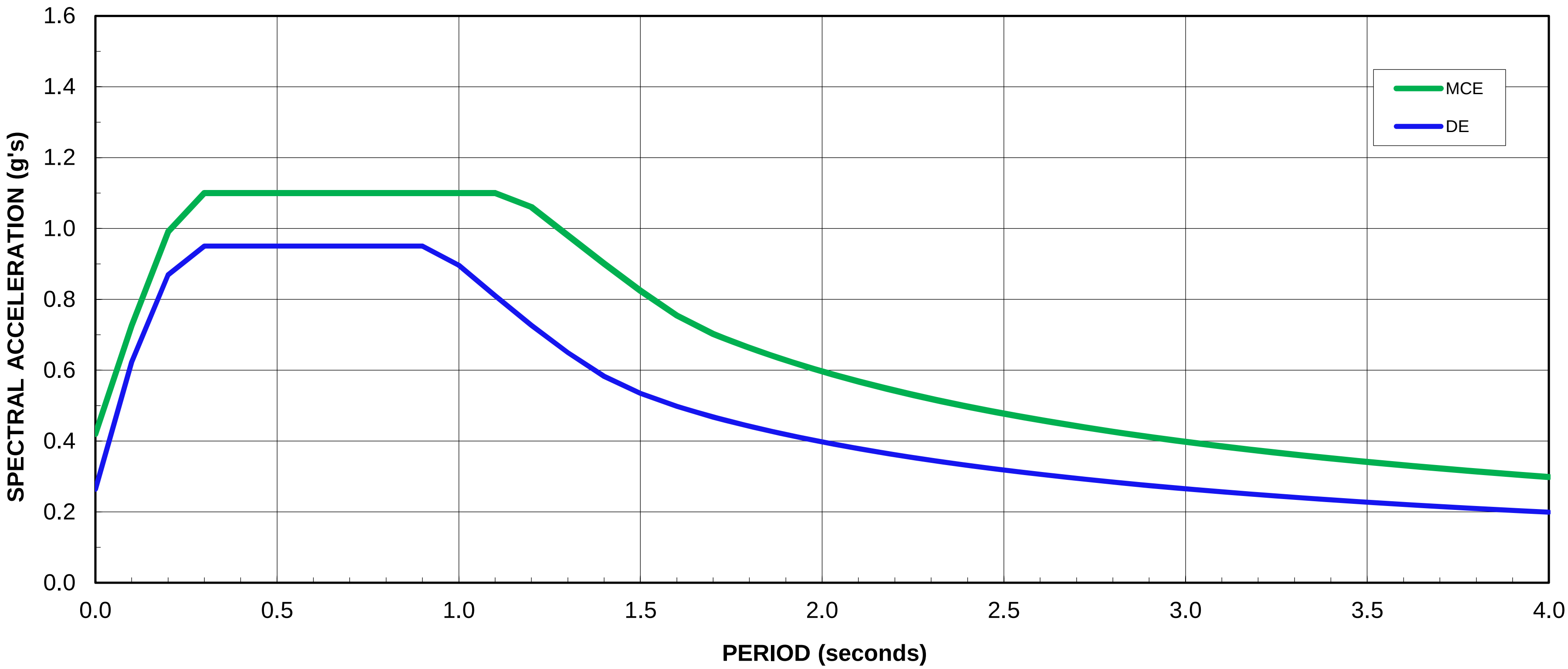
Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29 THROUGH 32
MISSION BAY
San Francisco, California

RECOMMENDED SPECTRA
BLOCK 29

Date 12/13/11	Project No. 750603902	Figure E-31

Treadwell&Rollo
A LANGAN COMPANY



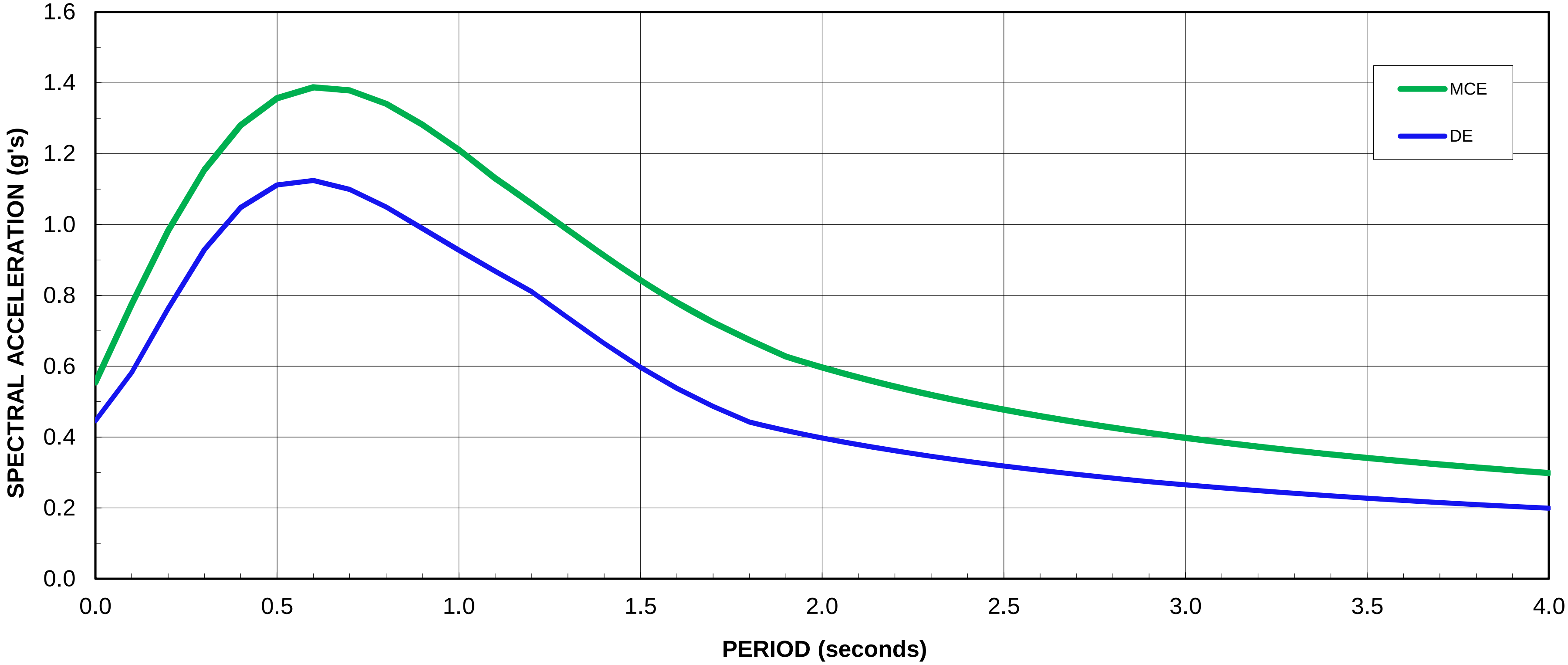
Damping Ratio = 5%

DRAFT

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29 THROUGH 32 **MISSION BAY** San Francisco, California		
RECOMMENDED SPECTRA **BLOCK 30**		
Date 12/13/11	Project No. 750603902	Figure E-32

Treadwell&Rollo
A LANGAN COMPANY



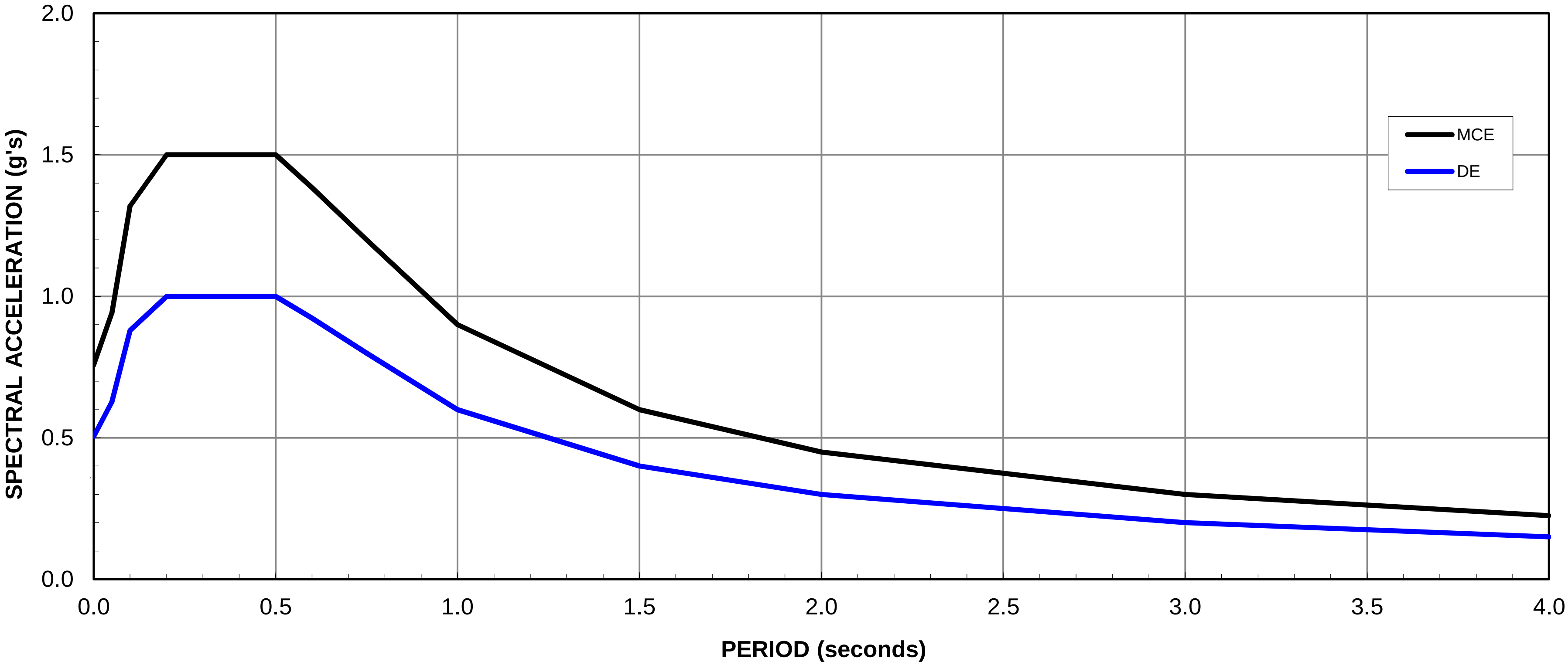
DRAFT

Damping Ratio = 5%

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29 THROUGH 32 **MISSION BAY** San Francisco, California		
RECOMMENDED SPECTRA **BLOCK 31**		
Date 12/13/11	Project No. 750603902	Figure E-33

Treadwell&Rollo
A LANGAN COMPANY



DRAFT

Damping Ratio = 5%

Note: DE and MCE denote Design Earthquake and Maximum Considered Earthquake, respectively

BLOCKS 29-32 **MISSION BAY** San Francisco, California		
RECOMMENDED SPECTRA **BLOCK 32**		
Date 12/13/11	Project No. 750603902	Figure E-34

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