APPENDIX D:
GEOTECHNICAL DATA

TRANSIT-ORIENTED DEVELOPMENT #1 PROJECT

June 21, 2013
BAGG Job No. VAMMS-01-00

VAM Millbrae Serra
1818 Gilbreath Avenue
Burlingame, CA 94010

Attention: Sal Ariganello

DRAFT REPORT
Geotechnical Engineering Investigation
Proposed Development
Millbrae Station West Side Properties
Millbrae, California

Dear Mr. Arianello:

Transmitted herewith is our preliminary geotechnical engineering investigation report for the proposed development in Millbrae, California. The report presents a discussion of the existing subsurface conditions and potential geologic/geotechnical hazards that could affect the project site, and our preliminary conclusions and recommendations for site grading, drainage, and foundations support for the proposed development.

Thank you for the opportunity to be of service on this project. Please do not hesitate to contact us should you have any questions or comments.

Very truly yours,

BAGG Engineers

REGISTERED PROFESSIONAL ENGINEER
JASON VAN ZWOL
No.854
Exp
GEOTECHNICAL
STATE OF CALIFORNIA

Jason Van Zwol
Geotechnical Engineer

Distribution: Electronic & 4 hard copies

DRAFT REPORT
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DRAFT REPORT

GEOTECHNICAL ENGINEERING INVESTIGATION
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MILLBRAE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical engineering investigation performed to address the proposed development in Millbrae, California. The attached Plate 1, Vicinity Map, shows the general location of the site, which is located a short distance north of the intersection of Millbrae Avenue and El Camino Real, and Plate 2, Site Plan, shows the existing site topography as well as the approximate location of the exploratory borings we drilled at the site as part of this study. This study was conducted in accordance with the scope of services outlined in our Proposal No. 13-190 dated March 19, 2013.

The following sections of this report present the result of our research, field and laboratory investigations, and geotechnical recommendations for design and construction of the proposed project, which is still in the early stages of design.

2.0 PROJECT DESCRIPTION

The exact nature of the proposed project is not known at this time; however, it is anticipated that the development will include at least one level of below-grade parking and at least 3 above-grade floors with an unknown combination of commercial, office, and/or residential space. The type of building frame is also unknown, but it is anticipated the below-grade portion of the building will be reinforced concrete. It is also possible the overall development will be constructed in phases.

We anticipate that grading for the project will most likely be limited to excavation for the below-grade parking, which is expected to extend to the property lines, as do other developments in the area. Because the building will be very near the property lines, some form of shoring will be required to protect the adjacent properties.

3.0 SITE DESCRIPTION

The subject development will be located on a group of parcels that have been combined into an "L" shaped property, with the toe of the "L" fronting on El Camino Real. The north end of the parcel is bordered by commercial and residential properties, while the back (eastern) side of the property is bordered by CalTrain and BART tracks and the Millbrae train station. Linden and Serra Avenues border the property on the southern and western sides, respectively.

Two commercial buildings are on the property bordering El Camino, including a now-vacant lumber yard that used to occupy the northeastern portion of the site. The central portion of the site is occupied by a former convalescent home, which is also vacant, while the southern end of the site is covered with an empty gravel parking area. The old lumber yard is paved with asphalt, while the area around the convalescent home contains a few trees, grass, and planter areas.

The ground surface on the site generally slopes toward the east, with a low area near the north-central portion of the eastern property line. Site elevations vary from about 25 feet in the extreme southwest corner and 20 feet at El Camino, to a low of about 12 feet along the northeastern property line. A historic creek, which is now confined to a culvert traversing the north east corner of the property, was located in the low area.

4.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our investigation was to obtain geotechnical information regarding soil and groundwater conditions at the site in order to develop geotechnical recommendations for preliminary design of the proposed structure(s). The required information was obtained from seven borings advanced to depths varying from 35 to 65 feet. Based on our understanding of the site conditions and project requirements, our report presents conclusions, opinions, and recommendations regarding:

- potential for liquefaction beneath the site, including potential for liquefaction-related settlements and lateral spreading of the northern portions of the site.

- site seismicity, including appropriate soil profile type and parameters for seismic design per the 2010 California Building Code (CBC),

- specific soil and groundwater conditions discovered by our borings, such as loose, soft, compressible, saturated, expansive, or collapsible soils, that may require special mitigation or impose restrictions on the project, including the thickness and consistency of fill soils encountered at the site, and their ability to support foundations,

- criteria for site grading, including placement of engineered fills, backfills in utility trenches, and preparation of subgrades for slabs and pavements,

- alternative foundation types for multi-story structures, including allowable vertical and lateral resistance values under both static and seismic conditions,

- total and differential settlements expected for additional fill soils placed in various portions of the site, and bearing values required to limit settlements to about one inch with shallow foundations,

- criteria for design of below-grade basement walls, including cantilevered and restrained walls under both static and seismic conditions,

- general provisions for surface and subsurface drainage on the site.

Information required to fulfill the above purposes was obtained from seven exploratory borings drilled at the approximate locations shown on the attached Site Plan. A laboratory testing program was then performed on soil samples obtained from the borings in order to evaluate the engineering characteristics of the soils at the site. Information obtained from these tasks was used to develop conclusions, opinions, and recommendations oriented toward the above-stated purpose of our services. Accordingly, the scope of our services consisted of the following specific tasks:

1. Research and review pertinent geotechnical and geological maps and reports relevant to the site and vicinity.

2. Mark the borings at the site at least 72 hours in advance of the drilling, and notify Underground Service Alert to mark utility lines on or entering the site. We also obtained a permit from the San Mateo County Groundwater Protection Program.

3. Drill, log, and sample a total of seven exploratory borings to depths varying from 35 to 65 feet with a truck-mounted drilling rig using 8-inch diameter hollow-stem augers. The borings were drilled under the technical direction of one of our engineers, who also obtained disturbed bulk, Standard Penetration Test, and/or relatively undisturbed ring samples of the native soils for visual classification and laboratory testing. We also measured the depth to groundwater, as encountered within the borings. At the completion of drilling and sampling, the borings were sealed with cement grout per standard protocol, and the drill cuttings were left on site.

4. Perform a laboratory testing program on the collected soil samples to evaluate the engineering characteristics of the subsurface soils. Tests included shear strength testing, Atterberg Limits tests, consolidation tests, sieve analyses, and moisture-density measurements, as judged appropriate.

5. Based on information obtained from the above tasks, we performed engineering analyses oriented toward the above-described purpose of the investigation.

6. Prepare four paper copies and one electronic pdf copy of a report summarizing our findings and including a site plan showing the approximate location of the exploratory borings, the logs of the borings, the results of the laboratory testing, and our conclusions, opinions, and recommendations for design and construction of the project.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

Subsurface conditions at the site were explored by drilling a total of seven soil borings to depths ranging from 35 to 65 feet at the approximate locations shown on the attached Plate 2, Site Plan. The borings were advanced with a truck-mounted drilling rig using 8-inch O.D. hollow stem augers, and were technically directed by one of our engineers who maintained a continuous log of the subsurface conditions encountered in each borehole. Disturbed bulk, relatively undisturbed ring, and Standard Penetration Test samples of the site materials were obtained for visual examination and laboratory testing.

The subsurface materials were visually classified in the field; the classifications were then checked by visual examination of samples in the laboratory. In addition to sample classification, the boring logs contain interpretation of where stratum changes or gradational changes occur between samples. The boring logs depict BAGG's interpretations of subsurface conditions only at the locations indicated on Plate 2, Site Plan, and only on the dates noted on the logs. The boring logs are intended for use only in conjunction with this report, and only for the purpose outlined by this report.

The graphical representation of the materials encountered in the borings and the results of our laboratory tests, as well as explanatory/illustrative data are attached as follows:

- Plate 5, Unified Soil Classification System; illustrates the general features of the soil classification system used on the boring logs.
- Plate 6, Soil Terminology; lists and describes the soil engineering terms used on the boring logs.
- Plate 7, Rock Terminology: lists the terms used to describe the native bedrock materials on the boring logs.
- Plate 8-A and 8-B, Key to Symbols; describe various symbols used on the boring logs and describes general and specific conditions that apply to the boring logs.
- Plates 9-A through 15-C, Boring Logs; describe the subsurface materials encountered, show the depths and blow counts for the samples collected from the soils/bedrock materials, and summarize results of the strength tests and moisture-density data.
- Plates 16 and 17, Plasticity Data; present the results of several Atterberg Limits tests performed on selected samples of the near-surface soils to classify the materials and obtain an indication of their expansion potential and susceptibility to liquefaction.
- Plates 18 and 19, Consolidation Test Data, summarizes the results of consolidation tests performed on two samples of the softer soils at the site, to aid in estimating potential foundation settlements.

Strength tests, consisting of direct shear tests were performed on samples of the soils to evaluate the strength parameters of the site materials. The shear tests were performed at both natural (field) and artificially increased moisture contents and under various surcharge pressures. The moisture content and dry density of the undisturbed samples were also measured to aid in correlating their engineering properties. The results of the laboratory tests are presented on the borings and the plates described above.

6.0 GEOLOGY AND SEISMICITY

6.1 Regional Geology

The site and the San Francisco Bay Area lie within the Coast Ranges geomorphic province, a series of discontinuous northwest-trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting along the contact between the Pacific and North American tectonic plates. Due to the collision of these two tectonic plates, the site and Bay Area are a seismically active region with several major fault lines extending through the area. Four of the major, northwest-trending earthquake faults within the area include the San Andreas and San Gregorio faults to the west-southwest, and the Hayward and Calaveras faults to the east-northeast.

6.2 Site Geology and Geologic Setting

The site area is located where the marshlands at the edge of the Bay meet the gentle foothills along the base of the San Francisco Peninsula. The area to the northeast of the site, as well as the extreme northern corner of the site, has been mapped (Pampeyan, 1994) as artificial fill. Adjacent to the Bay marshlands, this fill is usually placed over soft Bay Mud deposits. The map shows a thin strip of coarse-grained alluvial soils between the fill and the Colma formation underlying the majority of the southern portion of the site. These areas are illustrated on Plate 3, Regional Geology Map. The three geologic units are described by Pampeyan as follows:

> **Qf** **Artificial Fill (Historic)** – Poorly consolidated to well consolidated gravel, sand, silt, and rock fragments in various combinations used in a variety of applications including riprap, highway-, railroad-, and airport runway-fills, earthfill dams, reservoirs embankments, and building-site grades. Thickness and consolidation dependent upon type of application and site.
>
> **Qac** **Coarse-grained Alluvium (Holocene)** – Unconsolidated, moderately sorted sand and gravel forming stream levees, fans, and flood plains in and close to upland areas. Grades coarser headward. Locally contains lenticular interlayes of well-sorted silt, sand, and gravel; locally contains modern vertebrate and invertebrate fossils. Interfingers with medium-grained alluvium and colluvial deposits. Maximum thickness probably less than 75 ft.
>
> **Qc** **Colma Formation (Pleistocene)** – Yellowish-gray and gray to yellowish-orange and red-brown, friable to loose, fine- to medium grained arkosic sand with subordinate amounts of gravel, silt, and clay.

The referenced geology map also shows the Merced formation, a sandstone formation slightly older than the Colma formation, and sheared rocks of the Franciscan Complex exposed farther uphill toward the southwest. The Franciscan bedrock is presumed to underlie the site area at depth.

Borings B-6 and B-7 each encountered soft Bay Mud soils as predicted by the geology map. Beneath the Bay Mud and surficial fill soils all borings encountered the Colma formation. In addition, Borings B-1 and B-3, located on the southeastern side of the site, encountered what appeared to be the Franciscan Complex consisting of harder sandstone and shale materials at depths of 29 and 39 feet, respectively.

6.3 Seismic Setting

The San Francisco Peninsula is within a seismically active region of faulting associated with the San Andreas fault and other related faults that are at the contact between the Pacific and North American tectonic plates. The faulting in this zone extends eastward from just off the Pacific Coast through the San Francisco Bay Area to the western side of the Great Valley. This region has one of the highest rates of seismic moment release per square mile of any urban area in the United States. It is emerging from the stress shadow of the 1906 San Francisco Earthquake and future large earthquakes are considered a certainty.

While the subject property is not within an Alquist-Priolo Earthquake Fault Zone designated by the California Geological Survey (CGS, 2000) around active fault traces, the San Andreas fault, located about 2.8 km west southwest, is considered to be the principal seismic hazard in this area because of its activity rate and proximity to the site. The Working Group on California Earthquake Probabilities (2008) has estimated that the probability for a major earthquake (M_W 6.7 or greater) within 30 years on the nearby San Andreas fault is about 21 percent. They also estimate the San Andreas may be capable of generating an earthquake as large as magnitude 8.0.

The Hayward fault, located roughly 27 km east northeast of the site on the opposite side of San Francisco Bay, has been estimated to have a probability of 31 percent for generating a major earthquake within 30 years; however, the distance to the Hayward fault and expected smaller earthquake magnitude considerably reduces the risk to this site.

The most significant, active faults, their distance from the site, and characteristic earthquake magnitudes are listed in the following table. Other significant regional faults are of greater distance, and/or have lesser probabilities of a major earthquake occurring in the next 30 years or so. The major active faults with respect to the subject site are also depicted on Plate 4, Regional Fault Map.

Table 1
Significant Earthquake Scenarios

Fault	Approximate Distance from Site (kilometers)[1]	Direction from Site	Potential Moment Magnitude (M_W)[2]
San Andreas (Entire)	2.8	W	7.9-8.0
San Andreas (Peninsula)	2.8	W	7.1-7.2
San Gregorio	14	W	7.4-7.5
Hayward – Rogers Creek	27	E	7.2-7.3
Calaveras	41	E	6.8-7.0

[1]USGS Fault files w/ Google Earth
[2]Working Group on California Earthquake Probabilities, 2008.

7.0 GEOTECHNICAL SITE CONDITIONS

Four borings were advanced along the lower, eastern side of the property, and three were located near the center and/or western side of the site. Borings B-2, B-4, B-5, and B-7 were also drilled through surface pavements which generally consisted of about 3 inches of AC over 4 inches of aggregate base, except that Boring B-7 only had 2 inches AC over 3 inches of base and B-4 encountered 3 inches AC over about 33 inches of aggregate fill material, which could be utility trench backfill.

7.1 Subsurface Conditions

The site elevations have been altered by past grading activities on the site to create the existing site configuration. Beneath the fill soils encountered in 6 of the 7 borings, native soils consisted of alluvial soils in most of the site area, with soft Bay Mud Soils existing in the northeast quadrant of the site.

7.1.1 Fill Soils

In the southeastern portion of the site, Borings B-1 and B-3 each encountered about 3 feet of fill materials, while Boring B-2, located at the higher elevation did not encounter any fill. In the northern portion of the site, the four borings all encountered between 6½ and 7½ feet of fill soils.

The fill soils were very similar to the sandy Colma formation beneath the site, expect they were generally much looser than the Colma sands. The fills generally consisted of brown, medium dense silty sand. Besides the 33 inches of aggregate base in B-4, the exceptions were a poorly graded sand at the base of the fill in B-6, and a gravelly sand in B-7.

7.1.2 Native Soils

Beneath the pavements and fill soils, our borings encountered from as little as 3 feet native soils in the southeast corner of the site to as much as 11 feet in the northeast corner. With the exception of the soft

Bay Mud soils in the northeast quadrant of the site, the native soils were mostly sandy clay to clayey sand and were generally stiff and medium dense. The Bay Mud was relatively sandy in nature, but was still quite soft and compressible.

The native alluvial soils, not including the soft Bay Mud, would provide fair to good support for lightly loaded structures, but not multi-story buildings. On a preliminary basis, we also have estimated that a strip footing loaded at 2,000 psf would settle on the order of 2 inches at the location of Boring B-7.

7.1.3 Colma formation

The Colma formation consisted predominantly of a dense, fine silty sand that varied from gray to yellow brown. The formation also included some significant zones of relatively clean sand, clayey sand, and even some sandy lean clay. As all sandy soils were dense to very dense and the clays were very stiff, the Colma formation will provide very good foundation support for shallow foundations.

7.1.4 Franciscan formation

As indicated Borings B-1 and b-3 encountered what appeared to be Franciscan sandstone and shale. At 29 feet, B-1 encountered about 4½ feet of highly weathered and sheared shale over a hard, moderately weathered sandstone. At 39 feet in Boring B-3, we encountered about 10½ feet of broken and crushed, weathered shale to the depth explored.

7.2 Groundwater

The groundwater levels encountered in our borings was inconsistent. Borings B-1 and B-5 reached depths of 35 and 60 feet, respectively, and water was not noted in the bore hole during drilling. (The drill holes were not allowed to stand open for a time to stabilize.) Water was noted in the other borings during the drilling operations at depths varying from 7½ feet to 27½ feet below the ground surface, or at elevations varying from roughly +10 feet in B-2 to -14 feet in B-4. In both Borings B-6 and B-7, the groundwater was encountered at an elevation of+4½ feet, while in B-3 it was measured at the end of drilling at -10 feet.

It should be noted that none of the borings were allowed to stand open for a period of time to allow the groundwater to stabilize within the bore hole. The measured levels therefore may not represent the true groundwater table. It is also possible that the observed levels represent random zones of seepage within the underlying Colma formation; however, the consistent water level observed within the Bay Mud area (B-6 and B-7) is what we would expect for this area. In the northeastern quadrant of the site, we would therefore anticipate a relatively consistent water table at an elevation no lower than +4½ to 5 feet.

For these reasons, it would be prudent to assume a groundwater table at elevation 5 feet during design of the structure, and provide drainage for basement walls above this elevation and appropriate water proofing measures below.

8.0 GEOHAZARD ANALYSIS

8.1 CBC Seismic Parameters

A seismic hazard analysis was performed for the subject site using the USGS "Seismic Hazard Curves, Response Parameters and Design parameters", (v5.1.0, 2011). The "mapped" values generally represent firm bedrock shaking with a 2 percent probability of being exceeded in a 50-year period. The values are then modified for a given site based on a broad classification of the soil profile at the site.

Based on the blow counts measured in our borings, we have estimated an average N value, as defined in Section 1613.5.5 of the 2010 CBC, to be on the order of 35 to 40 blows per foot. This gives a site classification of "D," defined as a "dense soil" profile with an average shear wave velocity in the range of 600 to 1,200 feet per second, average Standard Penetration Test (N) values between 15 and 50, and average undrained shear strength between 1,000 psf and 2,000 psf within the top 100 feet of the soil profile.

Using the site coordinates of 37.6007° North Latitude and 122.3885° West Longitude, and the USGS Seismic Hazards Curves, Response Parameters, and Design Parameters V5.1.0 (2011), earthquake ground motion parameters were computed in accordance with 2010 California Building Code are as listed in the following table.

Table 2
Parameters for Seismic Design

2010 CBC Site Parameter	Value
Site Latitude	37.6007°
Site Longitude	122.3885° W
Site Class, Table 1613.5.2	Class D, Stiff Soil
Mapped Spectral Acceleration for Short Periods S_s	2.10g
Mapped Spectral Acceleration for 1-second Period S_1	1.15g
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
Site-Modified Spectral Acceleration for short Periods S_{MS}	2.10g
Site-Modified Spectral Acceleration for 1-second Period S_{M1}	1.73g
Design Spectral Acceleration for short Periods S_{DS}	1.40g
Design Spectral Acceleration for 1-second Periods S_{D1}	1.15g

8.2 Liquefaction Potential

Soil liquefaction is a condition where saturated granular soils near the ground surface undergo a substantial loss of strength due to increased pore water pressure resulting from cyclic stress applications induced by earthquakes or other vibrations. In the process, the soil acquires mobility sufficient to permit both vertical and horizontal movements, if not confined. Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands, and loose silts with very low cohesion. In general, liquefaction hazards are most severe in the upper 50 feet of the soil profile. In the deeper deposits, the greater overburden soils tend to isolate the ground surface from the impact of any liquefaction in deeper soils, and the overburden pressure tends to limit shear strains that occur during liquefaction.

We evaluated the liquefaction potential of the sandy soils beneath the site in accordance with the methods described by Idriss and Boulanger (2008), Seed, et al (2003), and Cetin, et al (2009). The analyses were performed using a Peak Ground Acceleration (PGA) of 0.59g with a magnitude 8 earthquake, with were obtained from the USGS deaggregation web site and represent a 10 percent probability of being exceeded in 50 years, in accordance with CGS Special Publication 117A (2008). We also used a groundwater table at elevation +5 feet, unless the water was encountered at a higher elevation in the boring, in which case we used the water level in that boring. The three methods each predict differing levels of liquefaction and resulting settlement. The variation and the average values are tabulated in the following table.

Table 3
Estimated Liquefaction Settlements (Inches)

Boring	Range	Average
B-1	0.01 – 0.01	0.01
B-2	0.07 – 0.62	0.42
B-3	0.09 – 1.05	0.59
B-4	0.04 – 0.05	0.04
B-5	0.48 – 0.90	0.65
B-6	0.04 – 0.04	0.04
B-7	0.02 – 0.43	0.25

Based on these estimates, we anticipate there will be less than about 1 inch of settlement due to liquefaction at the site, with a differential amounting to less than one inch in 100 feet. It can be noted that the small settlements predicted for Borings B-1, B-4, and B-6 are from seismic consolidation of sandy soils above the water table, which will likely be removed by the planned basement excavation.

9.0 DISCUSSION AND RECOMMENDATIONS

9.1 General

Based on our analyses of the information obtained from our borings and laboratory testing program, it is our opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this report are incorporated into the project design and construction. When the project plans and specifications become available, they should be reviewed by this office to verify that the intent of our recommendations have been properly interpreted and incorporated into them, as well as to verify that our recommendations are appropriate for the project in its final form.

The most significant constraints on the proposed project are the loose and soft overburden soils covering the site, especially the soft Bay Mud soils in the northeastern quadrant of the site, and the variable liquefaction settlement that can occur at the site. The extent of the soft Bay Mud soils is not accurately known at this time, but has been estimated from our borings and published geology maps. To verify that the depth of 17 feet to firm soils as encountered in Borings B-6 and B-7 is not exceeded in other areas of the site, it may be prudent to advance several CPT soundings on the site, which would provide many more data points for the elevation of firm supporting soils beneath the site, as well as verify the lateral extent of the Bay Mud soils. Plates 20 and 21, Generalized Cross Sections, show the distribution of fill, soils, and the top of the firm Colma formation as interpolated between the seven borings on the site. The plates do not, however, distinguish between the soft Bay Mud soils and the native alluvial (or colluvial) soils, as we do not have sufficient data to do so at this time.

While details regarding foundation loads for the proposed development are not available at this time, it is anticipated that structures in excess of two or three stories would have to be founded on the firm Colma formation, either with drilled piers or spread footings. Even two-story buildings would likely experience excessive differential settlements if founded above the existing Bay Mud soils in the northeast quadrant of the site.

Most of the soils blanketing the site are low plasticity silty sands; however, some of the clayey soils encountered were found to be moderately expansive in nature. Therefore, some precautions will be needed in site preparation and selective soils should be used within slab subgrade areas.

The site could experience very strong ground shaking from future earthquakes during the anticipated lifetime of the project. The intensity of the ground shaking will depend on the magnitude of the earthquake, distance to the epicenter, and the response characteristics of the on-site soils. Our analyses has found that a PGA of about 0.6g has a 10% chance of being exceeded within the next 50 years and a PGA of 1.1g has a 2% probability of being exceeded in the next 50 years. While it is not possible to totally preclude damage to structures during major earthquakes, strict adherence to good engineering design and construction practices will help reduce the risk of damage. The 2010 California Building Code defines the minimum standards of good engineering practice.

9.2 Site Grading

While details of the development are not known at this time, it is anticipated that site grading will consist of little more than removing the existing pavements, curb and gutter and planter areas, removing old foundations, clearing and grubbing, excavation for the new basement/parking level, and possible reworking the upper portion of the on-site fill soils in some areas to support new pavements and sidewalks.

As used in this report, the term “compact” and its derivatives mean that all fill soils should be compacted to a minimum of 95 percent of the maximum dry density, as determined by the ASTM Test Method D1557, within the top 12 inches of pavement subgrades, and to 90 percent elsewhere, while at a moisture content that is slightly over optimum. Where encountered, the on-site clayey soils should be compacted while at a moisture content that is at least 1 to 2 percent over optimum as determined by the same test method.

The following grading procedures should be followed for preparation of areas to receive fill, pavements, concrete slabs, or flatwork:

- Strip and remove the asphaltic concrete (AC) and concrete pavement, curbs, and the underlying aggregate base (AB) and stockpile separately. Specifications for pulverizing the AC are outlined below. The AB material beneath the pavement may be used as Class II aggregate subbase or as general site fills.

- Strip and remove all bushes, vegetation, roots, and organically contaminated topsoil, abandoned underground utilities, and other debris from the existing planter areas. All organically-contaminated soils must be removed from the site and cannot be used as site fill. Where trees are to be removed, the removal should include all major root systems down to 1 inch in size.

- Any existing fill soils that do not meet the above compaction requirements should be excavated to their full depth and replaced as properly engineered fill.

- Scarify the resulting subgrades within any area to receive fill to a depth of 8 inches. Thoroughly moisture condition the scarified surfaces to the proper moisture content and re-compact as specified above. Further over-excavate as necessary any area still containing weak and/or yielding (pumping) soils, as determined in the field by the Geotechnical Engineer.

- Place fill on the over-excavated surfaces and in the holes/depressions created by the above actions in uniformly moisture conditioned and compacted lifts not exceeding 8 inches in loose thickness. Rocks or cobbles larger than 4 inches in maximum dimensions should not be allowed to remain within the foundation areas, unless they can be crushed in-place by the construction equipment.

The excavated soil materials form the site are suitable for use as structural fill, except the on-site clayey soils should not be placed directly below slabs-on-grade. Imported fill soils if needed, should be predominantly granular in nature and should be free of organics, debris, or rocks over 4 inches in size, and should be approved by the Geotechnical Engineer before importing to the site. As a general guide to acceptance, imported soils should have a Plasticity Index less than 15, and R-value of at least 20, and fines content between 15 and 65 percent. All aspects of site grading including clearing/stripping, demolition, placement of fills or backfills, and preparation of subgrades should be performed under the observation of BAGG's field representatives.

The existing pavement to be removed, as well as the concrete resulting from the demolition of the existing curbs and foundations, may be recycled if desired. Such materials should be pulverized in a manner that 100 percent of the particles will be less than 2 inches in size, and 90 percent finer than 1 inch, and reused as recycled Class 2 aggregate subbase (minimum R-value=50), or recycled Class 2 aggregate base (minimum R-value=78) pending confirmation R-value testing during construction.

It must be the Contractor's responsibility to select equipment and procedures that will accomplish the grading as described above. The Contractor must also organize his work in such a manner that one of our field representatives can observe and test the grading operations, including clearing, excavation, compaction of fill and backfill, and compaction of subgrades.

9.2.1 Temporary Shoring

We anticipate the excavation for the basement will require shoring to support the adjacent properties. Temporary shoring may consist of soldier-pile and wood lagging walls with tie-backs or other approved alternative. The temporary shoring should be designed to withstand an active earth pressure of 40 pcf above the water table, and 85 pcf below the water table. Where a sloping surface will exist above the temporary shoring, these pressures should be increased by 3 pcf for every 5 degree increase in the slope angle. Surcharge loads will be in addition to the active earth pressures and should be added at a rate of 1/3 the applied vertical load. Additionally, construction equipment should not be allowed at the top of the excavation closer than a distance equal to one-half of the height of the excavation unless the shoring is designed to support an appropriate surcharge.

Passive soil pressures on soldier piles below the base of the excavation should be taken as an equivalent fluid pressure of 165 pcf acting over 1½ pier diameters. This is the *net* passive pressure, assuming a potential groundwater table near the base of the excavation, i.e., buoyant soil weights.

Where a *temporary* sloped excavation is desired, it may be opened at a gradient of 2:1 (horizontal to vertical) where the excavation exposes soft Bay Mud soils and 1½:1 (H to V) if the excavation contains granular materials.

9.2.2 Construction Dewatering

Based on the free water encountered in our borings, it appears the basement excavation will penetrate below a possibly localized groundwater table. (Our analyses and recommendations, however, have assumed the groundwater table is everywhere present.) Our investigation has not evaluated the permeability of the on-site soils, nor estimated the flow rate at which it will be necessary to pump water from the excavation. It may be necessary for this estimation to be based on pump tests performed in the vicinity of Borings B-6 and B-7. In addition, if the shoring and excavation system is designed to completely dewater the excavation area, an evaluation of the effects of lowering the water table should be made for the adjacent properties, especially the residences to the north and the rail tracks to the east.

It may also be possible to use a relatively impermeable shoring system to minimize pumping. In this case, it will be necessary to evaluate the shoring penetration needed to prevent piping and sand boils in the base of the excavation. Some of the information required for these evaluations can be obtained from pore pressure measurements taken during CPTs advanced to further define the limits of the Bay Mud and depth to the Colma formation.

9.3 Building Foundations

In order to limit differential settlements, the proposed multi-story structure should be supported on foundations that are established within the underlying Colma formation. The top of this formation is illustrated on Plates 20 and 21 of this report. If basement excavations remove all upper overburden soils, the building can be supported on conventional shallow foundations, otherwise the building will have to be supported on drilled piers founded within the Colma formation. Recommendations for these foundation types are presented below.

9.3.1 Shallow Foundations

On a preliminary basis, light ancillary structures not associated with the subject building may be supported on conventional spread footings founded near existing grades and designed using permissible bearing values of 2,000 psf for dead and 3,000 psf for total design loads, provided the footing is established in properly engineered and approved fill soils. Settlements for these structure should be evaluated once details regarding site grading, foundation loads, and the particular location on the site are known.

The bearing values may be increased to 4,000 psf for dead and 6,000 psf for total design loads when established in the underlying Colma formation as identified on Plates 20 and 21, and confirmed by this office in the field. The allowable bearing values for total design loads may be further increased by one-third when seismic and transient loads are included.

Shallow foundations should be founded a minimum of 24 inches below the lowest adjacent final grade and should not be less than 12 inches in width for continuous and 24 inches in width for isolated footings. The bottom of all footing excavations should be clean and free of loose soils and debris. To the extent possible, the footings should be poured in neat excavations without the use of side forms. Appropriate reinforcing steel must be provided in the design of the spread footing foundations.

9.3.2 Drilled Pier & Grade Beam Foundations

Where the proposed development scheme will not remove all the overburden soils, the building should be supported on drilled, reinforced concrete piers driving support from skin friction within the Colma formation. The piers should be at least 24 inches in diameter, and should penetrate a minimum of 10 feet into the Colma formation as determined by this office in the field. Skin friction on the pier shaft within the supporting soils may be taken as 550 psf for total design loads, including seismic loads. Below a depth of 10 feet into the Colma formation, the skin friction value may be increased by 20 psf per additional foot of depth. Uplift capacity of the piers will be equal to their downward capacity on a preliminary basis. To limit reduction in pier capacity due to group action, drilled piers should be spaced no closer than 3 diameters, center-to-center.

Loads between piers should be supported on grade beams designed to span between the piers with no support from soils beneath them. All piers must be appropriately reinforced and pier and grade beam reinforcement should be properly tied together to enable the entire foundation system to act as a unit. On a preliminary basis, the main building floor may consist of a slab-on-grade floor without the need to span between the pier caps or grade beams; however, the need for a structurally supported slab will depend on the site location and proposed site grading.

The skin friction values used in calculating the axial capacity of drilled piers have been estimated based on our laboratory strength testing. To confirm or improve these design assumptions and the actual pier capacity, consideration can be given to static compression (downward) and tension tests performed in accordance with ASTM D1143 and D3689, respectively. Note that the piers subject to load tests will not be sacrificial and can be used for foundation support barring any observed structural failure of the pier, which is unlikely. Load testing of the piers may provide justification for increasing the allowable skin friction values.

While it is not clear there is an actual groundwater table beneath the site, there is a good probability that seepage will be encountered within the drilled pier holes. In the event caving conditions are encountered due to excessive seepage, it may be necessary to case the pier holes or drill with a heavy slurry in the hole, so as to prevent them from collapsing. The drilling contractor should therefore be prepared to deal with these issues as the field conditions dictate. While casing the piers may not be necessary for most, if not all, of the piers, we nevertheless recommend that casing the pier holes should be a line item in the drilling subcontract documents.

Depending on the amount of seepage, it should be pumped out of the pier holes before placing steel and concrete. Or, the concrete may be tremied into the holes and placed from bottom up. The bottom of the pier holes should be relatively clean, firm, and free of any loose cuttings before concrete is poured. If the bottom of the pier hole cannot be cleaned of loose cuttings, depending on the amount of water in the pier hole, it may be possible to place a couple bags of cement in the hole and mix the cuttings and cement with water. It is imperative that full-time observation of the pier drilling and pouring operations be provided by the Geotechnical Engineer.

9.4 Foundation Settlements

We have estimated that the total post construction settlements of the structure supported on properly designed and installed drilled pier foundations will be less than one inch under total loads, including seismic loads. We anticipate these settlements will be predominantly elastic in nature, with a minor amount of creep over time.

Estimates of settlements for shallow foundations can be made only after the structural loads have been estimated. However, we anticipate the above bearing pressures will result in settlements for a 4 to 5 story building of less than an inch or so, and will also be predominantly elastic in nature.

As indicated earlier, we have estimated that a 2-foot wide strip footing loaded at 2,000 psf would settle on the order of 1⅓ to 2 inches in the vicinity of Borings B-6 and B-7. To reduce this settlement to 1 inch, the bearing value would have to be reduced to 900 psf. We have also estimated that one foot of additional fill at the northeast edge of the site could settle on the order ¾ to 1½ inches.

9.5 Lateral Design

Lateral loads may be resisted by passive pressures against the sides of the drilled piers, pier caps, grade beams, and/or spread footings, and by friction on the bottom of shallow foundations. Passive resistance against the pier caps, grade beams, and footings may be taken as an equivalent fluid pressure of 350 pcf within properly engineered fill materials. Within the Colma formation, this value may be increased to 450 pcf. For isolated piers, spaced at least three diameters center to center, the passive soil pressure can be assumed to act over 1½ pier diameters. After foundation details have been developed, a more detailed analysis of lateral pier capacities can be provided.

Friction on the bottom of shallow foundations (not pier-supported grade beams) can be taken as 0.35 times the downward load on the footing.

9.6 Retaining Walls

Freestanding retaining walls supporting on-site fill materials or native soils above the groundwater table (elevation +5 feet) should be designed to resist active lateral pressures taken as an equivalent fluid pressure of 45 pounds per cubic foot (pcf) for level backfill. Walls that are restrained from movement at the top, such as basement and loading dock walls, should be designed to resist "at-rest" soil pressures that are based on an equivalent fluid weight of 65 pcf. Below the groundwater table, the restrained basement walls should be designed for an equivalent fluid pressure of 100 pcf for the native alluvial soils and 105 pcf for the soft Bay Mud soils.

The above-recommended earth pressures are for level backfill conditions. For sloping backfill, the above pressures should be increased by 3 pcf per every 5 degree increase in the backfill slope angle. Surcharge loads should be added to the above pressures at a rate of 33% and 50% percent of the applied surcharge load for cantilever and restrained walls, respectively.

The seismic pressures on the retaining walls may be simulated by a rectangular pressure distribution equal to 10H, where H is the height of the wall.

The lateral pressures given above assume that the basement walls above elevation +5 feet are adequately drained. Therefore, all walls (or portions thereof) that are above the water table and are over 2 feet in height should be provided with a drainage blanket behind the wall. The drainage blanket should consist of a pre-manufactured drainage panel or a one-foot-thick blanket of Caltrans Class 2 Permeable material or free-draining gravel encapsulated by a suitable filter fabric. A 12-inch cap of relatively impermeable soil should be placed at the top of the drainage blanket to minimize infiltration of surface water. The cap material should be compacted to a minimum of 90 percent relative compaction. A 4-inch diameter perforated PVC pipe should be installed at the base of the drainage layer to facilitate removal of water collected behind the wall.

General backfill behind the walls, excluding drainage materials, should conform to the fill requirements included under the "Site Grading" section of this report.

9.7 Slabs-on-Grade Building Floor and Exterior Flatwork

Concrete slabs should be constructed on a subgrade consisting of soils that have been prepared and compacted as recommended under "Site Grading."

Interior floor slabs near existing site grades should be underlain by a vapor barrier consisting of 15-mil-thick plastic membrane (such as Stegowrap® or an approved equivalent), and a 4-inch-thick capillary break. The capillary break should consist of free draining gravel such as ¾ inch by No. 4 crushed rock or gravel. An optional 2-inch-thick cushion of sand can be placed over the membrane to protect the membrane during construction, and should be moistened just prior to pouring the slab to aid in curing the concrete. For areas that will be used for parking and/or driveways, a minimum of 6 inches of Class II Aggregate Base compacted to at least 95% relative compaction should be placed beneath the concrete slab.

Floors in underground parking areas will likely (or at least potentially) be below the groundwater table and should be appropriately water proofed. Garage slabs should be underlain by 6 inches of Class II Aggregate Base compacted to 95 percent relative compaction, or should be underlain by a waste slab to provide uniformly firm support.

All building floors and exterior slabs should be appropriately reinforced with deformed bars. Experience suggests wire mesh contributes very little to the structural capacity of the slab, as more often than not, it ends up at the bottom of the slab rather than in the middle. The exterior concrete flatwork should have appropriately-spaced construction joints.

9.8 Drainage

Site drainage should be considered an integral part of the proposed construction. Drainage swales and contouring of the ground surface should be incorporated into the grading plan, and designed to provide sufficient slope from structures (5% minimum for at least 5 feet from foundations in unpaved areas) toward appropriate discharge points. Drainage swales should be cut into the graded pads and sloped to drain (1% minimum) to approved outfalls. Any area where surface run-off becomes concentrated should be provided with a catch basin that discharges the collected runoff in a manner that will not cause erosion.

The run-off from building roofs and intercepted water from surface drainage should be collected and discharged to suitable outfall locations (preferably in closed pipes unless the downspouts discharge the roof runoff onto paved surfaces) and in a manner that will not allow ponding adjacent to foundations. Where subdrains are necessary to intercept seepage, they should contain perforated pipes placed with perforations facing down. The pipe should be surrounded by drainage material (crushed rock or drain rock), and the drainage material should be wrapped in a suitable filter fabric. Surface and subsurface drainage facilities and catchment areas should be checked frequently and cleaned or maintained throughout the project life, as necessary.

Both Borings B-6 and B-7 encountered free water at a depth of 7½ feet. We therefore recommend the proposed basement should be designed for a groundwater elevation of +5 feet. Water encountered in our borings and likely to be encountered by the basement excavations, which are above this elevation, can be assumed to be zones of isolated seepage and drained accordingly. Below this depth, the building should be appropriately water-proofed.

9.9 Pavements

9.9.1 Flexible Pavements

We have calculated alternative pavement sections using the design method described in the Caltrans Highway Design Manual (Topic 633, July 2008) with the added safety factors. The method characterizes the subgrade soil conditions with laboratory R-value tests, and characterizes the traffic loading conditions with a Traffic Index.

R-value tests were not performed as a part of this investigation; however, based on our experience, we anticipate the sandy soils at the site will have an R-value on the order of 25. This value has been used in our analyses, but should be verified by testing of the subgrade soils once the rough subgrade has been achieved.

Using an assumed R-value of 25, the calculated pavement sections for Traffic Indices of 4.5, 5.0, 6.0, 7.0, and 8.0 using an aggregate base and a deep lift AC section are tabulated below. Generally, a Traffic Index of 4.5 is appropriate for automobile parking stalls, whereas a Traffic Index of 6.0 would be appropriate for heavily-used automobile driveways with only occasional use by heavy trucks (such as once a week or so by garbage trucks), and Traffic Indices of 7.0 or higher are used where the pavement would be subject to more frequent truck traffic such as daily use by delivery trucks.

Table 4
Summary of Asphaltic Concrete Pavement Sections
(Subgrade R-value=25)

Pavement Component	TI=4.5		TI=5.0		TI=6.0		TI=7.0		TI=8.0	
Asphaltic Concrete (AC) in Inches	5½	2½	6	3	8	3½	10	4	12	4½
Class II Aggregate Base (R_{Min}=78)		6		6		8½		10½		13
Total Thickness in Inches	5½	8½	6	9	8	12	10	14½	12	17½

All materials and construction procedures, including placement and compaction of pavement components, should be performed in conformance with the latest edition of the Caltrans Standard Specifications, except that compaction should be performed in accordance with ASTM Test Method D1557, and at moisture contents specified under the Site Grading section of this report.

The life of the asphaltic concrete pavement can be extended by placing concrete slab under and around the garbage dumpsters. The garage trucks lift up the dumpster and load the asphaltic pavement in the same location weekly, causing failure of the AC paving.

9.9.2 Rigid Pavements

Where Portland Cement Concrete (rigid) Pavements are to be used (recommended in garbage dump box storage areas), they should be supported on a subgrade that has been prepared as recommended under "Site Grading". Concrete pavements exposed to as much as 10 commercial trucks a day (TI= 7.0), should consist of 5½ inches of concrete with a compressive strength of 3,400 psi (MR=550 psi) supported on at least 6 inches of Class II Aggregate Base material compacted to a minimum of 95 percent relative compaction.

As a minimum, concrete pavements should be reinforced with deformed bars in both directions to control cracking, and joints should be provided in both directions within the pavement designed to prevent formation of irregular cracks.

Where traffic can drive over the edge of the concrete pavement, such as at transition to AC paving, the Portland Cement Association suggests the thickened edge should be increased by 20 percent, or about 1 inch in this case, and tapered back to normal slab thickness over a distance of 10 times the slab thickness, or in about 5 feet.

9.10 Utility Trench Backfill

Vertical trenches deeper than 5 feet will require temporary shoring. Where shoring is not used, the sides should be sloped or benched, with a maximum slope of 1:1 (horizontal: vertical) if the trench exposes clayey soils, and 1½:1 (H to V) if the material is granular and sandy in nature. The trench spoils should not be placed closer than 3 feet or one-half of the trench depth (whichever is greater) from the trench sidewalls. All work associated with trenching must conform to the State of California, Division of Industrial Safety requirements. Based on our boring and laboratory results, it is our opinion most of the soils at the site can be classified as a type "B" soil.

The utility lines should be properly bedded and shaded with granular material, such as, sand or pea gravel. As a general rule, the shading layer should extend at least 4 inches above the pipe. The bedding and shading layers should be compacted using a vibratory compactor. The contractor should use extreme caution with the vibratory compactor on the shading layer, as excessive vibrations and/or imbalanced shading materials could result in dislodging the pipe and loosening of the joints. The utility trenches may then be backfilled with on-site soils, provided they are free of debris, roots and other organic matter, and rocks or lumps exceeding 3 inches in greatest dimension. The fill material should be uniformly moisture conditioned to the proper moisture content and compacted as per the recommendations included in the "Site Grading" section of this report.

In order to avoid accumulation of surface water runoff in the utility trenches, the top 12-inches of the utility trench backfill should consist of uniformly moisture conditioned, and compacted, native soils with relatively low permeability. BAGG Engineers should be allowed an opportunity to observe the trench backfill operations and perform field compaction tests to evaluate the moisture content and relative compaction of the fill materials. Trench jetting should not be allowed.

Alternatively, the utility trenches may be backfilled with flowable fill (a cementitious slurry consisting of a mixture of fine aggregate or filler, water, and cementitious material(s) capable of filling all voids in irregular excavations and hard to reach places. The flowable fill is a self-leveling material that hardens in a matter of hours without the need for compaction in layers, and is sometimes referred to as controlled density fill (CDF), controlled low strength material (CLSM), and lean concrete slurry. A 2-sack flowable fill material is considered to be acceptable for the subject project.

The utility trenches located adjacent to shallow footings should not extend below an imaginary 1H:1V plane projected downward from the base of the footing. If deeper utility trenches are located adjacent to the footings, the footing depths should be increased so that the utility trench excavation is outside this imaginary plane.

9.11 Plan Review

It is recommended that the Geotechnical Engineer (BAGG Engineers) be retained to review the final grading, drainage, and foundation, plans. This review is intended to verify the appropriate implementation of our recommendations into the project plans and specifications, as well as to assess the general suitability of the recommendations contained in this report for the project in its final form.

9.12 Geotechnical Observation & Testing

It is recommended that the BAGG Engineers be retained to provide observation and testing services during the grading, excavation, shoring installation, foundation construction, backfilling, and preparation of subgrades. This is intended to verify that the work in the field is performed as recommended and in accordance with the approved plans and specifications; and more importantly, to verify that subsurface conditions encountered during construction are similar to those anticipated during the design phase. Changed or unanticipated conditions may warrant revised recommendations. For this reason, BAGG Engineers cannot assume responsibility or liability for the recommendations contained in this report if we do not provide observations and testing services during construction.

10.0 CLOSURE

This report has been prepared in accordance with generally accepted engineering practices for the strict use of VAM Millbrae Serra and other professionals associated with the specific project described in this report. The recommendations presented in this report are based on our understanding of the proposed construction as described herein, and upon the subsurface conditions encountered in several exploratory borings drilled on the site.

The conclusions and recommendations contained in this report are based on subsurface conditions revealed by widely spaced exploration points. It is not uncommon for unanticipated conditions to be encountered during foundation installation and it is not possible for all such variations to be found by a field exploration program appropriate for this type of project. The recommendations contained in this report are therefore contingent upon the review of the final grading, drainage, and foundation plans by this office, and upon geotechnical observation and testing by BAGG of all pertinent aspects of site grading and foundation installation.

Subsurface conditions and standards of practice change with time. Therefore, we should be consulted to update this report, if the construction does not commence within 18 months from the date this report is submitted. Additionally, the recommendations of this report are only valid for the proposed development as described herein. If the proposed project is modified, our recommendations should be reviewed and approved or modified by this office in writing.

The following references and plates are attached and complete this report:

Plate 1	Vicinity Map
Plate 2	Site Plan
Plate 3	Regional Geologic Map
Plate 4	Regional Fault Map
Plate 5	Unified Soil Classification System
Plate 6	Soil Terminology
Plate 7	Rock Terminology
Plates 8-A thru 15-C	Boring Logs
Plates 16 and 17	Atterberg Limits
Plates 18 and 19	Consolidation Data
Plate 20	Generalized Cross Section A-A'
Plate 21	Generalized Cross Section B-B'

11.0 REFERENCES

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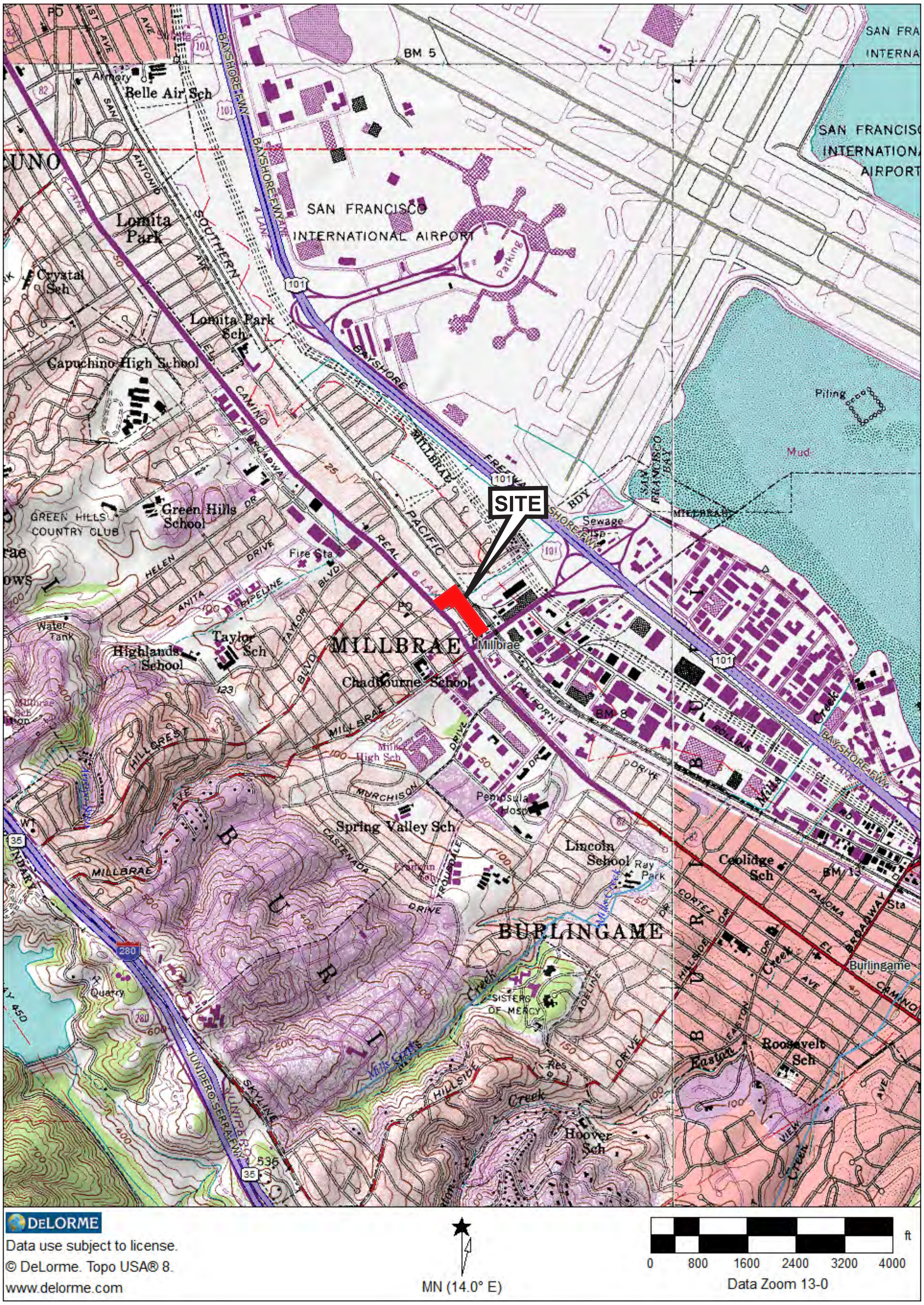
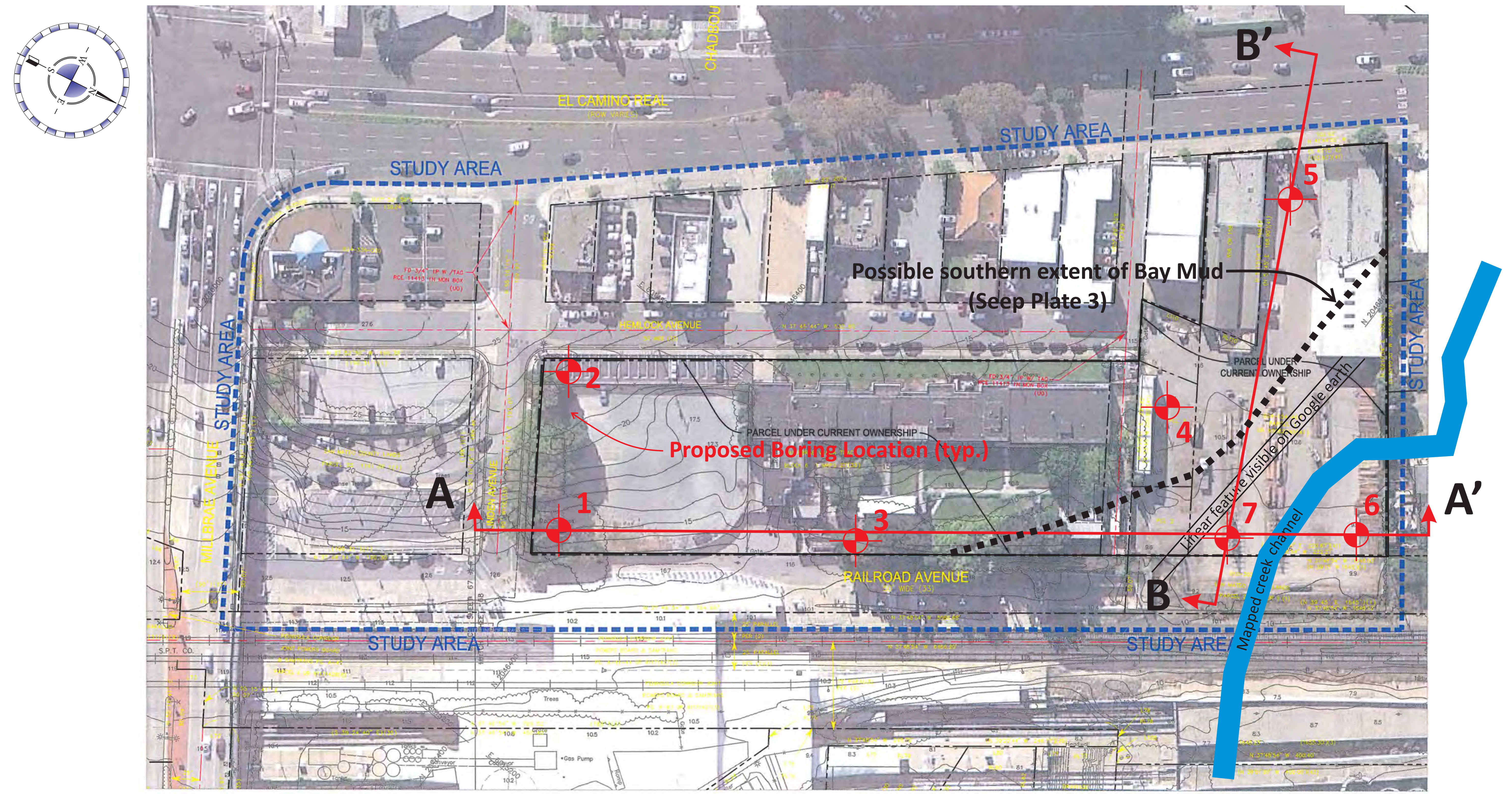


PROPOSED MILLBRAE STATION **EL CAMINO REAL @ MILLBRAE AVENUE** **MILLBRAE, CALIFORNIA**	**VICINITY MAP**		
	DATE: June 2013	JOB NUMBER: VAMMS-01-00	PLATE 1



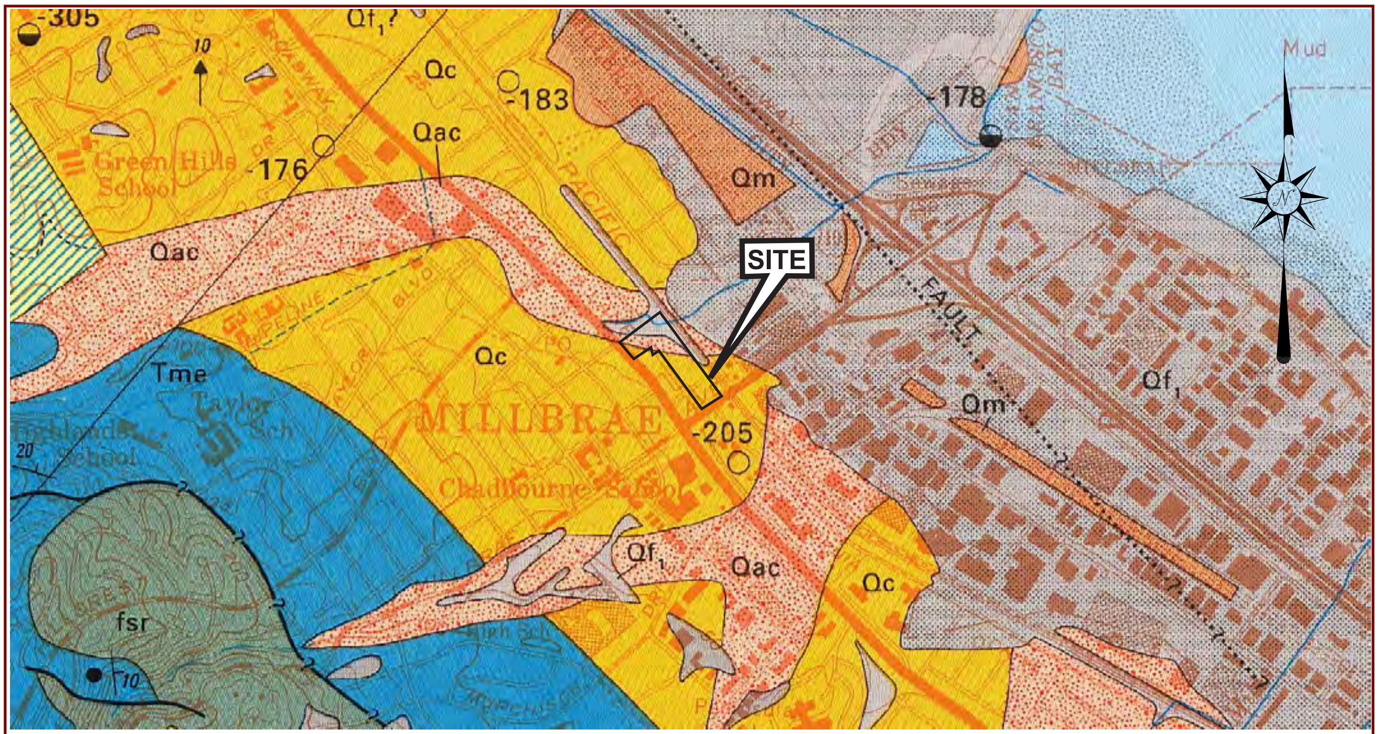
Base Map:
Study Area Exhibit, Millbrae Station West Side Properties, Millbrae, California,
Sheet EX1, by BKF Engineers, dated 3/6/13

PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

BAGG ENGINEERS

SITE PLAN

JOB NUMBER:	SCALE:	DATE:	PLATE
VAMMS-01-00	None	June 2013	2



LEGEND

Qm **Bay Mud (Holocene)**

Qf **Artificial Fill (Historic) -** Poorly consolidated to well consolidated gravel, sand, silt, and rock fragments in various combinations used in a variety of applications including riprap, highway-, railroad-,l and airport runway-fills, earthfill dams, reservoirs embankments, and building-site grades. Thickness and consolidation dependent upon type of application and site.

Qac **Coarse-grained Alluvium (Holocene) -** Unconsolidated, moderately sorted sand and gravel forming stream levees, fans, and flood plains in and close to upland areas. Grades coarser headward. Locally contains lenticular interlayes of well-sorted silt, sand, and gravel; locally contains modern vertebrate and invertebrate fossils. Interfingers with medium-grained alluvium and colluvial deposits. Maximum thickness probably less than 75 ft.

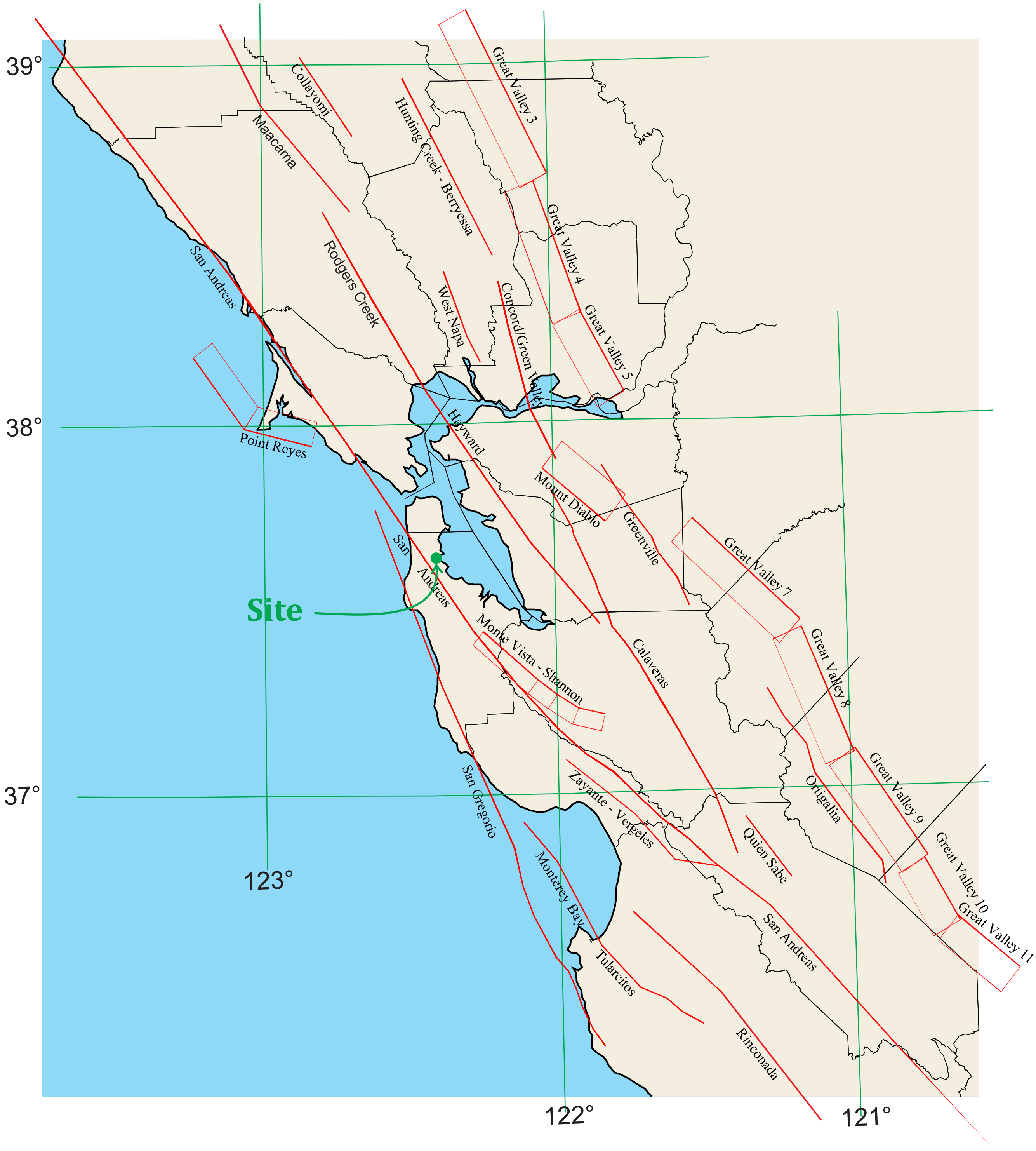
Qc **Colma Formation (Pliestocene) _** Weakly consolidated, moderately well bedded, yellowish-gray and tan sandy clay and silty sand, and friable light- to reddish-brown, poorly sorted to well sorted sand and gravel. Thin- to thick-bedded with cross bedding commonly present in friable sands. Silty sand beds commonly contain zone os scattered well-rounded and polished chert pebbles. A shallow-water marine deposit. Total thickness unknown but probably exceeds 100 feet.

Tme **Merced Formation (Upper Pliocene)**

fsr **Franciscan Complex, sheared rock (Cretaceous and Jurassic)**

Reference: *Geologic Map of the Montara Mountain and San Mateo 7-1/2' Quadrangles, San Mateo county, California*, 1994 by Earl H. Pampeyan, U.S.G.S. Miscellaneous Investigations Series, Map I-2390.

PROPOSED MILLBRAE STATION **EL CAMINO REAL @ MILLBRAE AVENUE** **MILLBRAE, CALIFORNIA**	**REGIONAL GEOLOGY MAP**		
	DATE: June 2013	JOB NUMBER: VAMMS-01-00	PLATE 3



Reference: Taken from the 2002 California Geological Survey Fault Model.

PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

REGIONAL FAULT MAP

DATE:	JOB NUMBER:	PLATE
June 2013	VAMMS-01-00	4

BAGG ENGINEERS

COARSE-GRAINED SOILS
LESS THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
GW	Well graded gravel Well graded gravel with sand	**GRAVELS** More than half of coarse fraction is larger than No. 4 sieve size
GP	Poorly graded gravel Poorly graded gravel with sand	
GM	Silty gravel Silty gravel with sand	
GC	Clayey gravel Clayey gravel with sand	
SW	Well graded sand Well graded sand with gravel	**SANDS** More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly graded sand Poorly graded sand with gravel	
SM	Silty sand Silty sand with gravel	
SC	Clayey sand Clayey sand with gravel	

NOTE: Coarse-grained soils receive dual symbols if:
(1) their fines are CL-ML (e.g. SC-SM or GC-GM) or
(2) they contain 5-12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS
MORE THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
CL	Lean clay Sandy lean clay with gravel	**SILTS AND CLAYS** liquid limit less than 50
ML	Silt Sandy silt with gravel	
OL	Organic clay Sandy organic clay with gravel	
CH	Fat clay Sandy fat clay with gravel	**SILTS AND CLAYS** liquid limit more than 50
MH	Elastic silt Sandy elastic silt with gravel	
OH	Organic clay Sandy organic clay with gravel	
PT	Peat Highly organic silt	**HIGHLY ORGANIC SOIL**

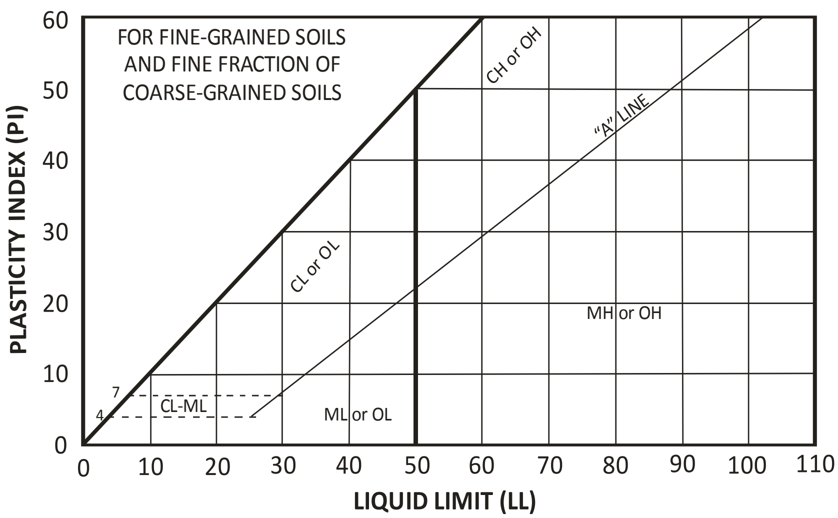
NOTE: Fine-grained soils receive dual symbols if their limits in the hatched zone on the Plasticity Chart(L-M)

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	¾ in to 3 in.
Fine	No. 4 to ¾ in.
SAND	No. 200 to No.4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
*FINES:	BELOW No. 200

NOTE: Classification is based on the portion of a sample that passes the 3-inch sieve.

PLASTICITY CHART



Reference: ASTM D 2487-06, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).

GENERAL NOTES: The tables list 30 out of a possible 110 Group Names, all of which are assigned to unique proportions of constituent soils. Flow charts in ASTM D 2487-06 aid assignment of the Group Names. Some general rules for fine grained soils are: less than 15% sand or gravel is not mentioned; 15% to 25% sand or gravel is termed "with sand" or "with gravel", and 30% to 49% sand or gravel is termed "sandy" or "gravelly". Some general rules for coarse-grained soils are: uniformly-graded or gap-graded soils are "Poorly" graded (SP or GP); 15% or more sand or gravel is termed "with sand" or "with gravel", 15% to 25% clay and silt is termed clayey and silty and any cobbles or boulders are termed "with cobbles" or "with boulders".

UNIFIED SOIL CLASSIFICATION SYSTEM

SOIL TYPES (Ref 1)

Boulders: particles of rock that will not pass a 12-inch screen.
Cobbles: particles of rock that will pass a 12-inch screen, but not a 3-inch sieve.
Gravel: particles of rock that will pass a 3-inch sieve, but not a #4 sieve.
Sand: particles of rock that will pass a #4 sieve, but not a #200 sieve.
Silt: soil that will pass a #200 sieve, that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry.
Clay: soil that will pass a #200 sieve, that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry.

MOISTURE AND DENSITY

Moisture Condition: an observational term; dry, moist, wet, or saturated.
Moisture Content: the weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage.
Dry Density: the pounds of dry soil in a cubic foot of soil.

DESCRIPTORS OF CONSISTENCY (Ref 3)

Liquid Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty.
Plasticity Index: the difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Ref's 2 & 3)

Very Soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by a pencil point

*__N__=blows per foot in the Standard Penetration Test. In cohesive soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 1.2 to get N (Ref 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND SILTS) (Ref's 2 & 3)

Very Loose	N=0-4**	RD=0-30	Easily push a ½-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a ½-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a ½-inch reinforcing rod
Dense	N=31-50	RD=70-90	Drive a ½-inch reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a ½-inch reinforcing rod a few inches

**__N__=Blows per foot in the Standard Penetration Test. In granular soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 2 to get N (Ref 4).

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Ref 1: ASTM Designation: D 2487-06, **Standard Classification of Soils for Engineering Purposes** (Unified Soil Classification System).

Ref 2: Terzaghi, Karl, and Peck, Ralph B., **Soil Mechanics in Engineering Practice,** John Wiley & Sons, New York, 2nd Ed., 1967, pp. 30, 341, and 347.

Ref 3: Sowers, George F., **Introductory Soil Mechanics and Foundations: Geotechnical Engineering,** Macmillan Publishing Company, New York, 4th Ed., 1979, pp. 80, 81, and 312.

Ref 4: Lowe, John III, and Zaccheo, Phillip F., **Subsurface Explorations and Sampling,** Chapter 1 in "Foundation Engineering Handbook," Hsai-Yang Fang, Editor, Van Nostrand Reinhold Company, New York, 2nd Ed, 1991, p. 39.

SOIL TERMINOLOGY

WEATHERING DESCRIPTORS

Fresh — No discoloration, not oxidized, no separation, hammer rings when crystalline rocks are struck.

Slight — Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull, no visible separation, hammer rings when crystalline rocks are struck, body of rock not weakened.

Moderate — Discoloration extends from fractures, usually throughout ;Fe-Mg materials are “rusty”, feldspar crystals are “cloudy”, all fractures are discolored or oxidized, partial separation of boundaries visible, texture generally preserved, hammer dose not ring when rock is struck, body of rock is slightly weakened.

Intense — Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, all fracture surfaces are discolored or oxidized, surfaces friable, partial separation, texture altered by chemical disintegration, dull sound when struck with hammer, rock is significantly weakened.

Decomposed — Discolored or oxidized throughout, but resistant mineral such as quartz may be unaltered, all feldspars and Fe-Mg minerals are completely altered to clay, complete separation of grain boundaries, resembles a soil, partial or complete remnant of rock structure may be preserved, can be granulated by hand, resistant minerals such as quartz may be present as “stringers” or “dykes”.

BEDDING FOLIATION AND FRACTURE SPACING DESCRIPTORS

Millimeters	Feet	Bedding	Fracture Spacing
>10	<0.03	Laminated	Very Close
10-30	0.03-0.1	Very Thin	Very Close
30-100	0.1-0.3	Thin	Close
100-300	0.3-1	Moderate	Moderate
300-1000	1-3	Thick	Wide
1000-3000	3-10	Very Thick	Very Wide
>3000	>10	Massive	Extremely Wide

ROCK HARDNESS/STRENGTH DESCRIPTORS*

Extremely Hard — Core, fragment, or exposure cannot be scratched with knife or sharp pick; can only be chipped with repeated heavy hammer blows.

Very Hard — Cannot be scratched with knife or sharp pick. Core or fragment breaks with repeated heavy hammer blows.

Hard — Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.

Moderately Hard — Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.

Moderately Soft — Can be grooved $^{1}/_{16}$ inch (2mm) deep by knife or sharp pick with moderate or heavy pressure. Core fragment breaks with light hammer blow or heavy manual pressure.

Soft — Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks wit light to moderate manual pressure.

Very Soft — Can be readily indented, grooved, or gouged with fingernail, or carved with a knife. Breaks with light manual pressure.

*Note: Although “sharp pick” is included in those definitions, descriptions of ability to be scratched, grooved, or gouged by a knife is the preferred criteria.

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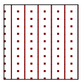
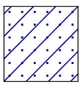
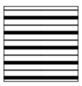
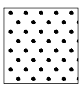
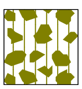
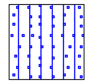
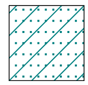
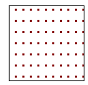
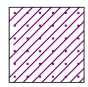
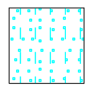
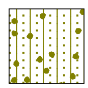
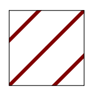
"Engineering Geology Field Manual, Second Edition, Volume 1, by U.S. Department of Interior, Bureau of Reclamation, 1998

ROCK TERMINOLOGY

BAGG ENGINEERS

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Silty sand
	Sandy lean clay
	Shale
	Sandstone
	Paving
	Silty gravel

Symbol	Description
	Borderline silty sand to sandy silt
	Clayey sand
	Poorly graded sand
	Borderline sandy lean clay to clayey sand
	Well graded sand with silt
	Silty sand with gravel
	High plasticity (fat) clay

Notes:

1. The borings were drilled on April 29 through May1, 2013 with a truck-mounted drill rig using 8" O.D. hollow- stem aaugers.

2. Free gourndwater was noted at various depths in all borings except B-1 and B-5. The water levels in the bore holes were not allowed to stabilize, and may not be representative of the true groundwater table.

3. Following the completion of drilling, each boring was sealed with a neat cement grout.

4. The borings were located on the site by pacing distances from landmarks shown on the site plan. The plotted locations are therefore only approximate.

5. The shear strength values shown on the boring logs are the yield strengths, or the strength measured when the material began to deform plastically.

6. The soils were visually classified in the field in accordance with the Unified Soil Classification System (Plate 5). The logs were then edited in the laboratory based on visual examination, and supported where indicated, by classification tests. In additions to interpretations for sample classification, the logs cootain interpretations of where soil changes occur between samples, where gradational changes substantially occur, and where minor changes within a stratum are significnat enough to log.

7. The boring logs depict BAGG's interprtation of subsurface conditions at the locations shown on the site plan, and on the dates indicated on the logs. The logs are intended for use only in coonjunction with this report, and only for the purposes outlined herein.

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KEY TO SYMBOLS

Symbol Description

Strata symbols

Lean Clay

Misc. Symbols

Boring continues

Water first encountered during drilling

Soil Samplers

Modified California Sampler: 2.375" ID by 3" OD, split-barrel sampler driven w/ 140-pound hammer falling 30 inches

Standard Penetration Test: 1 3/8" ID by 2" OD, split-spoon sampler driven with 140-pound hammer falling 30" (ASTM D 1586-99)

Bulk sample from augers.

Undisturbed, thin-wall Shelby tube (ASTM D 1587-00)

Line Types

Denotes a sudden, or well identified strata change

Denotes a gradual, or poorly identified strata change

Laboratory Data

Symbol	Description
DS	Direct shear test performed at field (Natural) water content. (ASTM D2166)
DSX	Dirct Shear test performed on a sample submerged in water until volume changes ceased. (ASTM D2166)
LL	Liquid Limit as determined by ASTM D4318
PI	Plasticity Index as determined by ASTM D4318

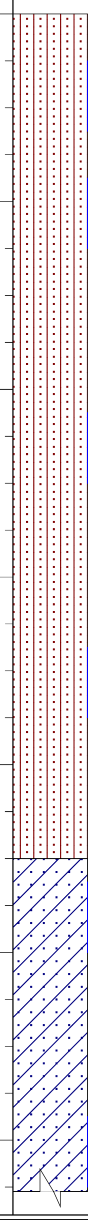
BORING LOG

Boring No. B-1

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 4/30/13
ELEVATION: 17±
LOGGED BY: KO
CHECKED BY:



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0				
								SM	SILTY SAND, dark brown, moist, medium dense, fine-grained sand	FILL
DSX	320	15	370	11	116		5 6 19			
								SM	SILTY SAND, dark brown, moist, medium dense, fine-grained sand, some rootlets, little fine-grained gravel	NATIVE topsoil
DSX	500	10	915	12	123	4	12 16 20			
								SM	SILTY FINE SAND, light yellow brown, moist, dense	Colma formation
						8			..some reddish brown staining, very dense	
				12	123		26 35 50			
						12				
				13	120		12 34 42			
						16				
								CL	SANDY LEAN CLAY, light yellow brown w/ some rust staining, moist, very stiff, fine-grained sand	
				13	123	20	21 26 31			
				15	119	24	12 24 25			

BORING LOG

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28				
DS	3000	NAT	1860	17	116		19 17 22	ROCK	SHALE, gray brown, highly weathered and sheared, moist	Franciscan formation?
						32				
				14.0	121		50/6"		...very clayey, medium gray	
								ROCK	SANDSTONE, light gray, hard, moderately weathered	
						36			Boring was terminated at 35 feet. Borehole was not stabilized for groundwater reading. Borehole was backfilled with neat cement grout.	
						40				
						44				
						48				
						52				

BORING LOG

Boring No. B-2
Page 1 of 2

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 4/29/13
ELEVATION: 25
LOGGED BY: KO
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		GM	3" AC over 4" Aggregate Base	
								CL	SANDY CLAY, gray brown, very stiff, some reddish brown staining, fine-grained sand	Native
DSX	350	20	650	15	119		4 11 23			PI=34, LL=46
				14	115.1	4	14 42 50/5"		...increase in sand content, some rootlets, light brown	
								SM	SILTY SAND, light yellow brown, moist, medium, dense	Colma formation
						8				
DS	1000	NAT	730	19 18	110		5 11 12		...olive brown, trace clay, some reddish brown staining	44% Passing No. 200 Sieve
						12				
DS	1500	NAT	1200	15	115		4 6 10			
						16				
				15	119	20	8 13 16			
						24	6 12 32		...brown, some fine gravel	
				16.0						

BORING LOG

JOB NAME: Proposed Milbrae Station *JOB NO.:* VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28		SC-SM	SILTY to CLAYEY FINE SAND, yellow brown, dense, saturated	
DS	3000	NAT	2530	24	103		19 22 31			43% Passing No. 200 Sieve
						32				
								SM	SILTY FINE SAND, yellow brown, saturated, dense, some medium to coarse sand	
				24			15 50/6"			
							15 25 32			
						36				
						40				
						44	11 17 32		...brown	
				22						28% Passing No. 200 Sieve
						48				
				17			21 37 50/5"		...less silt, w/ trace pea gravel	
									Boring terminated at 50 feet. Groundwater encountered at 15½ feet. Borehole backfilled with neat cement grout.	
						52				

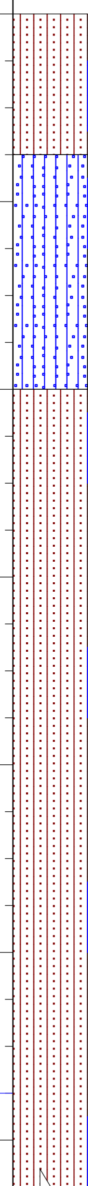
BORING LOG

Boring No. B-3

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 4/29/13
ELEVATION: 13±
LOGGED BY: KO
CHECKED BY:



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		SM	SILTY SAND, brown, dry, medium dense, some rootlets	FILL
DSX	320	7	140	18	113		11 10 12			
							6 7 12	SM/ ML	SILTY FINE SAND to SANDY SILT, dark brown, dry, medium dense, some rootlets	Native topsoil
DSX	500	12	795	15	120	4				
						8	7 27 35	SM	SILTY FINE SAND, light yellow brown, some reddish brown staining, some fine-grained gravel	Colma formation
DS	1000	NAT	680	7	116					
						12			...blue gray	
DS	1500	NAT	1405	15	119		11 17 18			
						16				
							6 17 27			
				17	122	20				PI=0; LL=18 32% Passing No. 200 Sieve
						24	11 19 23			
DS	2500	NAT	1800	15	116				...saturated, some fine-grained gravel	

BORING LOG

Boring No. B-3

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28			...medium dense	
DS	3000	NAT	2040	19	113		11 12 17			PI=0; LL=15 25% Passing No. 200 Sieve
						32				
DS	3500	NAT	2560	16	119		17 15 17		...trace fine-grained gravel	PI=11; LL=25 33% Passing No. 200 Sieve
						36				
				16	121		17 27 31	ROCK	SHALE, brown and olive brown, broken/crushed, weathered, clayey	Franciscan formation?
						40				
				28		44	21 28 35			
										Hard drilling
						48				
				20			17 50/6"			
									Boring was terminated at 49½ feet. Groundwater was encountered at 23 feet. Borehole backfilled with neat cement grout.	
						52				

BORING LOG

Boring No. B-4

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 5/1/13
ELEVATION: 14±
LOGGED BY: KO
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		GM	3" AC over 33" Aggregate Base	FILL
								SM	SILTY FINE SAND, dark brown, moist, medium dense, very fine-grained sand, reddish brown staining	FILL
DSX	500	16	540	16	114	4	8 9 8			
DSX	750	12	225	19	108		7 8 5		...brown, pockets of fine- and coarse-gained sand with occasional pea gravel	
						8		CL	SANDY LEAN CLAY, light yellow brown, moist, medium stiff, reddish brown staining	Native soils Lost sample
							8 11 12		...becomes stiff	
				21						PI=29, LL=39 52% Passing No. 200 Sieve
						12		SC	CLAYEY FINE SAND, light yellow brown, moist, medium dense	Colma formation
DS	1500	NAT	1600	17	115		6 14 21			32% Passing No. 200 Sieve
									...increase in sand content	
						16				
				15	119	20	6 14 20			
DS	2500	NAT	1790	17	114	24	7 13 18			

BORING LOG

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28		SM	SILTY SAND, light yellow brown, saturated, medium dense/dense	
							13 19 21			
				18	118					
						32				
								SC	CLAYEY SAND, light olive brown, saturated, dense, reddish brown staining	
							11 13 22			
DS	3500	NAT	2050	22	108					48% Passing No. 200 Sieve
						36				
							16 24 32			
				34	90					PI=26, LL=40
						40				
								SM	SILTY FINE SAND, light yellow brown, very dense, saturated, reddish brown and black staining	
						44	14 32 43			
DS	4500	NAT	2600	21	102					
						48	30 50/6"			
				25	100					
						52				

BORING LOG

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
							13 19 32		...little gravel	
						56				
				20			23 50/6"			
						60			Boring was terminated at 59.5 feet. Groundwater was encountered at 27.5 feet. Borehole backfilled with neat cement grout.	
						64				
						68				
						72				
						76				
						80				

BORING LOG

Boring No. B-5

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 5/1/13
ELEVATION: 15±
LOGGED BY: KO
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		GM	3" AC over 4" Aggregate Base	FILL
								SM	SILTY SAND, brown, moist, loose	
DS	350	NAT	285	16	112		4 5 6			
						4			...yellow brown	
DS	600	NAT	350	22	106		7 8 12			
								CL	SANDY CLAY, yellow brown, moist, stiff	Native Soil
						8		SC	CLAYEY SAND, yellow brown, moist medium dense	
DS	900	NAT	495	17	113		8 11 13			
								CL	SANDY CLAY, reddish brown, moist, stiff, trace coarse sand	
						12				
DS	1500	NAT	2040	16	117		6 9 11			
						16				
								SC	CLAYEY FINE SAND, yellow brown, moist, medium dense, reddish brown staining	Colma formation
				16	116	20	13 17 21			PI=10; LL=21 35% Passing No. 200 Sieve
				15		24	6 8 10		...becomes more clayey	PI=16,LL=26 33% Passing No.

BORING LOG

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
										200 Sieve
						28		SM/ ML	SILTY FINE SAND to SANDY SILT, Light brown, moist, medium dense	
DS	3000	NAT	1770	19 19	112		5 11 19 7 11 15			PI=0; LL=17 51% Passing No. 200 Sieve
						32				
				16			9 11 12	SC	CLAYEY FINE SAND, light brown, moist, medium dense	35% Passing No. 200 Sieve
						36				
				15			9 12 15	CL	SANDY CLAY, yellow brown, moist, very stiff, reddish brown and black staining, trace pea gravel	
						40				
DS	4500	NAT	2980	18	111	44	25 50/5"	SM	SILTY SAND, yellow brown, saturated, dense, reddish brown and black staining	
						48				
				29			15 19 23		...brown	
						52				

BORING LOG

Boring No. B-5

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				20			16 18 25			
						56				
				22			19 25 32		...trace clay	
						60			Boring was terminated at 60 feet bgs. Borehole was not stabilized for groundwater reading. Borehole was backfilled with neat cement grout.	
						64				
						68				
						72				
						76				
						80				

BORING LOG

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 4/30/13
ELEVATION: 12±
LOGGED BY: KO
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		SM	SILTY SAND, yellow brown, dense, dry, reddish brown staining, black spotting, little gravel	FILL
DSX	500	19.1	695	15	114		20 21 23			
DSX	500	17	490	13	111	4	12 9 8		...olive brown	
								SP	POORLY GRADED SAND, yellow brown, moist, loose	FILL
						8		CL/ SC	CLAYEY FINE SAND to SANDY CLAY, dark gray to black, wet, loose/soft, trace rootlets	Native Bay Deposits
DS	1000	NAT	135	81	52		3 4 4			
							0 psi			
DSX	1000	44	700	45	73	12			...trace of pae gravel in tip	Consol (Plate 17)
								SC	CLAYEY SAND, blue gray w/ reddish brown staining, saturated, medium dense	
DS	1500	NAT	840	19 21	112		6 9 13 5 11 13		...reddish brown staining	
						16		SW-SM	WELL GRADED SAND, dark yellow brown, wet, dense, some gravel and trace silt	Colma formation
				19 16	114	20	27 34 26 15 21 25			
								CL	SANDY LEAN CLAY, light olive brown, wet, very stiff, yellow brown and black staining	
DS	2500	NAT	1895	18	114	24	13 21 27			PI=22, LL=32

BORING LOG

Boring No. B-6

JOB NAME: Proposed Milbrae Station *JOB NO.:* VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28				
				20	110		50/6"		...increase in sand content	
								SP	FINE SAND, yellow brown, wet, very dense, trace silt, some reddish brown staining	
						32				
DS	3500	NAT	2225	22	109		21 50/6"		...brown	
						36				
				23	105		12 50/6"		...gray brown	
						40				
								SM	SILTY FINE SAND, brown, wet, very dense	
						44	20 50/6"			Lost sample
				23					...brown	
						48				
				21			34 50/5"			21% Passing No. 200 Sieve
						52				

BORING LOG

JOB NAME: Proposed Milbrae Station *JOB NO.:* VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				22			16 20 40			
						56				
				22		60	12 14 38			
				24		64	19 23 33			
									Boring terminated at 65 feet. Groundwater encountered at 7.5 feet. Borehole backfilled with neat cement grout.	
						68				
						72				
						76				
						80				

BORING LOG

Boring No. B-7

BAGG ENGINEERS

JOB NAME: Proposed Milbrae Station
CLIENT: VAM Milbrae Serra LLC
LOCATION: 200 El Camino Real
DRILLER: Exploration GeoServices
DRILL METHOD: Truck Mounted Drilling Rig with 4" Continuous Flight Augers

JOB NO.: VAMMS-01-00
DATE DRILLED: 4/29/13
ELEVATION: 12±
LOGGED BY: KO
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		GM	2" AC over 3" Aggregate Base	FILL
							6 3 1	SM	SILTY SAND with gravel, brown, moist, very loose	
DS	400	NAT	340	15	109	4	3 6 8	SM	SILTY SAND, brown, moist, loose (no gravel)	FILL
DS	900	NAT	370	83	51	8	1 2 2	CH	FAT CLAY, dark gray, wet, soft ...some rootlets	Native Bay Mud
DSX	1000	81	675	81	51	12	0 psi			Consol (Plate 18)
DS DS	1300 2000	NAT NAT	620 740	26 24	91 103		3 7 8	CL	FINE SANDY CLAY, black, wet, medium stiff	PI=35, LL=45
						16		SM	SILTY FINE SAND, light grayish green, wet, dense, some gravel, little clay binder	Colma formation
				17	118	20	17 24 28			
				25	101	24	6 12 21		...becomes very fine-grained,	

BORING LOG

Boring No. B-7

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
									yellow brown w/ rust staining	
						28				
DS	3000	NAT	1910	29	96		11 13 16		...less silt	
						32				
				21	107		28 50/6"		...olive brown, very dense	
						36				
				21	107		19 29 50		...brown, very fine-grained sand, increase in silt content	
						40				
				21	107	44	17 50/6"		...medium to fine sand, less silt	
						48				
				24			19 32 50		...more silt content	36% Passing No. 200 Sieve
						52				

Plate 15 - C

BORING LOG

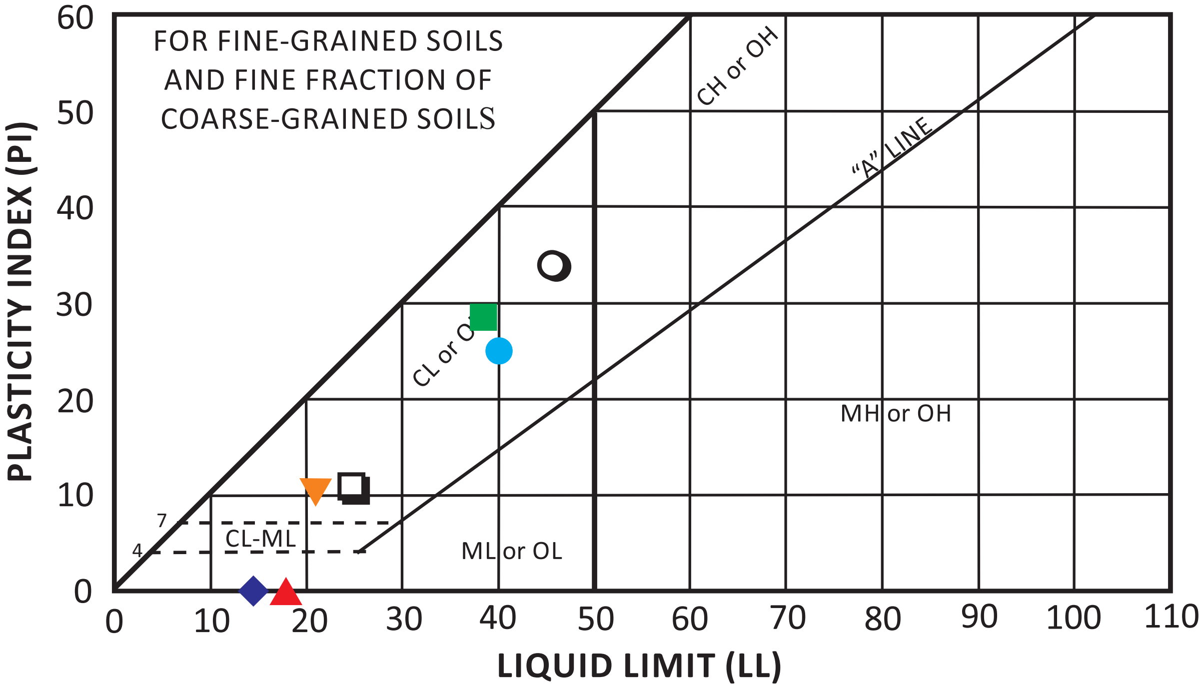
Boring No. B-7

JOB NAME: Proposed Milbrae Station

JOB NO.: VAMMS-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				19			22 29 38		...some gravel	
						56				
				18		60	19 21 23		...blue gray, dense, no gravel	32% Passing No. 200 Sieve
				24		64	11 13 17		...yellow brown	
									Boring terminated at 65 feet. Groundwater encountered at 7.5 feet. Borehole backfilled with neat cement.	
						68				
						72				
						76				
						80				

PLASTICITY CHART



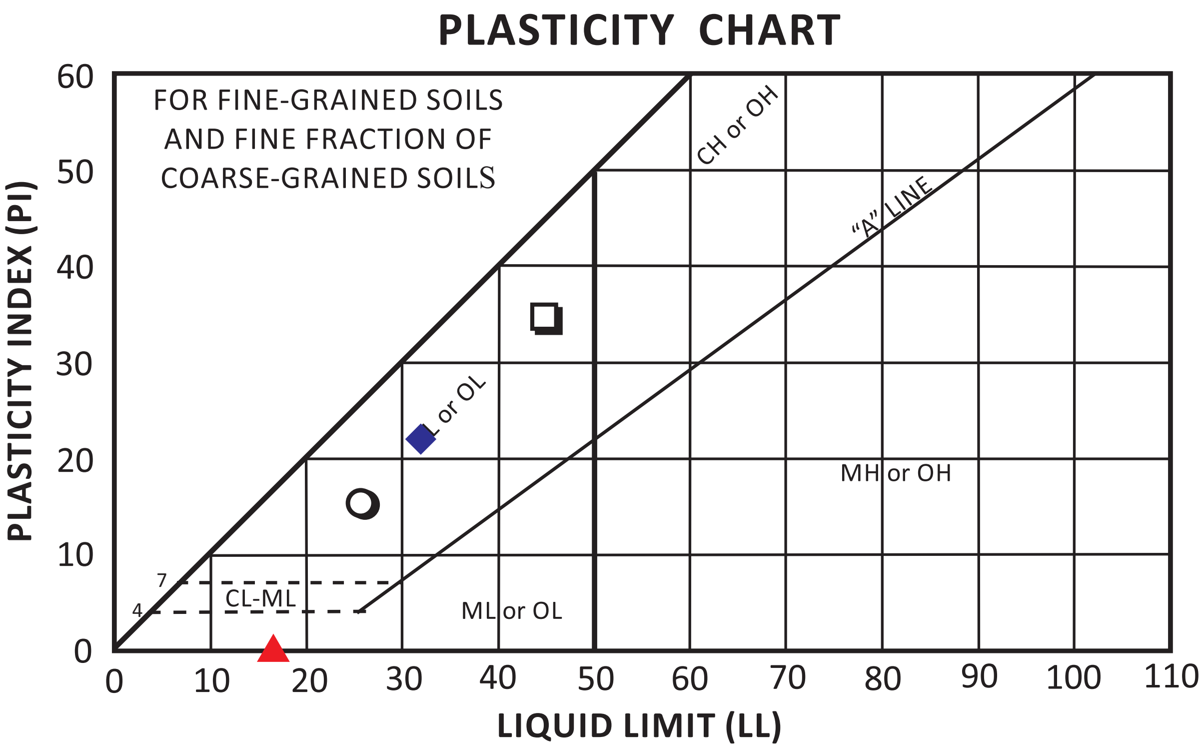
SYMBOL	SAMPLE SOURCE	DEPTH (FEET)	NATURAL WATER CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION
○	Boring B-1	2	15	46	12	34	Gray brown sandy clay (CL)
▲	Boring B-3	19.5	17	18	–	0	Gray brown silty sand (SM)
◆	Boring B-3	29.5	19	15	–	0	Gray brown silty sand (SM)
□	Boring B-3	34.5	16	25	14	11	Gray brown silty sand (SM)
■	Boring B-4	10	21	39	10	29	Light yellow brown sandy clay (CL)
●	Boring B-4	39.5	34	40	14	26	Light olive brown clayey sand (SC)
▼	Boring B-5	19.5	16	21	11	10	Yellow brown clayey sand (SC)

PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

ATTERBERG LIMITS

DATE:	JOB NUMBER:	PLATE
June 2013	VAMMS-01-00	16

BAGG ENGINEERS



SYMBOL	SAMPLE SOURCE	DEPTH (FEET)	NATURAL WATER CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL DESCRIPTION
○	Boring B-5	23.5	15	26	10	16	Yellow brown clayey sand (SC)
▲	Boring B-5	29.5	19	17	–	0	Light brown silty sand to sandy silt (SM/ML)
◆	Boring B-6	24.5	18	32	10	22	Light olive brown sandy clay (SM)
□	Boring B-7	14	24	45	10	35	Black sandy clay (SM)

PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

ATTERBERG LIMITS

DATE: June 2013

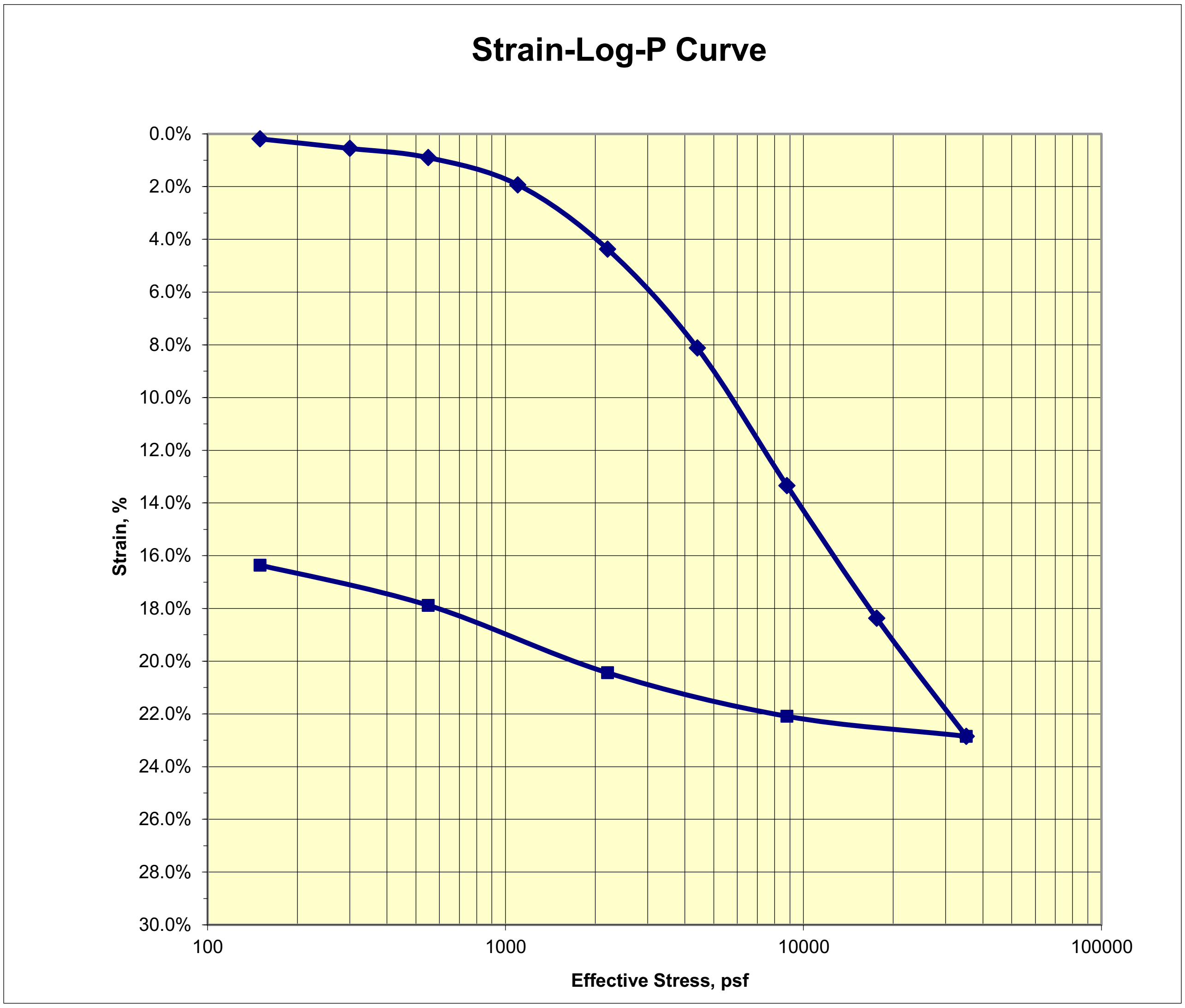
JOB NUMBER: VAMMS-01-00

PLATE 17

BAGG ENGINEERS

Consolidation Test
ASTM D2435

Job No.:	011-553	**Boring:**	B-6	**Run By:**	MD
Client:	BAGG	**Sample:**	4	**Reduced:**	PJ
Project:	VAMMS-01-00	**Depth, ft.:**	10(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Black Sandy CLAY w/ organics			**Date:**	5/28/2013



Ass. Gs = 2.7	**Initial**	**Final**
Moisture %:	38.6	28.3
Dry Density, pcf:	78.7	95.7
Void Ratio:	1.142	0.762
% Saturation:	91.2	100

Remarks:

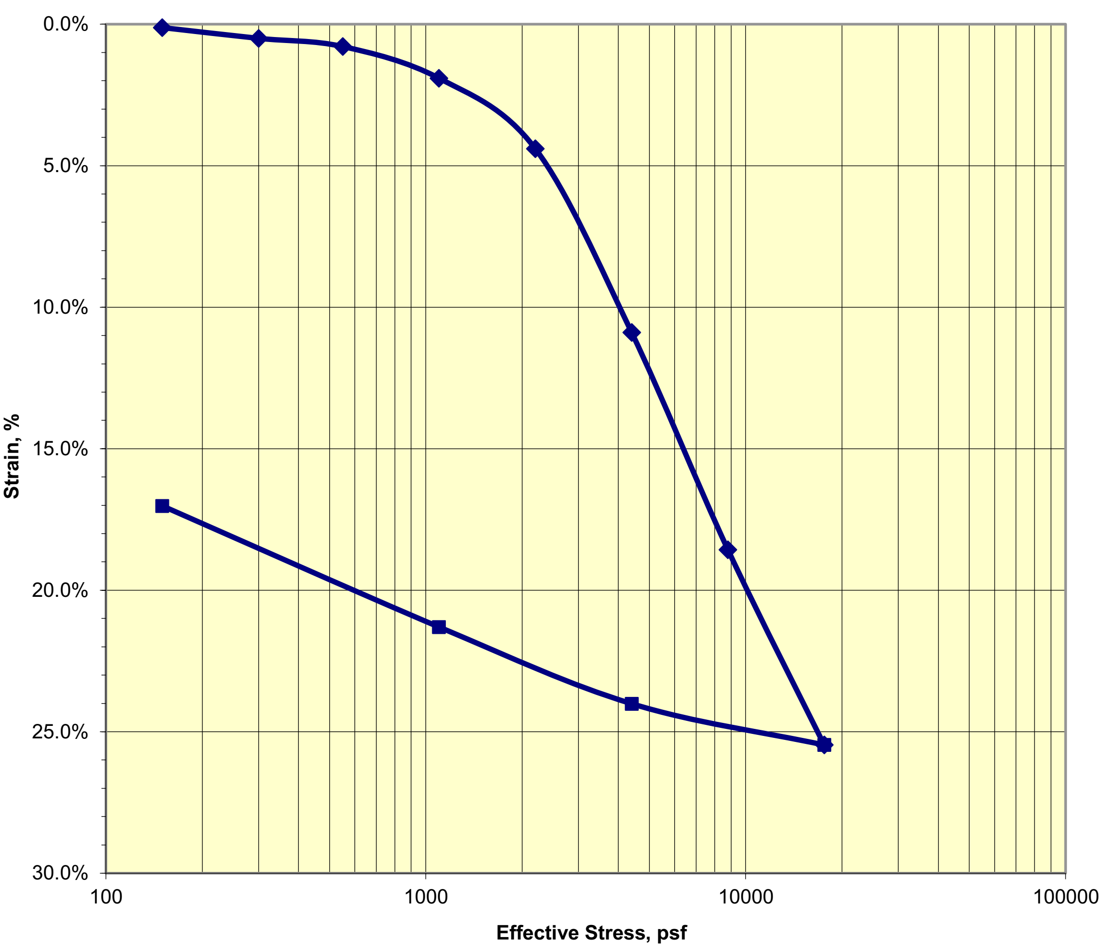
PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

CONSOLIDATION TEST DATA

DATE:	JOB NUMBER:	PLATE
June 2013	VAMMS-01-00	18

Consolidation Test
ASTM D2435

Job No.:	011-553	**Boring:**	B-7	**Run By:**	MD
Client:	BAGG	**Sample:**	4	**Reduced:**	PJ
Project:	VAMMS-01-00	**Depth, ft.:**	10(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Black Sandy CLAY w/ organics			**Date:**	5/24/2013



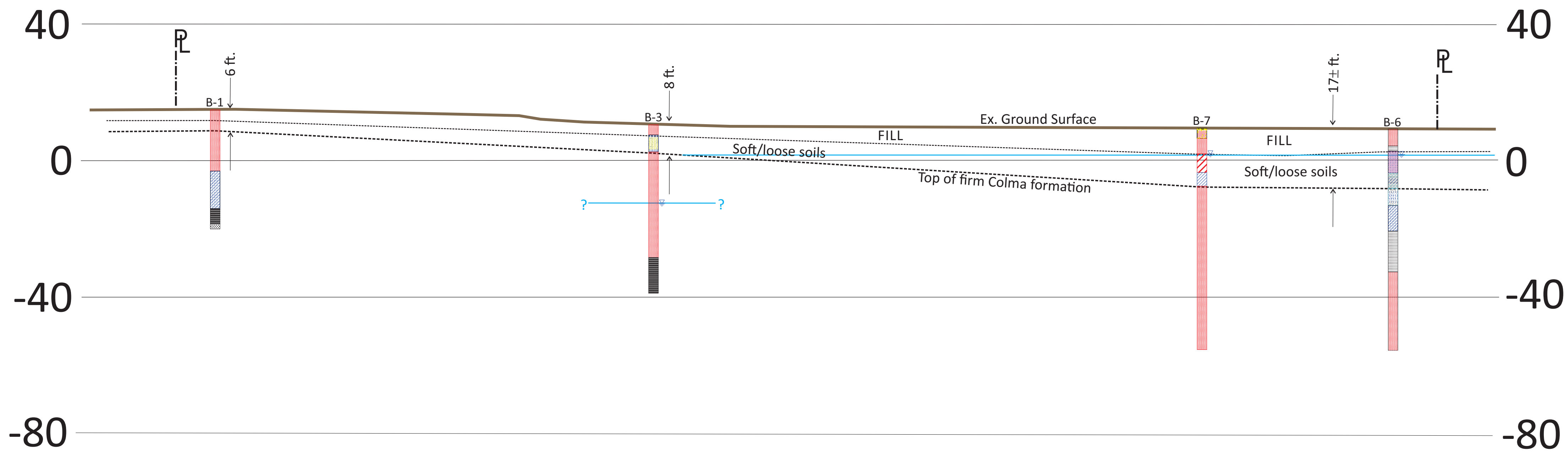
Ass. Gs = 2.65	Initial	Final
Moisture %:	57.4	42.7
Dry Density, pcf:	62.8	77.7
Void Ratio:	1.636	1.130
% Saturation:	93.0	100

Remarks:

PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

CONSOLIDATION TEST DATA

DATE:	JOB NUMBER:	PLATE
June 2013	VAMMS-01-00	19



SCALE:
HorizontaL: 1" = 80'
Vertical: 1" = 40'

Strata symbols

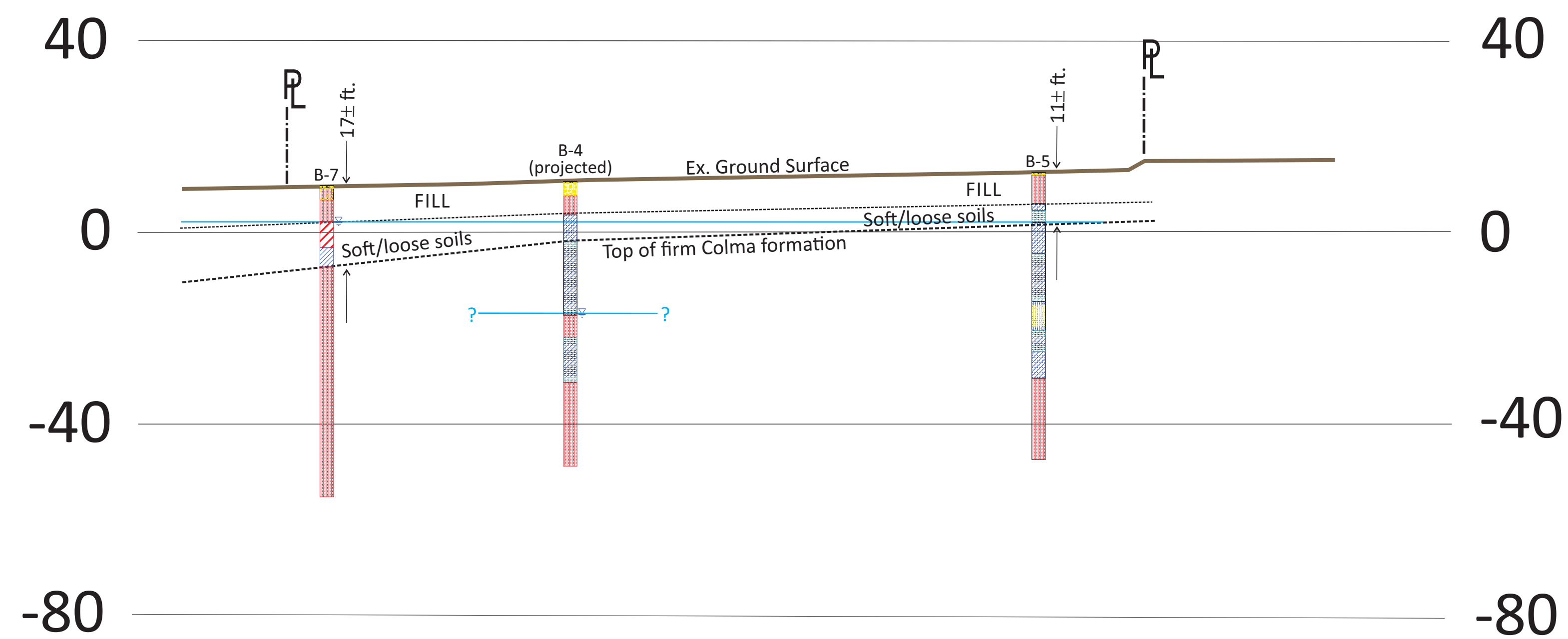
Paving	Shale	High plasticity (fat) clay
Silty gravel	Sandstone	Lean Clay
Sandy lean clay	Borderline silty sand to sandy silt	Poorly graded sand
Silty sand	Clayey sand	Borderline sandy lean clay to clayey sand
	Silty sand with gravel	Well graded sand with silt

PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

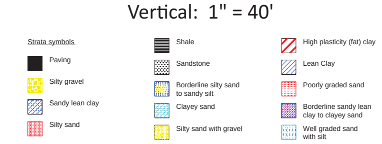
BAGG ENGINEERS

GENERALIZED CROSS SECTION A - A'

JOB NUMBER:	SCALE:	DATE	PLATE
VAMMS-01-00	1 inch =80 feet	June 2013	20



SCALE:
HorizontaL: 1" = 80'
Vertical: 1" = 40'



PROPOSED MILLBRAE STATION
EL CAMINO REAL @ MILLBRAE AVENUE
MILLBRAE, CALIFORNIA

BAGG ENGINEERS

GENERALIZED CROSS SECTION B - B'

JOB NUMBER:	SCALE:	DATE	PLATE
VAMMS-01-00	1 inch =80 feet	June 2013	21

Transit-Oriented Development #2 Project

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Millbrae BART Transit Oriented Development
LOCATION	Millbrae Avenue and El Camino Real Millbrae, California
CLIENT	Republic Urban Properties LLC
PROJECT NUMBER	183-5-2
DATE	June 13, 2014

CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Millbrae BART Transit Oriented Development**
Location	**Millbrae Avenue and El Camino Real** **Millbrae, California**
Client	**Republic Urban Properties LLC**
Client Address	**84 West Santa Clara Street, Suite 600** **San Jose, CA**
Project Number	**183-5-2**
Date	**June 13, 2014**

REGISTERED PROFESSIONAL ENGINEER
DANH T. TRAN
C59705
EXP. 12/31/15
CIVIL
STATE OF CALIFORNIA

Prepared by **Danh T. Tran, P.E.**
Senior Principal Engineer
Geotechnical Project Manager

C. Barry Butler, P.E., G.E.
Senior Principal Engineer
Quality Assurance Reviewer

TABLE OF CONTENTS

Type of Services	**Geotechnical Investigation**
Project Name	**Millbrae BART Transit Oriented Development**
Location	**Millbrae Avenue and El Camino Real** **Millbrae, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Republic Urban Properties LLC for the Millbrae Transit Oriented Development in Millbrae, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of renderings titled, "Millbrae TOD," including, "Aerial – Site Plan," and, "Birdseye - Looking SW," prepared by Robin Chiang & Company, dated June 7, 2013.

- A rendering titled, "Millbrae TOD," including, "Birdseye – Looking NW," prepared by Robin Chiang & Company, dated June 14, 2013.

- A document titled, "Millbrae T.O.D. - Conceptual Design Summary," prepared by The Republic Family of Companies, dated June 14, 2013.

- A plan set titled, "Republic at Millbrae Station: Specific Plan Amendment & Zoning Application," prepared by Republic Urban Properties, dated December 9, 2013.

1.1 PROJECT DESCRIPTION

The project will consist of a mixed-use, transit oriented development (TOD) located at Millbrae Avenue and Rollins Road in Millbrae, California. As shown in Figure 2, Site Plan, there are four areas where developments are planned. The approximately 8½ acre site will include Sites 5A, 5B, 6A and 6B.

Site 5A will consist of a new 6-story structure including 19,100 square feet of retail space and 121,400 square feet of commercial/office space. Floors 1 and 2 will include garage parking as well as retail space. The third floor will consist of garage parking only. Floors 4 through 6 consist of commercial/office space. This structure will be at-grade.

Site 5B will consist of a new 7-story structure including 45,780 square feet of retail space, 263 residential units, and a three level parking garage with one level below-grade parking for the south-east portion of the site.

Site 6A will consist of a new 3-story extended-stay hotel with 110 hotel units and 4,810 square feet of retail space. The at-grade parking lot to the north of the new hotel will total 383 parking spaces.

Site 6B will include a new 3-story structure, one story of commercial space over one story of double height retail. The space will include 15,200 square feet of retail space and 15,200 square feet of commercial/office space. The at-grade parking lot to the north of this structure will total 44 parking spaces.

Appurtenant utilities, landscaping and other improvements necessary for site development are also planned. Site grading is anticipated to include cuts on the order of 12 to 14 feet for the one-level below grade Site 5B parking garage. Cuts and fills are anticipated to be on the order of 5 feet for at-grade structures. Structural loads have not been provided at the time of this report but are expected to be typical for similar structures.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 28, 2013 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of six borings drilled on April 26, May 3, and May 4, 2014 with truck-mounted, hollow-stem auger drilling equipment and eleven Cone Penetration Tests (CPTs) advanced on April 26 and 27, 2014. The borings were drilled to depths of 50 to 80 feet; the CPTs were advanced to depths of 33 to 65¾ feet, where theoretical refusal was encountered. Seismic shear wave velocity measurements were collected from CPT-10. Two of the borings, EB-1 and EB-4 were advanced adjacent to CPT-2 and CPT-7, respectively, for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with San Mateo County requirements; exploration permits were obtained as required by San Mateo County.

The approximate locations of our exploratory borings and CPT's are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, washed sieve analyses, Plasticity Index tests, consolidation tests, and triaxial compression tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including a Phase 1 site assessment; environmental findings and conclusions are provided under a separate cover.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

San Francisco Bay is a northwesterly trending structural depression that lies along the boundary of the Pacific and North America tectonic plates. The Bay is within the Coast Ranges geomorphic province of California, which is characterized by a series of nearly parallel mountain ranges (Goldman, 1969). Active faults, including the San Andreas, Hayward, and Calaveras Faults, roughly parallel the western and eastern limits of the Bay. The Bay began forming during the Pleistocene Epoch, approximately 2 million years ago, the San Francisco-Marin block began to tilt eastward along the Hayward Fault. The eastern side of the block became a depression and filled with sediment and water.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2007 estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	1.9	3.1
San Gregorio	7.9	12.7
Monte Vista-Shannon	13.3	21.4

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

Site 5A is currently a bus drop-off area. Sites 5B, 6A and 6B are currently parking lots for the adjacent Bay Area Rapid Transit (BART) Station. The lots include parking stalls, drive aisles, and landscape areas. The overall site is relatively level but graded to drain to storm drainage facilities.

Surface pavements for the bus drop-off area generally consisted of 9 to 12 inches of Portland Cement Concrete (PCC) over 10 to 12 inches of aggregate base. Based on visual observation, the existing PCC pavements are in good condition. Surface pavements for the parking lot areas generally consisted of 3½ to 4 inches of asphalt concrete over 9 to 12 inches of aggregate base. Based on visual observations, the existing asphalt pavements are in poor condition with localized areas of alligator cracking.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations encountered existing undocumented fill underlain by young estuarine and older alluvial soils to the maximum depth explored of 80 feet. A more detailed description of the subsurface conditions is presented in the following sections.

3.2.1 Existing Undocumented Fill

Our borings encountered approximately 3½ to 9 feet of undocumented fill blanketing the site. Generally, the fill consisted of stiff to very stiff lean clay with variable amounts of sand, medium dense clayey sand, very dense silty sand, and very dense poorly graded gravel with clay and sand. Site 5A was blanketed by up to about 4½ to 8½ feet of undocumented fill. Site 5B had up to about 3½ to 6 feet of undocumented fill. Site 6A had approximately 7 to 9 feet of undocumented fill. Site 6B encountered approximately 4 to 6½ feet of undocumented fill.

Laboratory testing indicated that the in-situ moisture contents of the fill range from 2 to 6 percent over the estimated laboratory optimum moisture.

3.2.2 Bay Mud

The existing fill is underlain by estuarine deposits consisting of soft to medium stiff clay with variable amounts of fine sand, known locally as Bay Mud. The Bay Mud ranges in thickness from approximately 8½ to 28 feet and extends to depths ranging from 4 to 27 feet below existing grades. In-situ moisture contents of the Bay Mud encountered range from about 57 to 130 percent.

3.2.3 Older Alluvial Soils

The Bay Mud is generally underlain by older alluvial soils consisting of clayey sand, silty sand, poorly graded sand and sandy clay to 80 feet, the maximum depth explored for this investigation. These older alluvial soils, often referred to as Older Bay Clays, are generally over-consolidated and considered relatively incompressible.

Our CPT's encountered similar subsurface conditions to about 33 to 65¾ feet below existing site grades where we encounter refusal. Refusal occurred in mostly very dense silty sand or clayey sand.

3.2.4 Plasticity/Expansion Potential

We performed ten Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of the existing undocumented fill at the surface, and the plasticity of the fines in potentially liquefiable layers. The results of the surficial PI tests performed on the fill indicated PIs ranging from 12 to 20, indicating low to moderate expansion potential to wetting and drying cycles.

3.3 GROUND WATER

Ground water was encountered in our exploratory borings at depths ranging from 5½ to 30 feet below current grades. However, ground water is generally considered to be at or near the top of the Bay Mud, which we encountered at depths of 4½ to 9 feet below existing grade. Table 2 summarizes ground water depths encountered during our investigation and pertinent information. We recommend a design ground water depth of about 5 feet below the ground surface. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

In general, fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, tidal influence, regional fluctuations, and other factors.

Table 2: Depth to Ground Water

Boring Number	Date Drilled	Initial Depth to Ground Water (feet)	Final Depth to Ground Water (feet)	Depth of Boring (feet)
EB-1	4/26/2014	24	15	60
EB-2	5/4/2014	30	19	49½
EB-3	5/4/2014	22	15	80
EB-4	5/3/2014	7	7	70
EB-5	4/26/2014	23	10	50
EB-6	5/3/2014	5½	5½	60

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to $PGA_M = F_{PGA} \times PGA_G$ (Equation 11.8-1) as allowed in the 2013 California Building Code. For our liquefaction analysis we used a PGA of 0.88g.

4.3 LIQUEFACTION POTENTIAL

The site is currently not mapped for liquefaction potential by the State of California, but is within a zone mapped as having a moderate to very high liquefaction potential by the Association of Bay Area Governments (ABAG, 2011). Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 5 feet. Following the procedures in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation.

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the "Estimated Ground Shaking" section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are unreliable in sands below ground water. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers.

The results of our CPT analyses (CPT-1 through CPT-11) are presented on Figures 5A through 5K of this report.

4.3.3 Summary

For Site 5A, our analyses from CPT-1 and CPT-2 indicate that several layers could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement ranging from 2 to 2¼ inches based on the Yoshimine (2006) method. For Site 5B, our analyses from CPT-3 to CPT-6 indicate seismic settlements ranging from about ¾-inch to 1 inch. For Site 6A, our analyses from CPT-7 to CPT-9 indicate seismic settlements ranging from about ¼-inch to 1¼ inches. For Site 6B, our analyses from CPT-10 and CPT-11 indicate seismic settlements ranging from about ¼-inch to ½-inch.

As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement. In our opinion, differential settlements for Site 5A, Site 5B, Site 6A, and Site 6B are anticipated to be on the order of up to 1½-inch, ½-inch, ¾-inch, and ¼-inch between independent foundation elements, or over a horizontal distance of about 30 feet, respectively.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 11½-foot thick non-liquefiable cap is sufficient to prevent ground rupture; therefore the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at the exploration location.

A drainage right of way connected to the San Francisco Bay forms the northern project boundary for Site 6A. The depth of the concrete-lined channel is approximately 7 feet. Our analysis of CPT's performed on Site 6A resulted in a negligible LDI; therefore, we judge that lateral spreading is not a significant geologic hazard for the proposed improvements.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. The potential for differential seismic settlements of the fills above the ground water table are considered high and mitigation measures for this will be discussed further in the following sections of report.

4.6 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately ½ mile inland from the San Francisco Bay shoreline, and is approximately 8 to 13 feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, described as, "Areas determined to be outside the 0.2% annual chance floodplain." We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The Association of Bay Area Governments has compiled a database of Dam Failure Inundation Hazard Maps (ABAG, 1995). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is not located within a dam failure inundation area.

SECTION 5: BAY MUD SETTLEMENT

The existing fill at the site is underlain by up to approximately 8½ to 28 feet of highly compressible Bay Mud. Analyses were performed to estimate the potential settlement associated with foundation loads from the new improvements. The analysis assumed a design period of 30 years commencing with the completion of construction.

Our settlement analysis was based on the site history, recent laboratory data, and proposed construction. Given the site history, we assumed the Bay Mud was normally to over-consolidated under the weight of the existing fill and prior to introducing foundation loads.

We assumed average contact pressures ranging from approximately 1,200 to 1,250 pounds per square foot for the proposed building foundations. The results of our settlement analyses indicate that approximately 7¾ to 8½ inches of total consolidation settlement will occur due to building loads for Site 5A. Our analyses for Sites 5B, 6A, and 6B indicate that consolidation settlement due to building loads will total approximately 7, 10½, and 10½ inches, respectively. In addition to the consolidation settlement, we estimate that up to about 1-inch of immediate settlement will occur across the site. Differential settlement across the sites are estimated to range from about 5¼ to 7¾ inches between foundation elements.

A more detailed discussion regarding the potential impacts to the proposed development is presented in the "Conclusions" section of this report.

SECTION 6: CONCLUSIONS

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Due to the highly variable subsurface conditions present between the different Sites, our discussion and recommendations for each Site are discussed separately. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

6.1 SITE 5A

Our outline and recommendations of each geotechnical concern for Site 5A are discussed below.

- Potential for significant static and seismic settlements
- Presence of undocumented fill
- Differential movement from exterior grades to structure

6.1.1 Static and Seismic Settlement

A geotechnical concern is the potential for substantial differential settlement to occur due to long-term settlement of the highly compressible Bay Mud immediately underlying the existing undocumented fill materials. Site 5A is underlain by 8½ to 12 feet of highly compressible Bay Mud. The Bay Mud will undergo settlement due to the weight of any new fill or building loads. Differential settlement is anticipated to occur in areas where the thickness of any new fill, existing fill, or building loads vary abruptly, or where the thickness of the Bay Mud varies significantly over a short horizontal distance. Finish grading plans and structural systems of the proposed structures should be designed to avoid abrupt grade changes and irregular concentration of building loads to reduce differential settlement. If feasible, we recommend that proposed site grades remain as close to existing grades as possible to minimize post-construction settlement.

As discussed in Section 5, we estimated approximately 7¾ to 8½ inches of total consolidation settlement will occur due to the estimated building loads for Site 5A. In addition, approximately 1-inch of immediate settlement can be expected, resulting in differential settlement of up to 6½ inches.

As discussed in Section 4.3, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of 2 to 2¼ inches could occur, resulting in differential settlement up to 1½ -inches.

To mitigate the effects of the anticipated differential settlements, we recommend that the structure be supported on rigid shallow foundations overlying ground improvement, or on deep foundations. Foundations should be designed to resist any expected differential settlements after ground improvement. In addition, gravity flow utilities should be designed to account for any future settlement to avoid grade reversal or leakage from joint separation. A discussion of mitigation options and detailed foundation recommendations are presented in the "Foundations" section of this report.

6.1.2 Presence of Undocumented Fill

Our borings encountered between 4½ to 8½ feet of undocumented fill in Borings EB-1 and EB-5. We estimated that the fill in this area could experience up to ½-inch of dry sand settlement during a seismic event. As noted above, the building will be supported on shallow foundation over ground improvement. We recommend the ground improvement system also be used to mitigate the undocumented fill and the potential for dry sand settlement. Additional recommendations follow.

6.1.3 Differential Movement from Exterior Grades to Structure

The structure at Site 5A will be supported on a rigid mat foundation over ground improvement or on deep foundations. Exterior site grades and improvements supported on-grade, including buried utilities, are expected to be subject to long-term settlement. As a result, significant differential movement may occur between exterior improvements and structures. The following items will need to be considered in design to avoid significant distress.

- Concrete flatwork at building entrance/exits should be structurally tied to the structure, creating hinged connections, to allow access and limit trip hazards.

- Where utilities transition to the structure, flexible utility connections or other types of mitigation may be necessary to prevent damage or disruption of utilities.

- Perimeter grade beams should be deep enough to avoid exposure of the bottom of the structure due to long-term settlement. This can be an eye sore, and sometimes worrisome to the occupants.

6.2 SITE 5B

Our outline and recommendations of each geotechnical concern for Site 5B are discussed below.

- Potential for significant static and seismic settlements
- Presence of undocumented fill
- Differential movement between structures with different foundation types
- Differential movement at on-grade to on-structure transitions

6.2.1 Static and Seismic Settlements

Site 5B is underlain by 6½ to 11½ feet of highly compressible Bay Mud. The Bay Mud will undergo settlement due to the weight of any new fill or building loads. Differential settlement is anticipated to occur in areas where the thickness of any new fill, existing fill, or building loads vary abruptly, or where the thickness of the Bay Mud varies significantly over a short horizontal distance. Finish grading plans and structural systems of the proposed structures should be designed to avoid abrupt grade changes and irregular concentration of building loads to reduce differential settlement. If feasible, we recommend that proposed site grades remain as close to existing grades as possible to minimize post-construction settlement.

As discussed in Section 5, we estimated up to 7 inches of total consolidation settlement will occur for the at-grade structures due to the estimated building loads for Site 5B. In addition, approximately 1-inch of immediate settlement can be expected, resulting in differential settlement of up to 5½ inches between foundation elements.

As discussed in Section 4.3, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of ¾-inch to 1 inch could occur, resulting in differential settlement of up to ½-inch.

To mitigate the effects of the anticipated differential settlements, we recommend the structure over the one-level underground basement portion (southern) be supported on conventional rigid mat foundations. The at-grade portion of the structures (northern) should also be supported on rigid mat foundations overlying ground improvement. As an alternative, both, Sites 5A and 5B can also be supported on deep foundations. The rigid mat foundations should be designed to resist any expected differential settlements after ground improvement. In addition, gravity flow utilities should be designed to account for any future settlement to avoid grade reversal or leakage from joint separation. A discussion of mitigation options and detailed foundation recommendations are presented in the "Foundations" section of this report.

6.2.2 Presence of Undocumented Fill

Our borings and CPT's encountered between 3½ to 7½ feet of undocumented fill. In our opinion, the potential for seismic settlement of the existing undocumented fill is low. Since the proposed structures over the one-level underground basement will excavate the undocumented fill and the at-grade structures will be supported on rigid mat foundation overlying ground improvement or deep foundations, in our opinion, the existing fill material will have minimal impact to the structures.

6.2.3 Shallow Ground Water

We anticipate that shallow ground water exists at depths ranging from approximately 5 to 7½ feet below the existing ground surface. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, difficult underground utility installation, and difficulty installing shoring. Dewatering and shoring of utility trenches may be required to construct the planned basement as well as deeper utilities. The basement will also likely need to be designed for hydrostatic pressure on walks and floors, and waterproofing. Additional recommendations addressing this concern are presented in the "Earthwork" section of this report.

6.2.5 Differential Movement At On-grade to On-Structure Transitions

The landscaped courtyard and other improvements will transition from on-grade support to overlying the basement. Where the depth of soil cover overlying the basement roof in the courtyard area is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that sub-slabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see "Retaining Wall" section).

For the at-grade structures, exterior grades and improvements supported on-grade, including buried utilities are expected to be subject to long-term settlement. As a result, significant differential movement will occur between exterior improvements and structures. The following items will need to be considered in design to avoid significant distress.

- Concrete flatwork at building entrance/exits should be structurally tied to the structure, creating hinged connections, to allow access and limit trip hazards.

- Where utilities transition to the structure, flexible utility connections or other types of mitigation may be necessary to prevent damage or disruption of utilities.

- Perimeter grade beams should be deep enough to avoid exposure of the bottom of the structure due to long-term settlement. This can be an eye sore, and sometimes worrisome to the occupants.

6.3 SITE 6A

Our outline and recommendations of each geotechnical concern for Site 6A are discussed below.

- Potential for significant static and seismic settlements
- Presence of undocumented fill
- Differential movement from exterior grades to structure
- Shallow Ground Water

6.3.1 Static and Seismic Settlements

Site 6A is underlain by 15 to 28 feet of highly compressible Bay Mud. The Bay Mud will undergo settlement due to the weight of any new fill or building loads. Differential settlement is anticipated to occur in areas where the thickness of any new fill, existing fill, or building loads vary abruptly, or where the thickness of the Bay Mud varies significantly over a short horizontal distance. Finish grading plans and structural systems of the proposed structures should be designed to avoid abrupt grade changes and irregular concentration of building loads to reduce differential settlement. If feasible, we recommend that proposed site grades remain as close to existing grades as possible to minimize post-construction settlement.

As discussed in Section 5, we estimated up to 10½ inches of total consolidation settlement will occur due to the estimated building loads for Site 6A. In addition, approximately 1-inch of immediate settlement can be expected, resulting in differential settlement of up to 7¾ inches.

As discussed in Section 4.3, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of ¼-inch to 1¼ inches could occur, resulting in additional differential settlement of up to ¾-inch.

To mitigate the effects of the anticipated differential settlements, we recommend that the structure be supported on deep foundations. Gravity flow utilities should be designed to account for any future settlement to avoid grade reversal or leakage from joint separation. A discussion of mitigation options and detailed foundation recommendations are presented in the "Foundations" section of this report.

6.3.2 Presence of Undocumented Fill

As discussed, our boring and CPT's encountered between 7 to 9 feet of undocumented fill in the area. We estimated that the fill in this area could experience up to 2 inches of dry sand settlement during a seismic event. As noted above, the building will be supported on a deep foundation system. Therefore, the undocumented fill is expected to have minimal impact to the structure. However, at-grade improvements including hardscape and pavement areas can experience significant movements. We recommend the upper 3 feet of undocumented fill (beneath finished subgrade) be over-excavated and replaced as engineered fill. Prior to fill placement, the bottom of excavations should be scarified a minimum of 12 inches, moisture conditioned, and compacted. A minimum 20-ton heavy-duty vibratory drum roller should be used to compact the bottom of the building pad over-excavation and subsequent fill lifts.

6.3.3 Differential Movement from Exterior Grades to Structure

The structure at Site 6A will be supported on deep foundations, and exterior grades and improvements supported on-grade, including buried utilities, are expected to be subject to long-term settlement. As a result, significant differential movement will occur between exterior improvements and structures. The following items will need to be considered in design to avoid significant distress.

- Concrete flatwork at building entrance/exits should be structurally tied to the structure, creating hinged connections, to allow access and limit trip hazards.

- Where utilities transition to the structure, flexible utility connections or other types of mitigation may be necessary to prevent damage or disruption of utilities.

- Perimeter grade beams should be deep enough to avoid exposure of the bottom of the structure due to long-term settlement. This can be an eye sore, and sometimes worrisome to the occupants.

6.3.4 Shallow Ground Water

Shallow ground water was measured at a depth of approximately 7 feet below the existing ground surface. We anticipate that ground water exists at depths ranging from approximately 5 to 7 feet below the existing ground surface. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

6.4 SITE 6B

Our outline and recommendations of each geotechnical concern for Site 6B are discussed below.

- Potential for significant static and seismic settlements
- Presence of undocumented fill
- Differential movement from exterior grades to structure
- Shallow Ground Water

6.4.1 Static and Seismic Settlement

Site 6B is underlain by 15½ to 22 feet of highly compressible Bay Mud. The Bay Mud will undergo settlement due to the weight of any new fill or building loads. Differential settlement is anticipated to occur in areas where the thickness of any new fill, existing fill, or building loads vary abruptly, or where the thickness of the Bay Mud varies significantly over a short horizontal distance. Finish grading plans and structural systems of the proposed structures should be designed to avoid abrupt grade changes and irregular concentration of building loads to reduce differential settlement. If feasible, we recommend that proposed site grades remain as close to existing grades as possible to minimize post-construction settlement.

As discussed in Section 5, we estimated up to 10½ inches of total consolidation settlement will occur due to the estimated building loads for Site 6B. In addition, approximately 1-inch of immediate settlement can be expected, resulting in differential settlement of up to 7¾ inches.

As discussed in Section 4.3, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of ¼ to ½-inch could occur, resulting in differential settlement up to ¼-inch between foundation elements.

To mitigate the effects of the anticipated differential settlements, we recommend that the structure be supported on shallow foundations overlying ground improvement or on deep foundations. Foundations should be designed to resist any expected differential settlements after ground improvement. In addition, gravity flow utilities should be designed to account for any future settlement to avoid grade reversal or leakage from joint separation. A discussion of mitigation options and detailed foundation recommendations are presented in the "Foundations" section of this report.

6.4.2 Presence of Undocumented Fill

Our Boring EB-6 encountered about 6½ feet of undocumented fill. Since the proposed structure for Site 6B will be supported on a rigid mat foundation overlying ground improvement or deep foundations, the existing fill material will have minimal impact. However, at-grade improvements including hardscape and pavement areas can experience significant movements. We recommend the upper 3 feet of undocumented fill (beneath finished subgrade) be over-excavated and replaced as engineered fill. Prior to fill placement, the bottom of excavations should be scarified a minimum of 12 inches, moisture conditioned, and compacted. A minimum 20-ton heavy-duty vibratory drum roller should be used to compact the bottom of the building pad over-excavation and subsequent fill lifts in improvement and pavement areas.

6.4.3 Differential Movement from Exterior Grades to Structure

The structure at Site 6B will be supported on a rigid mat foundation over ground improvement or deep foundations, and exterior grades and improvements supported on-grade, including buried utilities, are expected to be subject to long-term settlement. As a result, significant differential movement will occur between exterior improvements and structures. The following items will need to be considered in design to avoid significant distress.

- Concrete flatwork at building entrance/exits should be structurally tied to the structure, creating hinged connections, to allow access and limit trip hazards.

- Where utilities transition to the structure, flexible utility connections or other types of mitigation may be necessary to prevent damage or disruption of utilities.

- Perimeter grade beams should be deep enough to avoid exposure of the bottom of the structure due to long-term settlement. This can be an eye sore, and sometimes worrisome to the occupants.

6.4.4 Shallow Ground Water

Shallow ground water was measured at a depth of approximately 5½ feet below the existing ground surface. We anticipate that ground water exists at depths ranging from approximately 5 to 7 feet below the existing ground surface. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the "Earthwork" section of this report.

6.5 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

6.6 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 7: EARTHWORK

7.1 SITE DEMOLITION, CLEARING AND PREPARATION

We have provided general guidelines for earthwork construction and highlighted some of the more difficult aspects of earthwork on sites underlain by Bay Mud in Appendix D, Construction Guidelines on Bay Mud.

7.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Demolition of existing improvements is discussed in detail below. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

7.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

7.1.3 Demolition of Existing Slabs, Foundations and Pavements

All slabs and pavements should be completely removed from within planned building areas. Slabs and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to provide subsurface drainage. A discussion of recycling existing improvements is provided later in this report.

7.1.4 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

7.2 REMOVAL OF EXISTING FILLS

As previously discussed, varying thicknesses of undocumented fill blankets the site. Recommendations for removal of undocumented fill for each site are provided in Section 6. Where fill removal is required, fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

We recommend the upper 3 feet of undocumented fill (beneath finished subgrade) be over-excavated and replaced as engineered fill. Prior to fill placement, the bottom of excavations should be scarified a minimum of 12 inches, moisture conditioned, and compacted. A minimum 20-ton heavy-duty vibratory drum roller should be used to compact the bottom of the building pad over-excavation and subsequent fill lifts in improvement and pavement areas.

7.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. Recommended soil parameters for temporary shoring are provided in the "Temporary Shoring" section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be reviewed if temporary slopes are to be used due to the weak, soft underlying Bay Mud, which is subject to slope failure.

Support of excavation and trench walls in Bay Mud may be accomplished using sheet piles, shoring, or an equivalent method. This choice should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is most appropriate. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. Trench shields should not be allowed in excavations unless special placement methods are used and approved by the geotechnical engineer. All excavations should be checked by contractor for stability and base heave.

In general, the contractor should be responsible for all temporary trenches and excavations at the site and design of any required temporary shoring. Support of adjacent existing roadways or other improvements without distress should also be the contractor's responsibility. We recommend that the contractor forward plans for the above support systems to the structural engineer and geotechnical engineer for review prior to construction.

Improper shoring and delays in construction could result in movement of the excavation bottom, often referred to as base heave. Slope excavations for manholes extending into Bay Mud may also experience deep-seated movement near the base of the excavation that may not be visible or observable during construction. To help reduce the potential for base heave, once pipe has been placed in an excavation, the trench should be backfilled the same day.

If care is taken during excavation, shoring, pipe placement, backfilling of the excavation and removal of the shoring, we do not anticipate circumstances or conditions that would adversely affect the long-term performance of the pipeline. However, lack of attention to detail, especially during removal of shoring during the trench backfilling process, could result in creation of voids in the soil or other conditions that could adversely affect the long-term performance of the pipeline.

We recommend that utilities with trench backfill extending into Bay Mud be designed to balance stresses with the removed soil to avoid inducing additional stresses in the underlying Bay Mud. For deeper utilities, the use of lightweight backfill may be required. Replacing excavated Bay Mud with heavier trench backfill material may result in additional local settlement, potentially causing a reversal in flows or sags in gravity lines.

7.4 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the "Temporary Cut and Fill Slopes" section above if space allows. Alternatively, temporary shoring may support the planned cuts up to 15 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

7.4.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, or potentially other methods. Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing and new structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 3: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	40 pcf
Restrained Wall – Trapezoidal Earth Pressure	Increase from 0 to 25H* psf
Passive Pressure – Starting at 2 feet below the bottom of the excavation	400 pcf up to 2,000 psf maximum uniform pressure

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

The restrained earth pressure may also be distributed as described in Figure 24 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the "Wall Drainage" section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam or tie-back installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) or difficult drilling or cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. The installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

7.4.2 Construction Dewatering

Ground water levels are expected to be as high as about 5 feet below the existing ground surface depending on the time of year and seasonal weather influence; therefore temporary dewatering will likely be necessary during construction. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain ground water at least 5 feet below the bottom of the mass excavation, and at least 2 feet below localized excavations such as deepened footings, elevator shafts, and utilities. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered.

Temporary draw down of the ground water table can cause subsidence outside the excavation area, causing settlement of adjacent improvements. The settlement of adjacent improvements should be considered during dewatering design.

Depending on the ground water quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

7.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below.

Due to the sandy soils likely to be encountered at the subgrade elevation, we suggest recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

7.6 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

7.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

7.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

If Bay Mud is encountered at the bottom of the over-excavation area, then a woven geotextile fabric such as Mirafi 600X (or equivalent) should be used instead of geogrid to reduce the potential for fines migration. Subsequent material should be placed and compacted with lightweight earthwork equipment only.

7.6.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

7.6.4 Below-Grade Excavation Stabilization

As the planned basement excavation will extend below the current ground water level, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade, place a layer of stabilization fabric (Mirafi 500X, or equivalent) at the bottom, and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

7.7 MATERIAL FOR FILL

7.7.1 Re-Use of On-site Soils

Excavated Bay Mud should not be used as engineered fill. Bay Mud may not be suitable for use as landscape soil due to its marine origin and generally high sulfate content. Bay Mud encountered during excavating or grading should be segregated from the fill such that the drier fill material is not mixed with wet, soft Bay Mud.

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

7.7.2 Re-Use of On-Site Site Improvements

We anticipate that significant quantities of asphalt concrete (AC) grindings and aggregate base (AB) and Portland Cement Concrete (PCC) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections, including within below-grade parking garage slab-on-grade areas (provided crushed rock is not required due to the proximity to ground water). AC/AB grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications. Due to the existing alligator cracking of the AC pavements, it is likely that the grinding operation will leave significant oversize chunks and won't meet the Class 2 AB gradation requirements but may meet Caltrans subbase requirements. Depending on the quantities of oversized material, the grindings may still be used within the structural section; however, the pavement design will need to be modified to account for the difference, typically resulting in the addition of about 1 inch to the structural section.

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the "Material for Fill" requirements of this report, it may be used as select fill within the habitable building areas, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

7.7.3 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

7.8 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report.

Table 4: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	>1

Table 4 Continues

Table 4 Continues

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using light-weight compaction or walls should be braced

7.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements. The underlying Bay Mud should not be used as general fill material.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Bay Mud underground utility construction means and methods are generally left up to the contractor. Because trench excavations will likely expose soft weak Bay Mud, the contractor will most likely need to shore or shield the trench excavation. In addition, dewatering will likely be necessary.

7.10 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

7.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- The site has a known geotechnical hazard consisting of soils subject to liquefaction and highly compressible Bay Mud; therefore, stormwater infiltration facilities may not be feasible.

- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

- Infiltration devices are not approved as a stand-alone measure for treating stormwater runoff from land uses that pose a high threat to water quality, including but not limited to industrial and light industrial activities, high vehicular traffic (i.e., 25,000 or greater average daily traffic on a main roadway or 15,000 or more average daily traffic on any intersecting roadway), automotive repair shops, car washes, fleet storage areas, or nurseries (per Provision C.3.d.iv.(2)(d)).

- Infiltration devices should be located at least 100 feet away from septic tanks and underground storage tanks with hazardous materials, as well as any other potential underground sources of pollution.

- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

- Local Water District policies or guidelines may limit locations where infiltration may occur, require greater separation from seasonal high groundwater, or require greater setbacks from potential sources of pollution.

7.11.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

7.11.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the sides and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

7.11.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.
- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.
- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.
- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.
- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.
- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.
- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12 inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.
- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

7.11.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the "Retaining Walls" section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

7.12 LANDSCAPE CONSIDERATIONS

Landscaping fill berms may adversely affect the development by contributing to differential settlements adjacent to structures and pavements. We should review the landscape plans to identify potential settlement concerns and, if needed, provide supplemental recommendations.

Due to the potential for water perching above the Bay Mud, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation,

- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes,

- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers, and

- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion and due to expected high settlement estimates from liquefaction and compressible soil, the proposed structures should be supported on the appropriate foundations outlined the following sections of this report.

8.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings, shear wave velocity in CPT-10, and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot and an average shear wave velocity of 790 feet per second (241 meters per second). In addition, liquefaction settlement and settlement of the Bay Mud will be mitigated by ground improvement or use of deep foundations. Therefore, we have classified the following sites as Site Class D:

- Site 5A
- Site 5B
- Site 6B

The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Seismic Design Maps*, revision date March 12, 2014, based on the site coordinates presented below and the site classification. Table 5 lists the various factors used to determine the seismic coefficients and other parameters.

Table 5: CBC Site Categorization and Site Coefficients – Sites 5A, 5B, and 6B Only

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.60089°
Site Longitude	-122.38474°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.275g
1-second Period Mapped Spectral Acceleration[1], S_1	1.089g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.275g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.633g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.516g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.089g

[1]For Site Class B, 5 percent damped.

Site 6A should be Site Class E and the design values shown in Table 6 should be used.

Table 6: CBC Site Categorization and Site Coefficients – Site 6A Only

Classification/Coefficient	Design Value
Site Class	E
Site Latitude	37.60089°
Site Longitude	-122.38474°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.275g
1-second Period Mapped Spectral Acceleration[1], S_1	1.089g
Short-Period Site Coefficient – Fa	0.900
Long-Period Site Coefficient – Fv	2.400
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.047g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	2.613g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.365g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.742g

[1]For Site Class B, 5 percent damped.

8.3 REINFORCED MAT FOUNDATION – SITE 5B

8.3.1 Reinforced Concrete Mat Foundations – Site 5B Below-Grade (Southern)

The southern portion of the proposed building on Site 5B with a one-level of underground garage may be supported on reinforced mat foundations bearing on natural soil or engineered fill prepared in accordance with the "Earthwork" section of this report, and designed in accordance with the recommendations below provided all the Bay Mud underlying the mat foundation is removed in its entirety. Variation in the Bay Mud thickness should be expected and planned for.

Structural loads were not available at the time of this report. Therefore we have estimated an average areal pressure of 1,000 pounds per square foot (psf) for the portion of the building located over the one-level underground garage. For design, we recommend maximum allowable localized bearing pressures be limited to 2,500 psf at walls and column locations. This reduced allowable bearing pressure is intended to minimize differential movement between the below-grade portion of the buildings to the at-grade portions. When evaluating wind and seismic conditions, the above allowable bearing pressures can be increased by one-third. These pressures are net values; the weight of the mat may be neglected.

The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the mat foundation pad prior to placement of reinforcing steel.

8.3.2 Mat Foundation with Ground Improvement – Site 5B At-Grade (Northern)

The northern portion of Site 5B will be at-grade. As a result, the underlying Bay Mud will not be removed. Therefore and to mitigate the expected settlement, we recommend that portion of the building be supported on a mat foundation overlying ground improvement. Ground improvement recommendations are provided in Section 8.4 below.

8.3.3 Mat Foundation Settlement

For the foundation constructed at the basement level (Site 5B southern portion), we assumed finished floor is at 12 feet below the ground surface and the bottom of mat 15 feet below the ground surface. Based on the assumed areal pressures, we estimate the total settlement for the basement level mat foundation due to static loading will be on the order of ¾-inch across the mat and the total post-construction differential movement should be about ½-inch from the mat center to its edges from static loading. We recommend the evenly split building (southern-most building on Site 5B) be evaluated further once the foundation plans and structural loads become available.

In addition, the reinforced mat foundation should be designed to accommodate an estimated 1-inch of seismic differential movement due to liquefaction. Accounting for both liquefaction-induced and differential settlements, we recommend the mat be designed to tolerate a total differential movement of approximately 1½ inch. If the mat is not capable of resisting such differential movement, additional reinforcement or increased mat thickness may be required. Otherwise, a deep foundation system or ground improvement may also need to be considered to support the structures.

In addition, gravity flow lines should be designed to account for any future settlement to avoid grade reversal or leakage from joint separation.

We should be retained to review the final loading and verify the settlement estimates above once the project plans are finalized.

8.3.4 Mat Modulus of Soil Subgrade Reaction

We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE, or similar, analysis for the mat. Preliminary modulus of subgrade reaction values for the initial analysis is provided below.

As discussed above we estimated an average areal pressure of about 1,000 psf for the proposed structures at Site 5B over one-level underground garage. Based on this pressure, we estimate a preliminary modulus of subgrade reaction of 20 pci be used toward the center portion of the mat. As discussed above, this modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design.

8.3.5 Mat Foundation Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above.

8.3.6 Mat Foundation Construction Considerations

The excavation areas should be cut neat to grade and be undisturbed with no rubber-tire equipment being used within three feet of the bottom of the excavations. If sandy and wet soils are encountered at the subgrade elevation, we recommend that consideration be given to pouring a 2 to 3 inch thick rat slab or a section of crushed rock to protect subgrade. Please refer to the "Subgrade Stabilization Measures" Section for additional recommendations if subgrade is not stable.

8.3.7 Hydrostatic Uplift and Waterproofing

Where portions of the structures extend below the design ground water level of 5 feet below the current site grade, including the bottom of the mat foundations, they should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design ground water should be waterproofed and designed to resist hydrostatic pressure for the full wall height below design ground water. Where portions of the walls extend above the design ground water level, a drainage system may be added as discussed in the "Retaining Wall" section and the walls will not need to be designed for the hydrostatic pressures for the full wall height. If drainage is not provided above the design ground water level then the entire wall should be designed to resist hydrostatic pressures for the full wall height.

In addition, the portions of the structures extending below design ground water should be waterproofed to limit moisture infiltration, including mat foundation/thickened slab areas, all construction joints, and any retaining walls. We recommend that a waterproof specialist design the waterproofing system.

8.4 SHALLOW FOUNDATION OVER GROUND IMPROVEMENT

As discussed in Section 6, we recommend that the remaining at-grade structures for Site 5A and Site 6B be supported on conventional spread footings overlying ground improvement meeting the requirements of this report. The ground improvement could also be extended to protect any other critical improvements if desired. In addition, the mat foundation for the northern portion of Site 5B (at-grade) will also be constructed over ground improvement as recommended herein.

8.4.1 Spread Footings

Provided ground improvement is performed, footings should extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following:

- bottom of the adjacent interior slab-on-grade, or
- finished exterior grade, excluding landscaping topsoil.

Based on our experience with similar ground improvement projects, we estimate spread footings may be designed for a maximum allowable bearing pressure on the order of 4,000 to 6,000 psf for combined dead plus live loads. The above estimates should be evaluated further once foundations loads become available and a design-build ground improvement contractor has been chosen.

8.4.2 Settlement

As discussed in the "Ground Improvement" section below, the ground improvement design should be such that the total foundation settlement (static and seismic) are reduced to less than about 1½ inches with no more than about 1-inch for either the static or seismic components. Differential settlements should be limited to less than about 1-inch.

8.4.3 Ground Improvement

As discussed above, shallow foundations may be used overlying ground improvement provided total and differential settlements are reduced to more tolerable levels. At a minimum, ground improvement is recommended for the at-grade buildings within Site 5A, Site 5B (northern, at-grade portion with mat foundation), and Site 6B.

Ground improvement, such as impact piers, stone columns, soil-cement mixing, or other similar methods, should be designed to provide vertical support through the existing site fills and Bay Mud, as well as partial mitigation of the liquefaction potential. Ground improvement should provide adequate confining improvement around all foundations. We anticipate that the ground improvement construction will be a design-build process where Cornerstone Earth Group will review preliminary design-build ground improvement designs, including proposed spacing and layout relative to the foundation plans and installation lengths, and anticipated densification improvement of the surrounding soils prepared by prospective contractors, provide comments, and come to a general agreement with the contractor on the intended design approach.

The intent of the ground improvement design would be to increase the density of the potentially liquefiable sands and undocumented fills by laterally displacing and/or densifying the existing in-place soils. The degree to which the density is increased will depend on the improvement method and spacing. In addition to increasing the density, the methods listed above could provide an additional increase in bearing capacity and soil stiffness at the individual improvement locations, which could be taken into consideration during evaluation of the post-construction consolidation settlements. The proposed ground improvements should underlie all footings and be uniformly distributed beneath the slabs and mat foundations.

We recommend the ground improvement contractor's design include, but not be limited to:

- drawings showing the ground improvement layout, spacing and column diameter,
- the foundation layout plan,
- proposed ground improvement length (depth), and
- top and bottom elevations.

We should be retained to review the ground improvement contractor's plan and settlement estimates and analysis methods prior to construction.

Ground improvement would generally be constructed as follows:

- clear the site of existing demolition debris,
- grade site to rough grades,
- install the ground improvement on the approved layout, and
- excavate the upper two to four feet and replace as engineered fill to repair disturbance to the near-surface soils resulting from ground improvement installation

8.4.4 Ground Improvement Performance Testing

Performance testing typically consists of a pre-construction test section with post-installation CPT testing to confirm that the necessary composite soil strength increases were achieved to meet the settlement criteria. Post-installation CPT testing is also required during production installation. We should observe and monitor installation of the ground improvement on a full-time basis and review the post-installation settlement analyses provided by the contractor.

8.4.5 Alternative Foundation

As an alternative to spread footings with ground improvement or if the estimated settlements exceed the structural requirements, the above referenced structures can also be supported on deep foundations as recommended in Section 8.5 below.

8.5 DEEP FOUNDATIONS – SITE 6A

8.5.1 Displacement Augercast Piles

We recommend the proposed at-grade hotel at Site 6A be supported on drilled, cast-in-place, displacement augercast piles (APGD piles). As noted above and as an alternative to spread footings over ground improvement, APGD piles can also be used as foundation support for structures on Sites 5A, 5B, and 6B.

Augercast piles are concrete piles that are cast in place using a hollow-stem auger that drills to the design depth and then the sand-cement grout (4,000 to 6,000 psi grout) is pumped through the hollow-stem as the drill stem is extracted. Two types of augercast piles are available: APG piles, which like piers, remove the soil column and replace it with grout; and APGD piles, which displace the soil column as the drill stem is advanced, similar to driven piles, prior to pumping the grout. Augercast piles are a low noise and vibration installation compared to driven piles. Various types of steel reinforcing, including rebar cages or H-piles may be installed into the still-wet grout after drilling to satisfy bending moment requirements. We anticipate that displacement augercast piles are feasible so that drilling spoils will be minor.

8.5.1.1 Vertical Capacity

Adjacent pile centers should be spaced at least three diameters apart; otherwise, a reduction for group effects may be required. Grade beams should span between piles and/or pile caps in accordance with structural requirements. Floors should be designed as structural slabs to span between grade beams.

As no significantly thick, dense sand layer was encountered during our investigation that would provide adequate end bearing support, vertical capacity is based on frictional resistance. We evaluated the allowable vertical capacity for 16-inch diameter APGD piles. We have estimated that the top of pile/bottom of pile cap occurs at about 3 feet below the existing grades. We have considered downdrag effects on the piles resulting from the undocumented fill and the underlying Bay Mud, which is anticipated to extend up to about 37 feet below current grades for Site 6A. The allowable capacities shown on Figure 5 are for dead plus live loads; dead loads should not exceed two-thirds of the allowable capacities. The allowable capacities may be increased by one-third for wind and seismic loads. Uplift loads should not exceed 75 percent of the allowable downward vertical capacity under seismic loading. Gross capacity of the piles should be less than the structural capacity of the piles.

8.5.1.2 Lateral Capacity

Lateral load resistance is developed by the soil's resistance to pile bending. The magnitude of the shear and bending moment developed within the pile are dependent on the pile stiffness, embedment length, the fixity of the pile into the pile cap (free or fixed-head conditions), the surrounding soil properties, the tolerable lateral deflection, and yield moment capacity of the pier.

We utilized the computer program L-Pile to model the load-deflection (p-y) curves representing the soil conditions surrounding the pile, and estimate the ultimate lateral load capacity of the pier. The following table presents the probable response of the piles under short-term loading conditions; the structural engineer should apply an appropriate factor of safety on the shears and moments presented. Pile stiffnesses (EI) of 8.1×10^9 lb-in^2 has been assumed in our analysis for 16-inch APGD piles. We assumed a concrete compressive strength of greater than 4,000 psi for the concrete modulus calculations. If the pile stiffness varies by less than 20 percent of our assumed stiffness, the lateral load parameters below may be interpolated by multiplying the values by the ratio of the different pier stiffness values. We should be retained to re-evaluate the lateral load capacity for piles with a stiffness significantly different from what was assumed.

Table 7: Ultimate Lateral Load Capacity

Pile Type	Fixity Condition	Lateral Deflection (inches)	Maximum Shear (kips)	Maximum Moment (kip-feet)	Depth to Maximum Moment (feet)	Depth to Zero Moment (feet)
16-inch APGD	Free-Head	0.25	12.1	24.3	5.1	14.9
		0.50	24.0	46.7	4.8	16.0
16-inch APGD	Fixed-Head	0.25	24.4	71.1	0	4.5
		0.50	47.2	134.2	0	4.5

The above lateral capacities are for single piles and may not be representative of piles in groups. Group effects, including the layout of the piers within a group, can significantly reduce the overall lateral capacity. We should be retained to review pier layouts and structural loads to evaluate what appropriate group efficiency reduction factors should be applied to the different group conditions, if applicable.

8.5.1.3 Passive Resistance against Pile Caps and Grade Beams

Passive resistance against pile caps and grade beams poured neat against native or engineered fill may also be considered; however, as the allowable lateral deflections of the piles are limited, full allowable passive will not be developed. We should be retained to work with the structural engineer to evaluate appropriate allowable passive pressures that maintain strain compatibility between the piles and pile caps, if additional passive resistance is required.

8.5.1.4 Pre-Production Test Program

One field pile load test should be performed per 150 to 250 piles, at locations throughout the building areas recommended by the geotechnical engineer. Static load tests include installing a test pile, which can either be in a production pile location or not, with four surrounding piles that serve as anchor piles to resist the jacking pressure. During test pile installation, the contractor should allow for monitoring at the bottom of Bay Mud (approximately) and within 5 feet of the pile tip. This can be accomplished either with provisions for telltales or strain gauges. This monitoring will allow for observation of the skin friction and downdrag. A member of our staff should be present during test pile installation and testing.

8.5.1.5 Construction Considerations

All test and production piles should be observed by a Cornerstone representative to confirm that the piles are constructed in accordance with our recommendations and project requirements. Since the piles will derive their capacity from skin friction, the production piles should be installed to the design tip elevation. The geotechnical project engineer should review the installation records for conformance. We may recommend additional testing of piles, or additional installations, if any pile installations vary from normal installation practices.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS – SITE 6A

Because of the substantial settlement that will occur due to the consolidation of the underlying soft Bay Mud, first level floors for the proposed at-grade structures supported on deep foundations should be designed to structurally span between pile caps and grade beams. In addition, we recommend that consideration be given to deepening grade beams constructed along the perimeter of the proposed structures, if necessary, to prevent the exposure of the bottom of the structures due to long-term settlement of the adjacent grades.

If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.2 INTERIOR SLABS – SITES 5A, 5B, AND 6B

Slabs-on-grade used in conjunction with spread footings over ground improvement should be supported on at least 6 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

As previously discussed, the recommended non-expansive section here is different than for soil subgrade stabilization or "dry out" provided in Section 6.5 above.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment. The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended. As an alternative, the crushed rock section can also be thickened to 6 inches to meet the non-expansive fill section recommended in Section 9.2 above.
- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.4 EXTERIOR FLATWORK

9.4.1 Pedestrian Concrete Flatwork

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

9.4.2 Pedestrian Pavers

Concrete unit pavers subject to pedestrian and/or occasional light pick up loading should be at least 60 mm thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. A maximum 1-inch-thick layer of sand may be used as a leveling/setting bed over the aggregate base. Pavers that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. Where pavers will span transitions from on-grade to on-structure, consideration should be given to including a concrete sub-slab supported on the basement wall capable of spanning over the first 2 to 3 feet of wall backfill.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on the clayey nature of surficial samples collected and engineering judgment considering the variable surface conditions.

Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value of 78

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be use the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the “Concrete Slabs and Pedestrian Pavements” section above. We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 9: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.5
130	6.0

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the variable surficial soils present, we recommend that the construction and expansion joints be dowelled.

10.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduce to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or "Deep-Root Moisture Barriers" that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall including building loads. Based on the current site plan for Site 5B, the at-grade buildings are within close proximity of the basement level wall. We recommend the wall be designed to account for the additional building load surcharge. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 10: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

Basement walls should be designed as restrained walls. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2010 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We reviewed the seismic earth pressures for the proposed basement using procedures generally based on the Mononobe-Okabe method. Because the walls are greater than 10 to 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures (Lew et al., SEAOC 2010), it appears that active earth pressures plus a seismic increment do exceed the fixed wall earth pressures. Therefore, an additional seismic increment equal to $20H^2$, should be applied as an additional triangular load, with the resultant occurring at a distance ⅓ the height of the wall above the bottom.

11.2 WALL DRAINAGE

11.2.1 At-Grade Site Walls

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.2.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain.

Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path. In addition, where drainage panels will connect from a horizontal application for plaza areas to vertical basement wall drainage panels, the drainage path must be maintained. We are not aware of manufactured corner protection suitable for this situation; therefore, we recommend that a section of crushed rock be placed at the transitions. The crushed rock should be at least 3 inches thick, extend at least 12 inches horizontally over the top of the basement roof and 12 inches down from the top of the basement wall, and have a layer of filter fabric covering the crushed rock.

Drainage panels should terminate 18 to 24 inches from final exterior grade unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the "At-Grade Site Walls" section may also be used.

11.3 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

11.4 FOUNDATIONS

At-grade site retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented below.

Spread footings should bear on natural, undisturbed soil or engineered fill and extend at least 18 inches below the lowest adjacent grade. Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report should be limited to a maximum allowable bearing pressure of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footings extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement. Lateral loading and resistance should be in accordance with the Lateral Loading section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Republic Urban Properties LLC specifically to support the design of the Millbrae BART Transit Oriented Development in Millbrae, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Republic Urban Properties LLC may have provided Cornerstone with plans, reports and other documents prepared by others. Republic Urban Properties LLC understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

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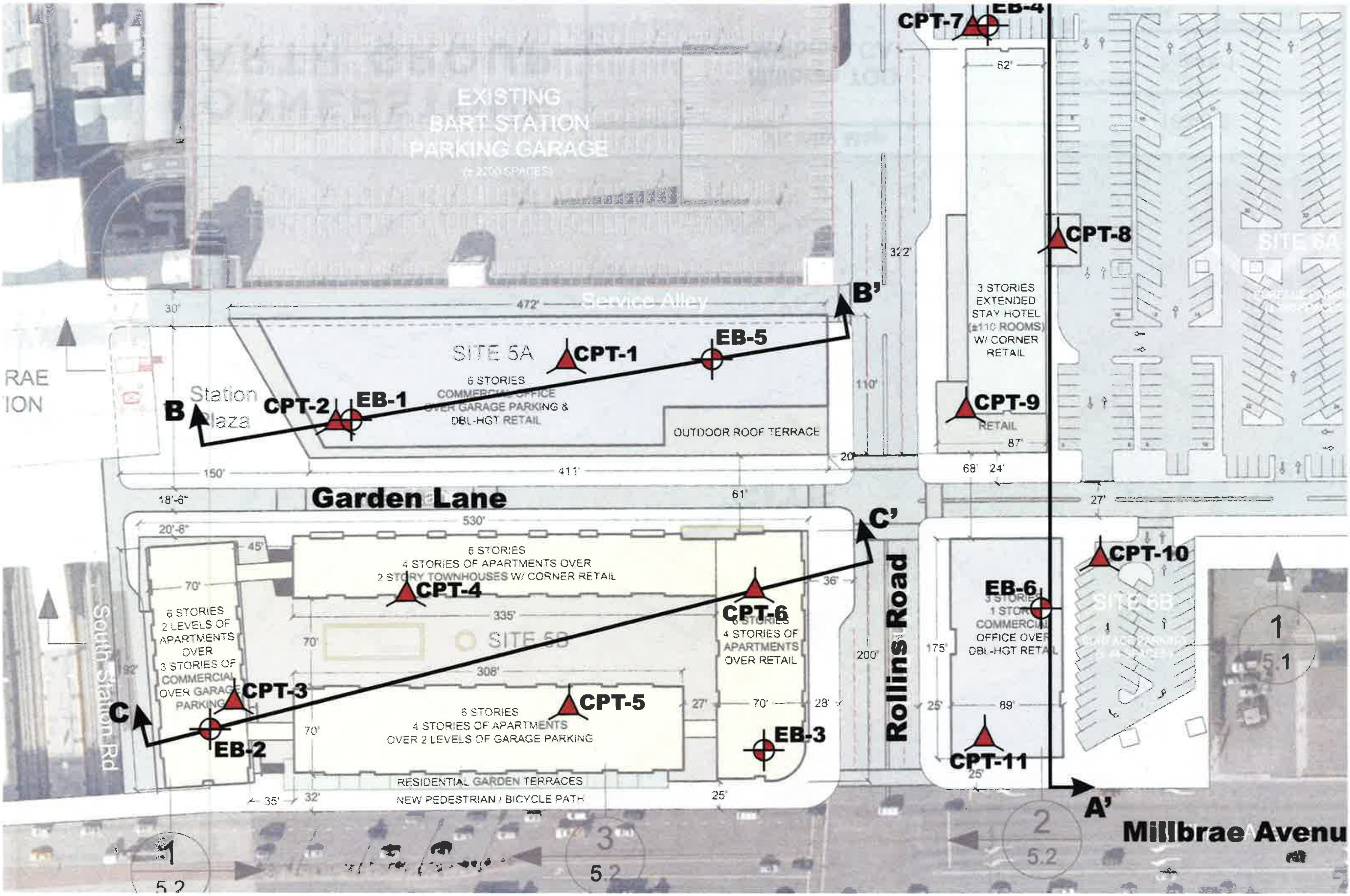
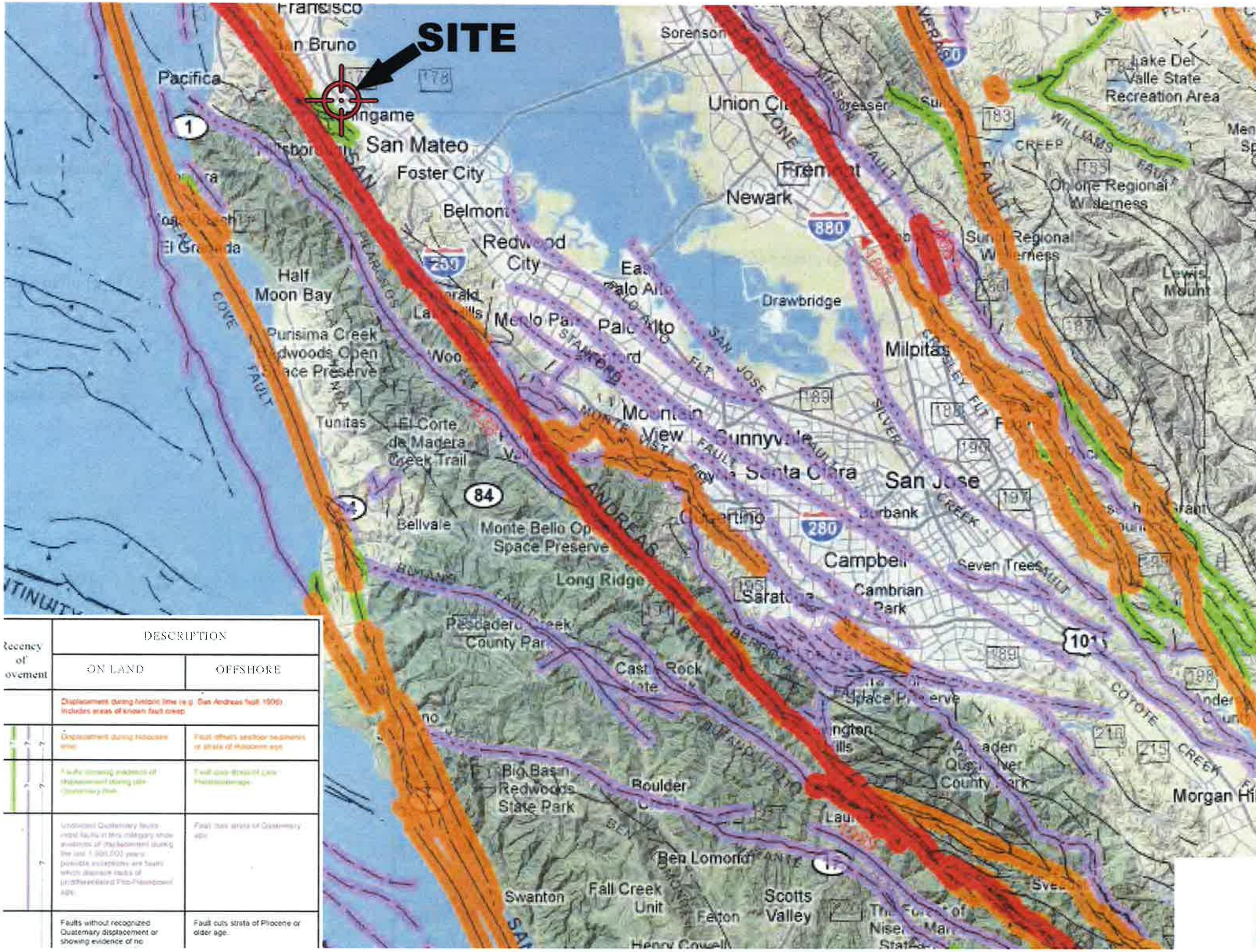
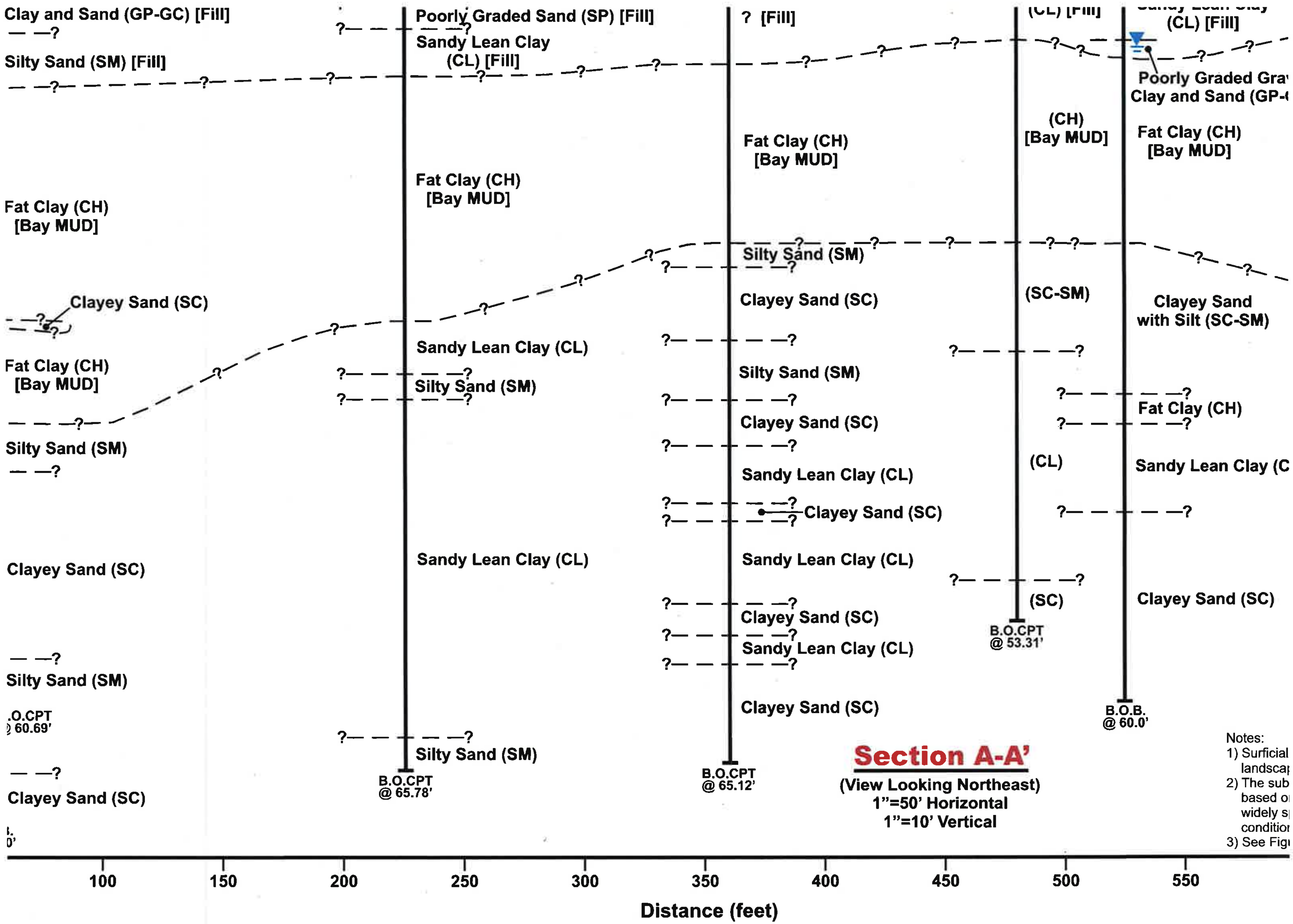
CORNERSTONE EARTH GROUP	Vicinity Map	Project Number: 183-5-2
	Millbrae TOD Millbrae, CA	Figure Number: Figure 1
		Date: May 2014 · Drawn By: RRN

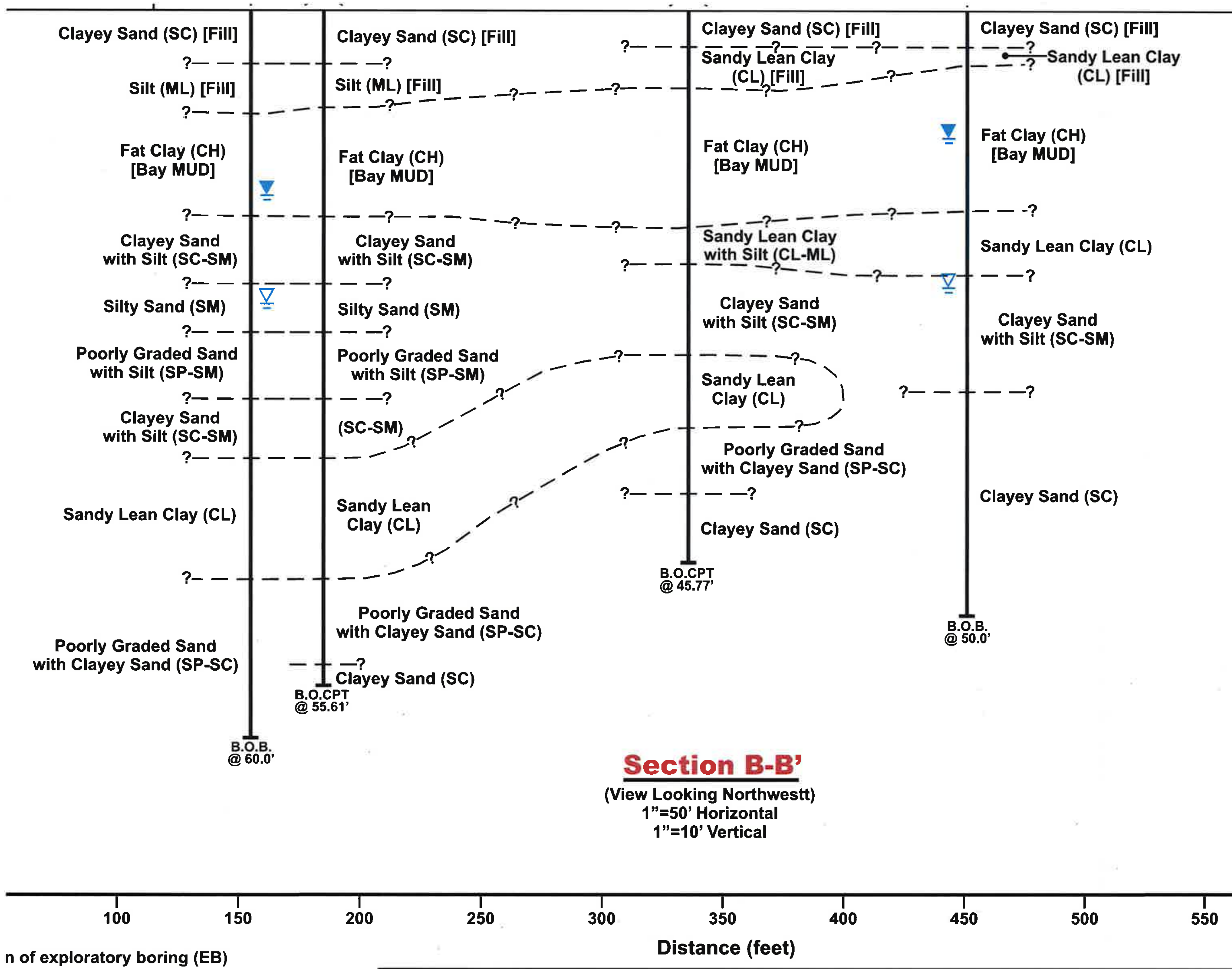


Legend

ocation of exploratory boring (EB)


SITE
Francisco
San Bruno
Pacifica
Burlingame
San Mateo
Foster City
Belmont
Redwood City
Half Moon Bay
El Granada
Purisima Creek Redwoods Open Space Preserve
Tunitas
El Corte de Madera Creek Trail
Bellvale
Monte Bello Open Space Preserve
Long Ridge
Pescadero Creek County Park
Castle Rock
Big Basin Redwoods State Park
Boulder
Ben Lomond
Swanton
Fall Creek Unit
Felton
Scotts Valley
Henry Cowell
Sorenson
Union City
Fremont
Newark
Drawbridge
East Palo Alto
Palo Alto
Menlo Park
Mountain View
Sunnyvale
Santa Clara
San Jose
Milpitas
Cupertino
Saratoga
Campbell
Cambrian Park
Burbank
Seven Trees
Morgan Hill
Lake Del Valle State Recreation Area
Ohlone Regional Wilderness
Sunol Regional Wilderness
Almaden Quicksilver County Park
The Forest of Nisene Marks State Park
SAN ANDREAS
PILARCITOS
SAN JOSE FLT.
SILVER CREEK
COYOTE CREEK
MONTE VISTA FAULT
BERROCAL
ZAYANTE
BUTANO FAULT
COVE FAULT
WILLIAMS FAULT
DISCONTINUITY
1
84
280
880
101
178
183
189
DESCRIPTION
Recency of Movement
ON LAND
OFFSHORE
Displacement during historic time (e.g. San Andreas fault 1906) Includes areas of known fault creep
Displacement during Holocene time.
Fault offsets seafloor sediments or strata of Holocene age.
Faults without recognized Quaternary displacement or showing evidence of no
Fault cuts strata of Pliocene or older age.


Clay and Sand (GP-GC) [Fill]
Silty Sand (SM) [Fill]
Fat Clay (CH)
[Bay MUD]
Clayey Sand (SC)
Fat Clay (CH)
[Bay MUD]
Silty Sand (SM)
Clayey Sand (SC)
Silty Sand (SM)
Clayey Sand (SC)
Poorly Graded Sand (SP) [Fill]
Sandy Lean Clay
(CL) [Fill]
Fat Clay (CH)
[Bay MUD]
Sandy Lean Clay (CL)
Silty Sand (SM)
Sandy Lean Clay (CL)
Silty Sand (SM)
B.O.CPT
@ 65.78'
? [Fill]
Fat Clay (CH)
[Bay MUD]
Silty Sand (SM)
Clayey Sand (SC)
Silty Sand (SM)
Clayey Sand (SC)
Sandy Lean Clay (CL)
Clayey Sand (SC)
Sandy Lean Clay (CL)
Clayey Sand (SC)
Sandy Lean Clay (CL)
Clayey Sand (SC)
B.O.CPT
@ 65.12'
(CH)
[Bay MUD]
(SC-SM)
(CL)
(SC)
B.O.CPT
@ 53.31'
(CL) [Fill]
Fat Clay (CH)
[Bay MUD]
Clayey Sand
with Silt (SC-SM)
Fat Clay (CH)
Clayey Sand (SC)
B.O.B.
@ 60.0'
Section A-A'
(View Looking Northeast)
1"=50' Horizontal
1"=10' Vertical
Notes:
1) Surficial
2) The sub
3) See Fig
100
150
200
250
300
350
400
450
500
550
Distance (feet)

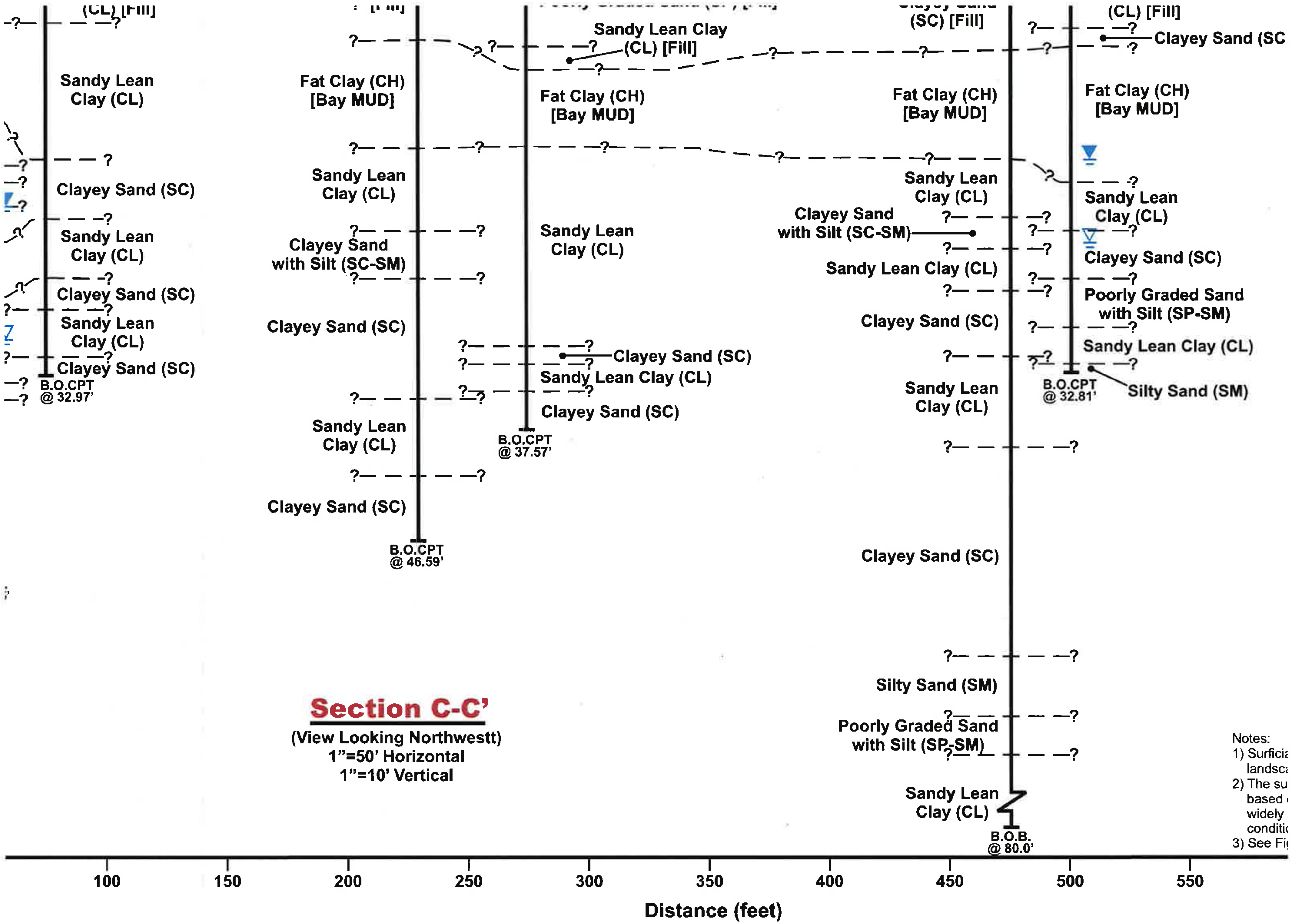


Section B-B'

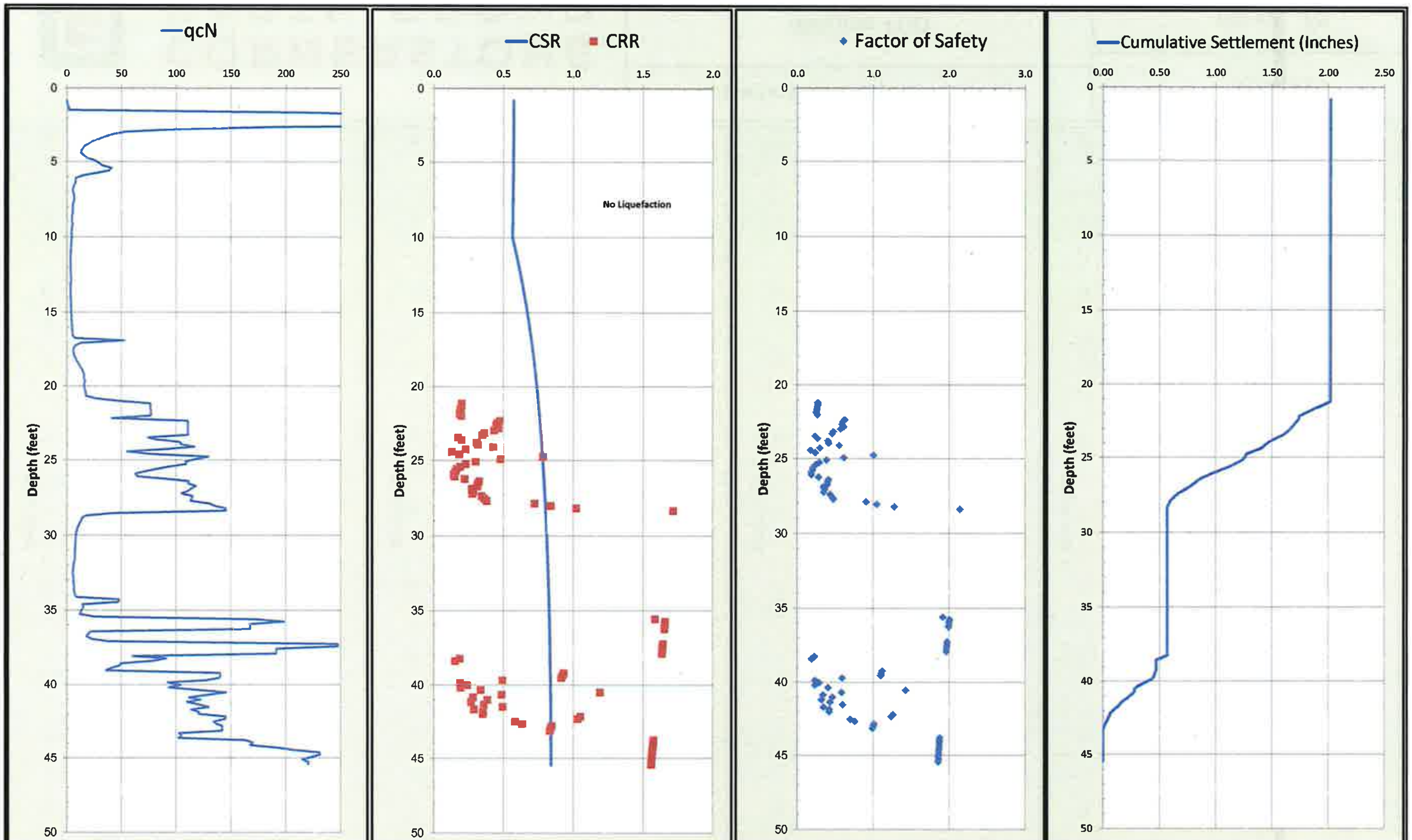
(View Looking Northwestt)
1"=50' Horizontal
1"=10' Vertical

Notes:
1) Surfi
lands
2) The s
base
widel
cond
3) See

n of exploratory boring (EB)



n of exploratory boring (EB)

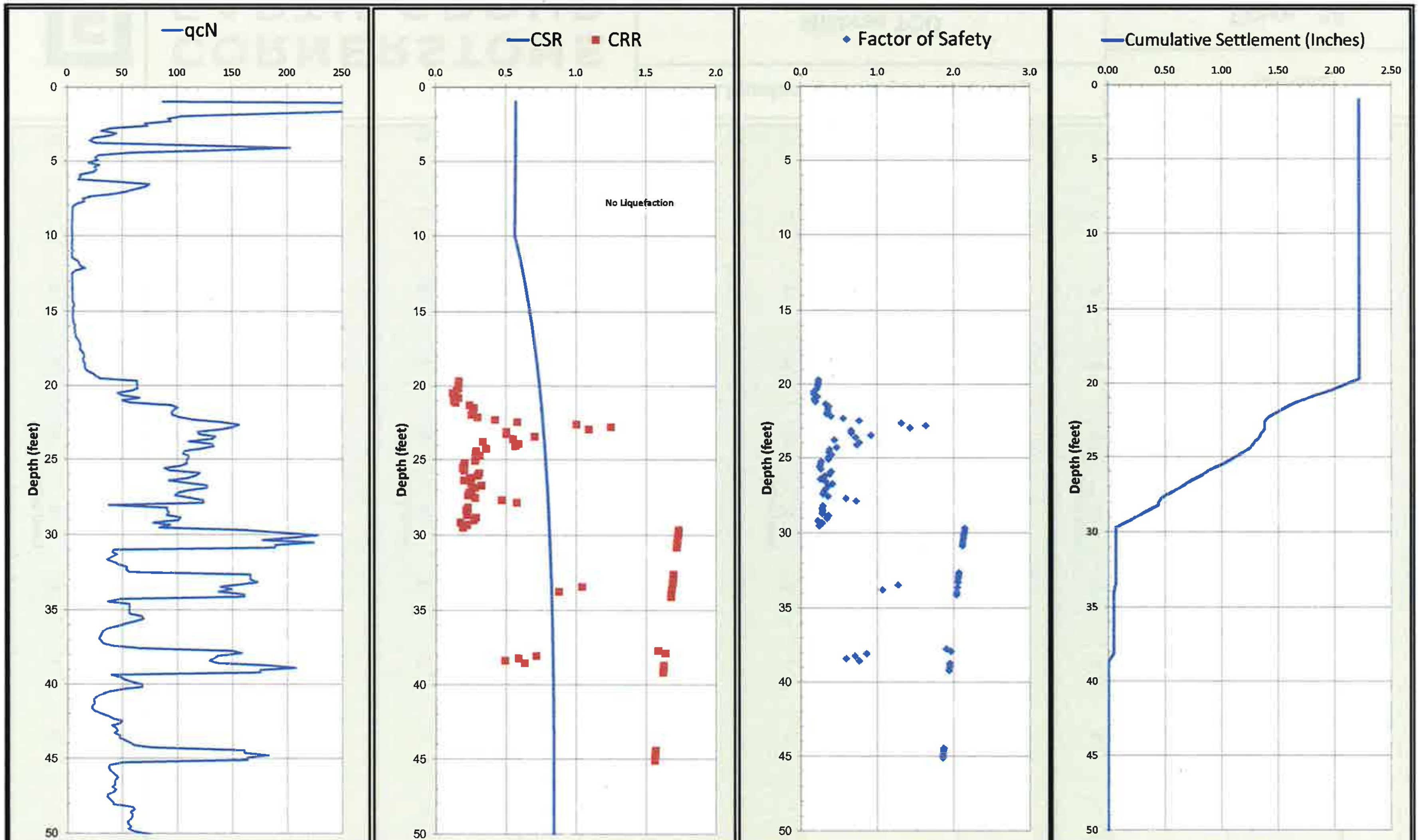


CORNERSTONE EARTH GROUP

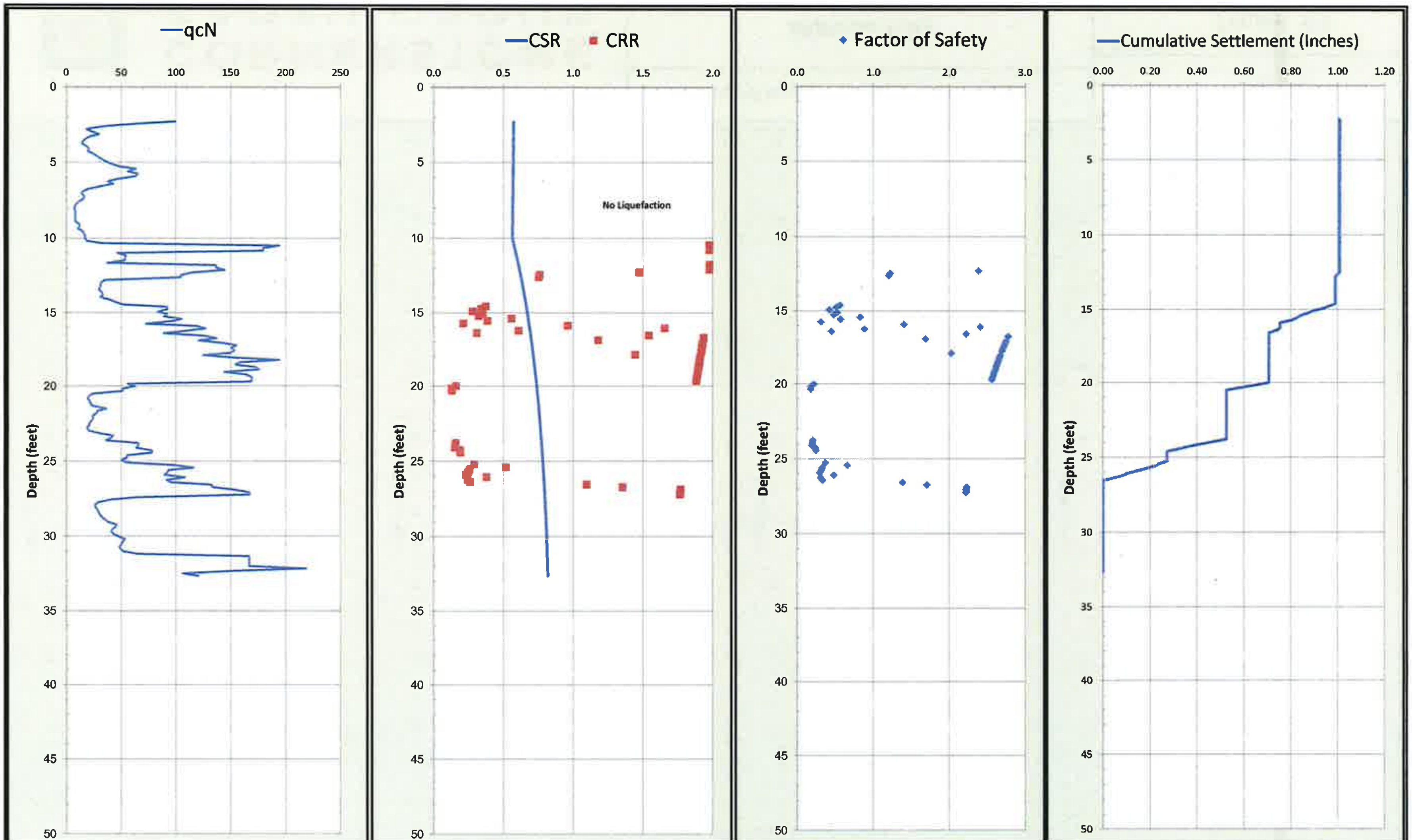
Liquefaction Analysis Summary

Millbrae TOD
Millbrae, California

Project Number	183-5-2
Figure Number	Figure 6A
6/13/2014	CPT No. 1

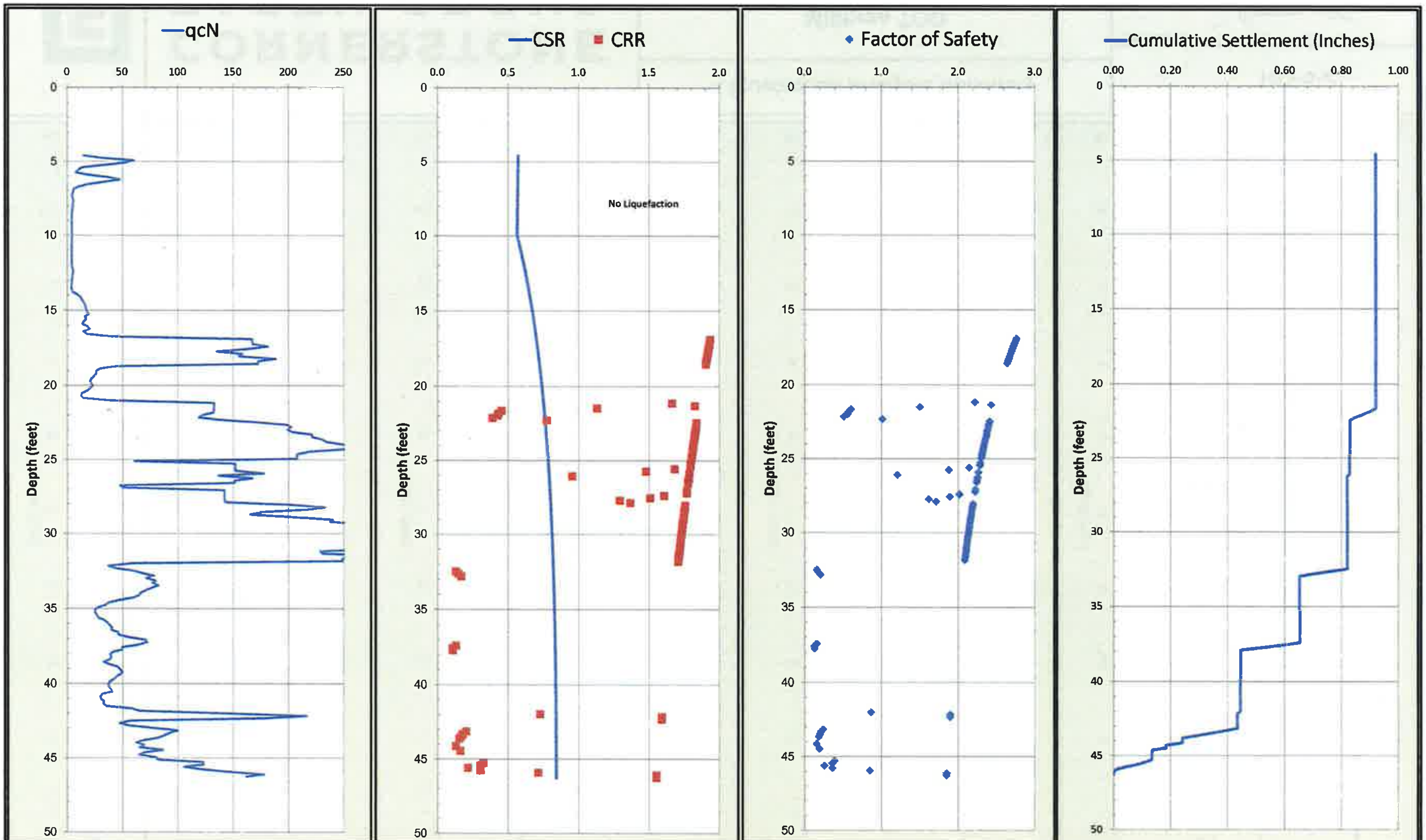


CORNERSTONE EARTH GROUP	Liquefaction Analysis Summary	Project Number 183-5-2
	Millbrae TOD Millbrae, California	Figure Number Figure 6B
		6/13/2014 — CPT No. 2



CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary	Project Number 183-5-2
Millbrae TOD Millbrae, California	Figure Number Figure 6C
	6/13/2014 / CPT No. 3



CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

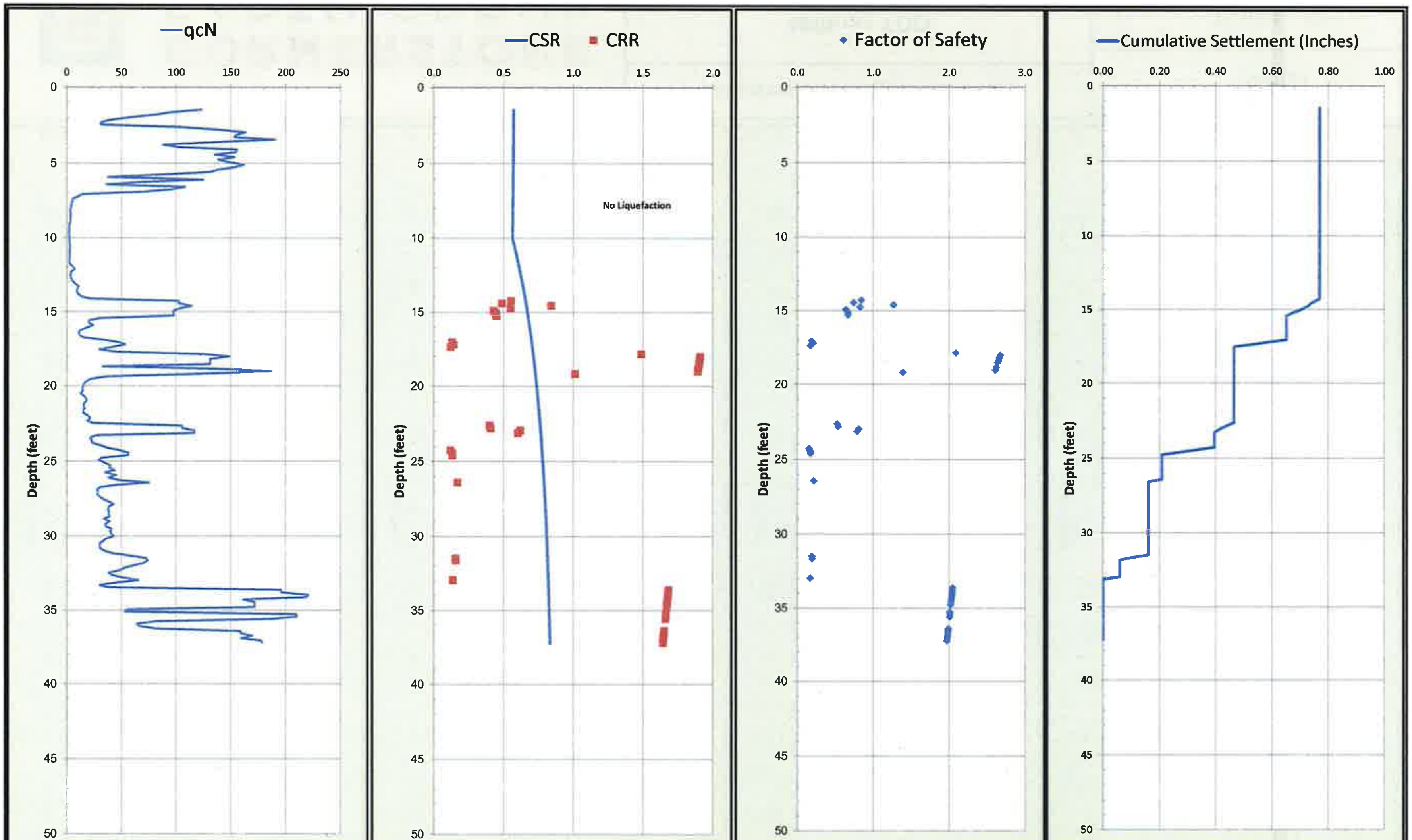
Millbrae TOD
Millbrae, California

Project Number
183-5-2

Figure Number
Figure 6D

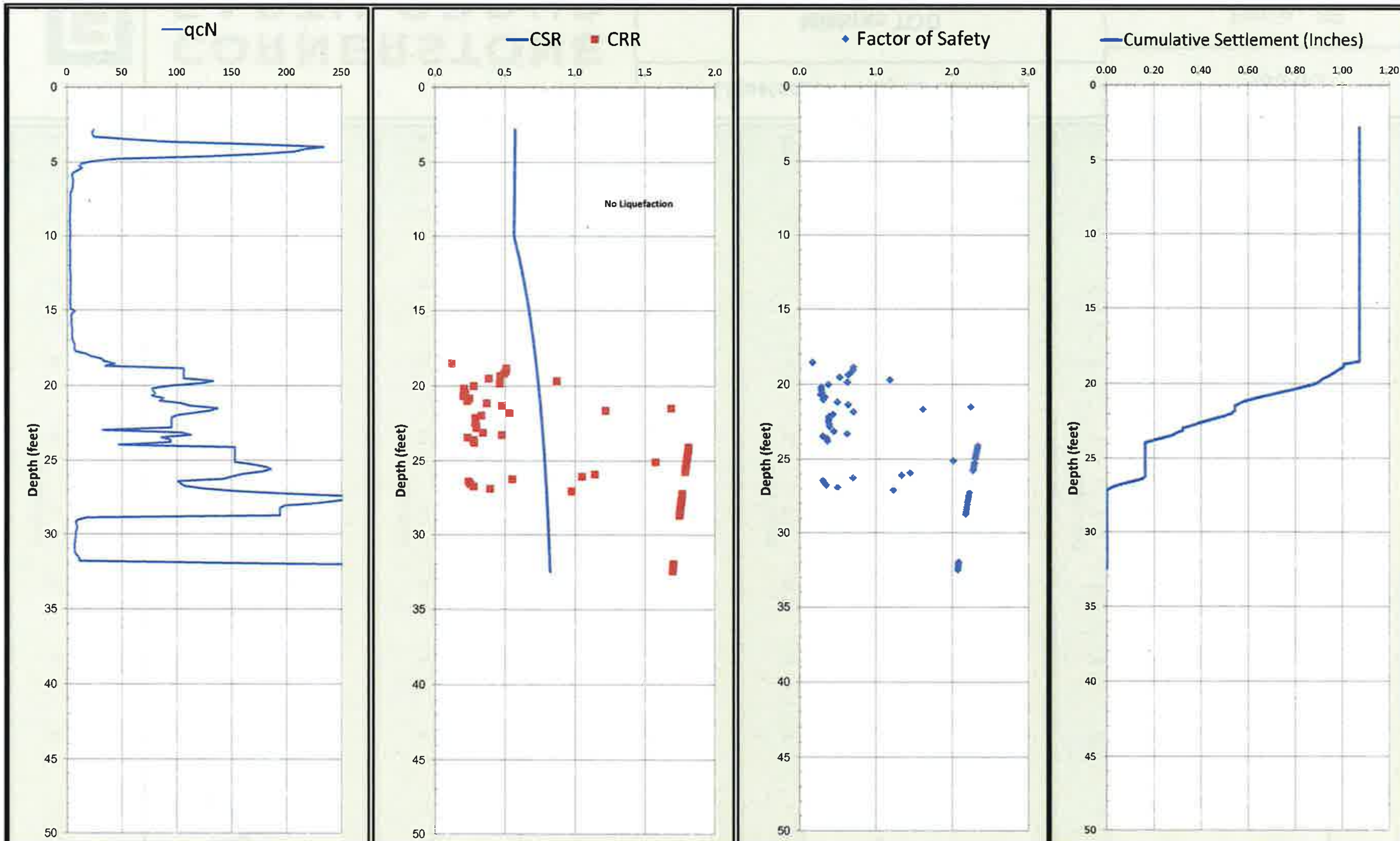
6/13/2014

CPT No. 4



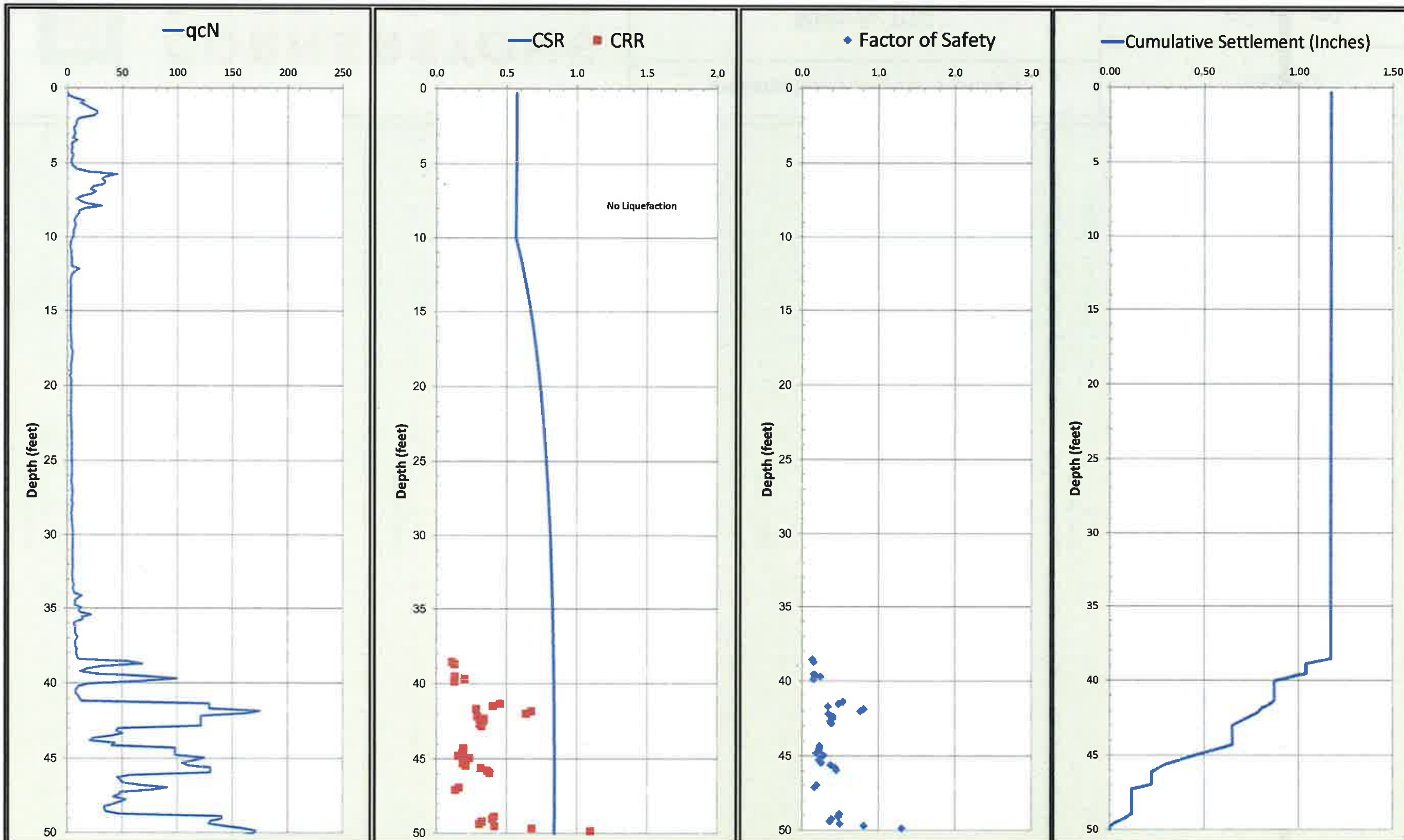
CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary	Project Number 183-5-2	
Millbrae TOD **Millbrae, California**	Figure Number Figure 6E	
	6/13/2014	CPT No. 5



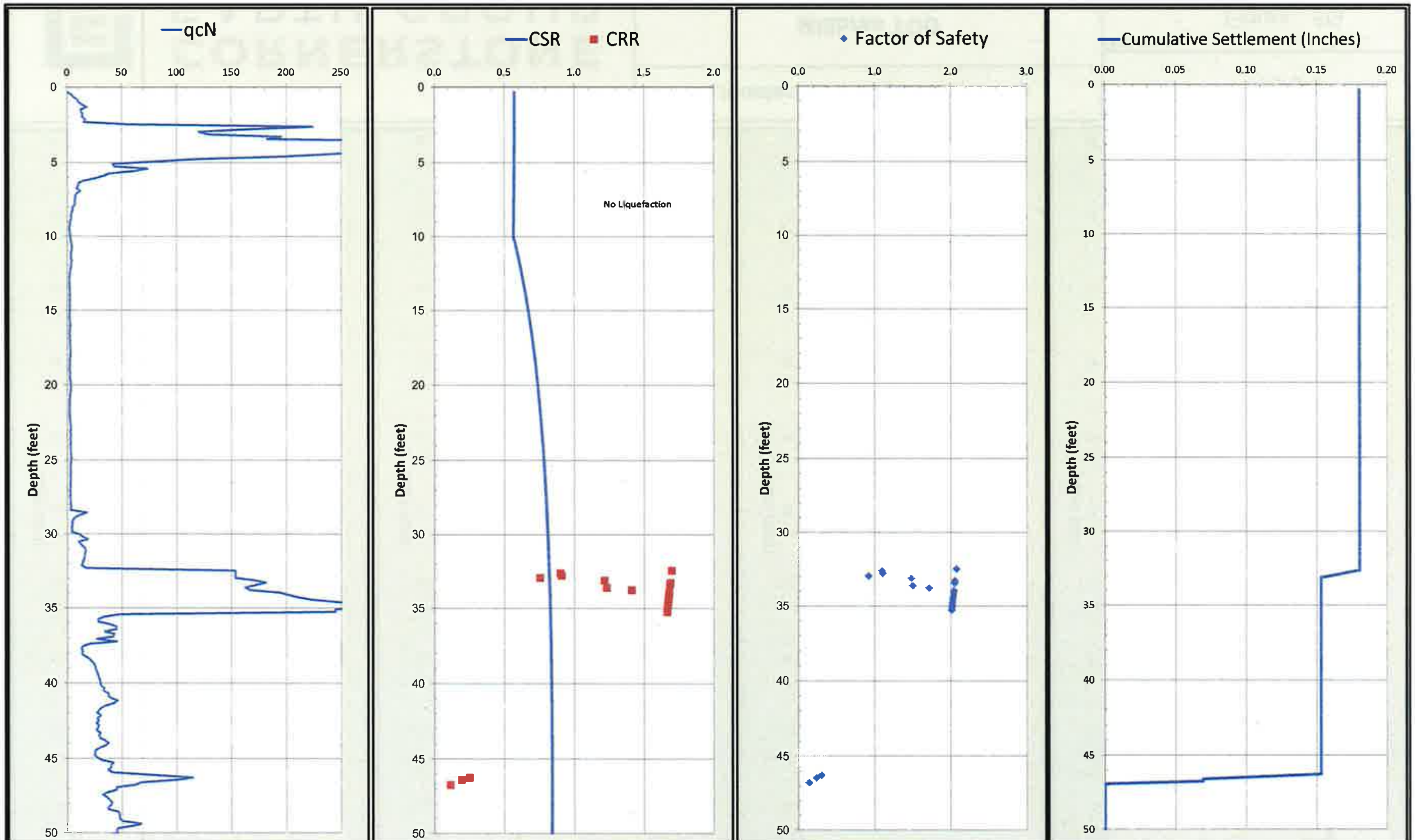
CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary	Project Number 183-5-2	
Millbrae TOD Millbrae, California	Figure Number Figure 6F	
	6/13/2014	CPT No. 6

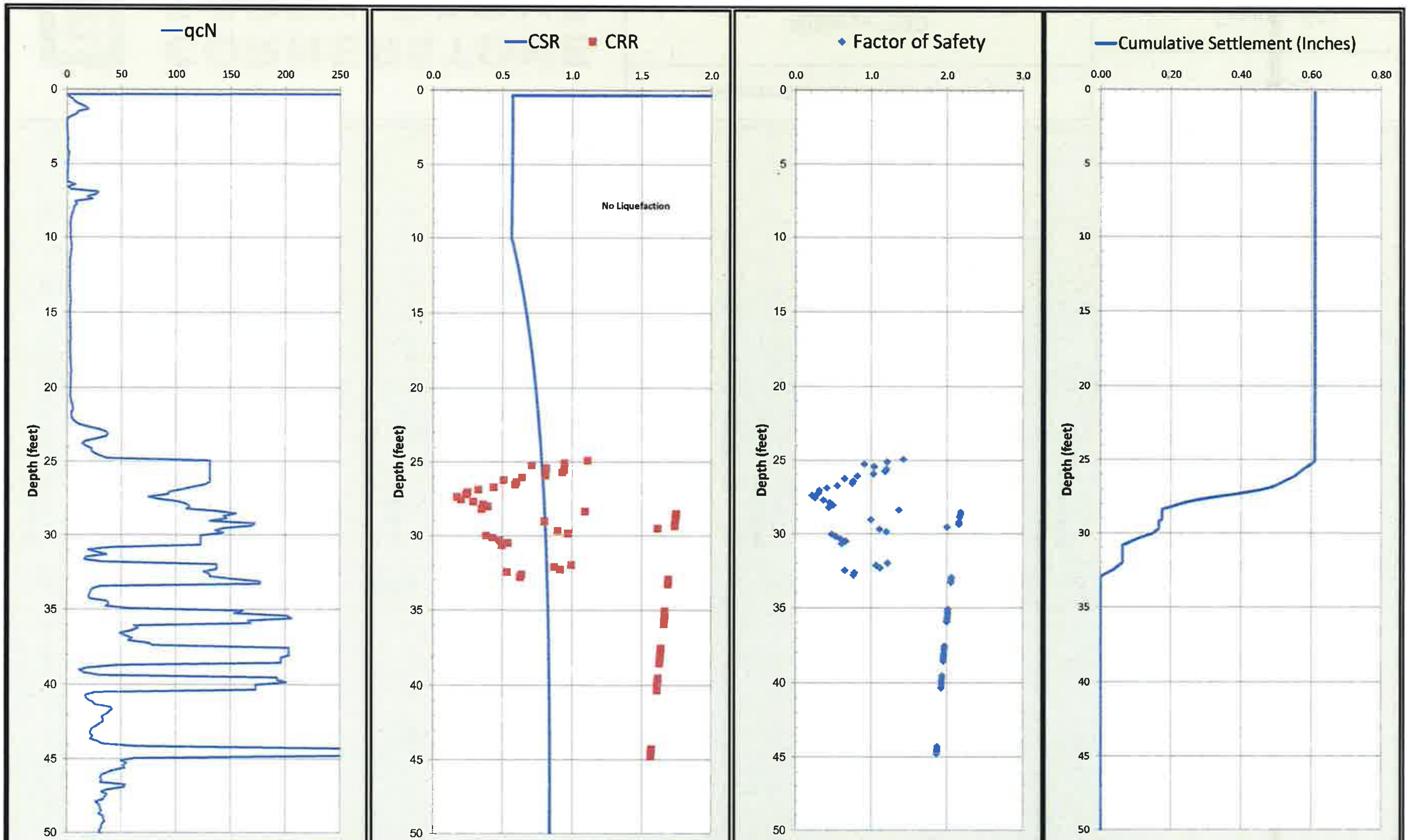


CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary	Project Number	183-5-2
Millbrae TOD **Millbrae, California**	Figure Number	Figure 6G
	6/13/2014	CPT No. 7

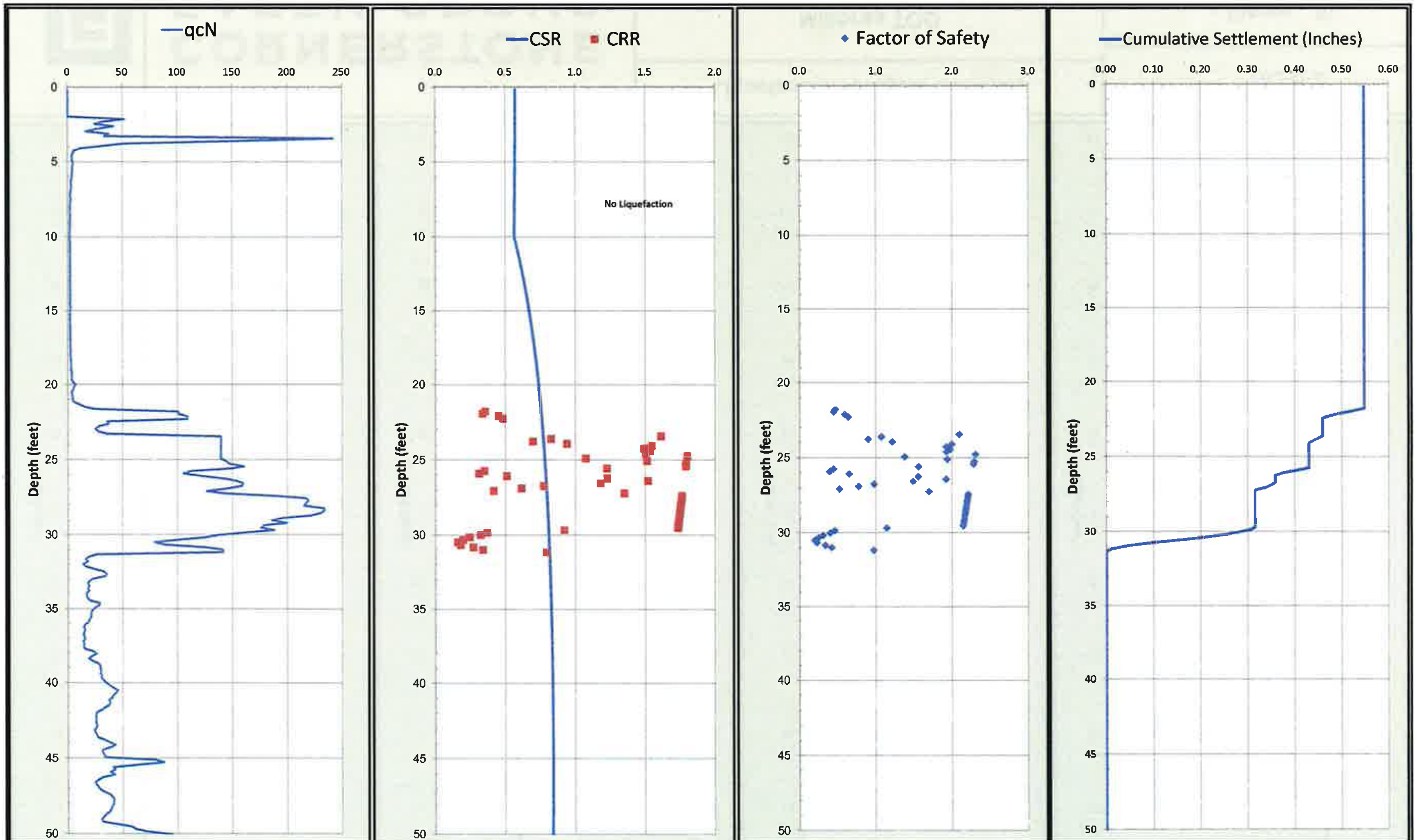


CORNERSTONE EARTH GROUP	Liquefaction Analysis Summary	Project Number 183-5-2
	Millbrae TOD Millbrae, California	Figure Number Figure 6H
		6/13/2014 / CPT No. 8



CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary	Project Number 183-5-2	
Millbrae TOD Millbrae, California	Figure Number Figure 6I	
	6/13/2014	CPT No. 9

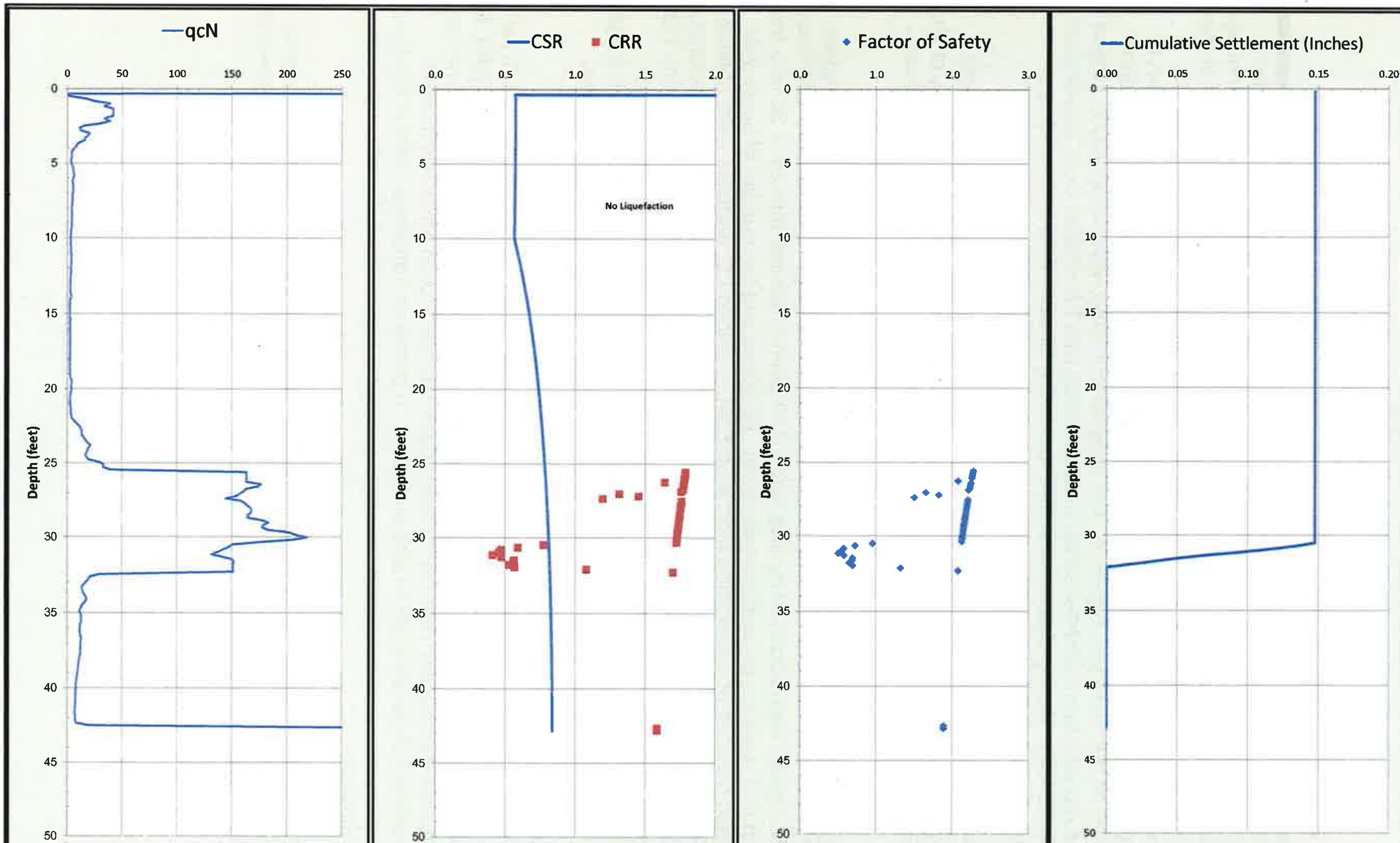


CORNERSTONE EARTH GROUP

Liquefaction Analysis Summary

Millbrae TOD
Millbrae, California

Project Number	183-5-2
Figure Number	Figure 6J
6/13/2014	CPT No. 10



CORNERSTONE EARTH GROUP	Liquefaction Analysis Summary	Project Number 183-5-2
	Millbrae TOD Millbrae, California	Figure Number Figure 6K
		6/13/2014 — CPT No. 11

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Six 8-inch-diameter exploratory borings were drilled on April 26, May 3, and May 4, 2014 to depths of 50 to 80 feet. Eleven CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on April 26 and 27, 2014, to depths ranging from 33 to 65¾ feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries and other site features as references. Boring and CPT elevations were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

Poorly-Graded Sand with Clay	Sand
Clayey Sand	Silt
Sandy Silt	Well Graded Gravelly Sand
Artificial/Undocumented Fill	Gravelly Silt
Poorly-Graded Gravelly Sand	Asphalt
Topsoil	Boulders and Cobble
Well-Graded Gravel with Clay	
Well-Graded Gravel with Silt	

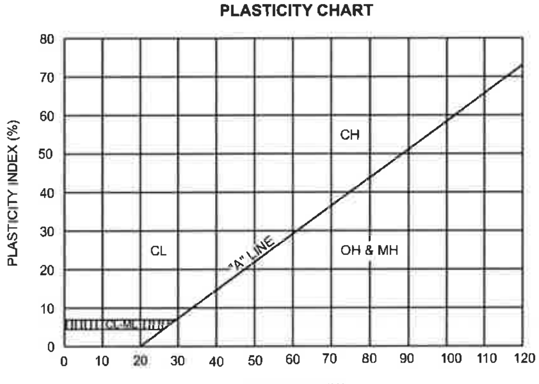
SAMPLER TYPES

SPT	Shelby Tube
Modified California (2.5" I.D.)	No Recovery
Rock Core	Grab Sample

ADDITIONAL TESTS

CA	CHEMICAL ANALYSIS (CORROSIVITY)	PI	PLASTICITY INDEX
CD	CONSOLIDATED DRAINED TRIAXIAL	SW	SWELL TEST
CN	CONSOLIDATION	TC	CYCLIC TRIAXIAL
CU	CONSOLIDATED UNDRAINED TRIAXIAL	TV	TORVANE SHEAR
DS	DIRECT SHEAR	UC	UNCONFINED COMPRESSION
PP	POCKET PENETROMETER (TSF)	(1.5)	(WITH SHEAR STRENGTH IN KSF)
(3.0)	(WITH SHEAR STRENGTH IN KSF)	UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
RV	R-VALUE		
SA	SIEVE ANALYSIS: % PASSING #200 SIEVE		
▼	WATER LEVEL		

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

DATE STARTED 4/26/14 **DATE COMPLETED** 4/26/14

GROUND ELEVATION ____ **BORING DEPTH** 60 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE ____ **LONGITUDE** ____

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY MAA

▽ **AT TIME OF DRILLING** 24 ft.

NOTES ____

▼ **AT END OF DRILLING** 15 ft.

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	9 inches Portland cement concrete over 12 inches aggregate base							
		Clayey Sand with Gravel (SC) [Fill] medium dense, moist, brown with gray mottles, fine to coarse sand, fine subangular gravel, some brick fragments	47	MC-1B	112	19			
			46	MC-2B	112	9			
	5	**Sandy Silt (ML) [Fill]** hard, moist, gray, fine sand, low plasticity	20	MC-3B	100	23			>4.5
	10	**Fat Clay (CH) [Bay Mud]** soft, moist, gray, some organics, trace fine sand, high plasticity	4	MC-4B	30	130			
				ST-5	48	90			
	15 ▼		4	SPT-6		60			
		Silty, Clayey Sand (SC-SM) medium dense, moist, gray and brown mottled, fine to medium sand	16	MC-7		21			
	20	Liquid Limit = 23, Plastic Limit = 18	29	MC-8		19	5	32	
	▽ 25	**Silty Sand (SM)** dense, moist, reddish brown, fine to medium sand, some fine subangular to subrounded gravel	50	MC-9		17			

Continued Next Page

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	30	**Poorly Graded Sand with Silt and Gravel (SP-SM)** dense, moist, dark olive brown, fine to medium sand, fine subangular to subrounded gravel	48	MC-10		17		10	
	35	**Silty, Clayey Sand (SC-SM)** medium dense, moist, bluish gray and olive brown mottled, fine to medium sand Liquid Limit = 24, Plastic Limit = 17	40	SPT-11		20	7	39	
	40	**Sandy Lean Clay (CL)** very stiff to hard, moist, bluish gray and olive brown mottled, fine to medium sand, low plasticity	38	MC-12B	111	17			○ 4.0
	45		50/5"	MC-13B	120	15			○ >4.5
	50	**Poorly Graded Sand with Silt (SP-SC)** dense, moist, light gray brown, fine to medium sand	59	MC-14B	111	18			
	55	becomes very dense	50/4"	SPT-15		15			
	60		60	SPT-16		17			
		Bottom of Boring at 60.0 feet.							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER 183-5-2
PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA
DATE STARTED 5/4/14 **DATE COMPLETED** 5/4/14
GROUND ELEVATION **BORING DEPTH** 49.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RSM
AT TIME OF DRILLING 30 ft.
NOTES
AT END OF DRILLING 19 ft.

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	3½ inches asphalt concrete over 10 inches aggregate base							
		Sandy Lean Clay (CL) [Fill] very stiff, moist, brown and dark gray mottled, fine to medium sand, low to moderate plasticity	49	MC-1B	116	16	19		○
		Liquid Limit = 34, Plastic Limit = 15	38	MC-2B	104	20			>4.5 ○
	5	**Sandy Lean Clay (CL)** very stiff, moist, bluish gray with light brown mottles, fine sand, low plasticity	40	MC-3B	113	16			○
			15	MC-4B	106	21			○
	10	**Lean Clay with Sand (CL)** stiff to very stiff, moist, light gray brown with olive mottles, fine sand, moderate plasticity	28	SPT-5		20			○
		Clayey Sand (SC) dense, moist, light gray brown, fine to medium sand	30	SPT-6		17		45	
	15	**Silty Sand (SM)** dense, moist, light gray brown with brown mottles, fine to medium sand	35	7A SPT		19			
		Clayey Sand (SC) dense, moist, light gray brown with reddish brown mottles, fine to medium sand		7B		13		25	
		Silty Sand (SM) dense, moist, light gray brown with reddish brown mottles, fine to medium sand	36	SPT-8		21		31	
	20	**Sandy Lean Clay (CL)** stiff, moist, brown with gray mottles, fine sand, low plasticity	18	SPT-9B		18			○
	25	**Clayey Sand (SC)** dense, moist, gray with reddish brown mottles, fine to medium sand, some fine subangular gravel							

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained shear strength (ksf): ○ Hand Penetrometer; △ Torvane; ● Unconfined Compression; ▲ Unconsolidated-Undrained Triaxial. Scale: 1.0, 2.0, 3.0, 4.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	30 (groundwater ▽)	SC	**Clayey Sand (SC)** dense, moist, gray with reddish brown mottles, fine to medium sand, some fine subangular gravel; Liquid Limit = 34, Plastic Limit = 15	55	MC-10B	118	15	19	47	
				53	SPT-11		14			
	35	CL	**Sandy Lean Clay (CL)** hard, moist, gray, fine to coarse sand, low plasticity	33	SPT-12		20			>4.5 (○)
		SM	**Silty Sand (SM)** dense, moist, light gray brown, fine to medium sand, some fine subangular to subrounded gravel	59	SPT-13		16			
	40		medium dense	20	SPT-14B	113	18		17	
	45		becomes very dense	50/5"	SPT-15B		13			
				50/6"	SPT-16B		13			
	50		Bottom of Boring at 49.5 feet.							
	55									
	60									

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER 183-5-2
PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

DATE STARTED 5/4/14 DATE COMPLETED 5/4/14
GROUND ELEVATION BORING DEPTH 80 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE LONGITUDE
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RSM
AT TIME OF DRILLING 22 ft.
NOTES
AT END OF DRILLING 15 ft.

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ELEVATION (ft) | DEPTH (ft) | SYMBOL

DESCRIPTION

3½ inches asphalt concrete over 9 inches aggregate base

Clayey Sand (SC) [Fill]
medium dense, moist, brown and gray mottled, fine to coarse sand, some fine subangular gravel
Liquid Limit = 25, Pastic Limit = 13

Fat Clay (CH) [Bay Mud]
soft, moist, gray, some organics, trace fine sand, high plasticity

some fine to medium sand

Sandy Lean Clay (CL)
very stiff to hard, moist, light gray brown with brown mottles, fine to medium sand, low plasticity

Silty, Clayey Sand (SC-SM)
medium dense, moist, gray and brown mottled, fine to medium sand

Sandy Lean Clay (CL)
very stiff, moist, light gray brown with brown mottles, fine to medium sand, low plasticity

Clayey Sand with Gravel (SC)
medium dense, moist, reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel

Depth scale (ft): 0, 5, 10, 15, 20, 25

Continued Next Page

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
16	MC-1B	119	12	12		
17	MC-2B	52	79			
2	MC-3B	45	96			
2	MC-4B	58	66			
	ST-5					
33	MC-6B	116	15			>4.5
21	SPT-7B		15			
20	NR					

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0
	30		**Clayey Sand with Gravel (SC)** medium dense, moist, reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	43	MC-8B	116	17		21	
			Sandy Lean Clay (CL) stiff, moist, light gray brown with reddish brown mottles, fine to medium sand, low plasticity	16	SPT-9		20			○
	35		**Lean Clay with Sand (CL)** hard, moist, light gray brown with olive mottles, fine to medium sand, low plasticity	35	SPT-10		16			>4.5 ○
	40		**Clayey Sand (SC)** dense, moist, light gray brown with reddish brown mottles, fine to coarse sand, some fine to coarse subangular to subrounded gravel							
	45			53	MC-11B	114	16		27	
	50			66	MC-12B	124	13			
	55		becomes very dense	76	SPT-13		13			
	60		**Silty Sand (SM)** very dense, moist, brown, fine to medium sand, some fine subangular to subrounded gravel	50/5"	SPT-14		15			

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CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER 183-5-2
PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
		Poorly Graded Sand with Silt and Gravel (SP-SM) medium dense, wet, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	30	SPT-15B		20			
	65	**Sandy Lean Clay (CL)** very stiff, moist, light gray brown with reddish brown mottles, fine to medium sand, low plasticity							
		color to light gray							
		Lean Clay with Sand (CL) very stiff, moist, gray, fine sand, moderate plasticity	59	MC-16B	109	19			○ 3.0
	70								
			13	SPT-17		21			○ 2.0
	75								
		becomes stiff	26	MC-18B	110	19			○ 1.3
	80	Bottom of Boring at 80.0 feet.							
	85								
	90								
	95								

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER 183-5-2
PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

DATE STARTED 5/3/14 DATE COMPLETED 5/3/14
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
LOGGED BY KOH
NOTES

GROUND ELEVATION BORING DEPTH 70 ft.
LATITUDE LONGITUDE
GROUND WATER LEVELS:
AT TIME OF DRILLING 7 ft.
AT END OF DRILLING 7 ft.

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	4 inches asphalt concrete over 12 inches aggregate base							
		Poorly Graded Gravel with Clay and Sand (GP-GC) [Fill] very dense, moist, brown and gray, fine to coarse subangular to subrounded gravel, fine to coarse sand	50/6"	MC-1B	130	5			
			51	MC-2B	111	10			
	5	**Silty Sand (SM) [Fill]** very dense, moist, bluish gray and brown mottled, fine sand							
			9	MC-3B	101	22			
			4	MC-4B	62	57			○ △
	10	**Fat Clay (CH) [Bay Mud]** soft, moist, gray, some organics, trace fine sand, high plasticity							
				ST-5	51	80			▲
	15		4	MC-6B	45	99			○△
			4	MC-7B	44	101			●▲
	20								
			4	MC-8B	51	83			○▲
	25								
		Clayey Sand (SC)							

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CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER 183-5-2
PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
		loose, wet, gray, fine to coarse sand	5	MC-9B	53	75		
	30	**Fat Clay (CH) [Bay Mud]** soft, moist, gray, some organics, trace fine sand, high plasticity						
	35		4	MC-10B	53	71		
		Silty Sand (SM) medium dense, moist, gray brown, fine to medium sand NP = non plastic						
	40		12	SPT-11		30	NP	24
		Clayey Sand with Gravel (SC) dense, wet, gray brown, fine to coarse sand, fine subangular to subrounded gravel	50	SPT-12		17		
	45							
	50		69	NR				
	55		81	MC-13B	123	14		17
		Silty Sand (SM) very dense, moist, gray brown, fine to medium sand						
			50/6"	SPT-14		22		24
	60							

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BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
			Silty Sand (SM) very dense, moist, gray brown, fine to medium sand	50/6"	MC-15B	108	19			
	65									
			Clayey Sand (SC) dense, moist, gray brown with bluish gray mottles, fine to medium sand	56	SPT-16		17			
	70		Bottom of Boring at 70.0 feet.							
	75									
	80									
	85									
	90									
	95									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/15/14 07:45 - P:\DRAFTING\GINT FILES\183-5-2 MILLBRAE TOD.GPJ

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME: Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER: 183-5-2
PROJECT LOCATION: 300 Millbrae Avenue, Millbrae, CA

DATE STARTED: 4/26/14 DATE COMPLETED: 4/26/14
GROUND ELEVATION: BORING DEPTH: 50 ft.
DRILLING CONTRACTOR: Exploration Geoservices, Inc.
LATITUDE: LONGITUDE:
DRILLING METHOD: Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY: MAA
AT TIME OF DRILLING: 23 ft.
NOTES:
AT END OF DRILLING: 10 ft.

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	12 inches Portland cement concrete over 10 inches aggregate base						
		Clayey Sand with Gravel (SC) [Fill] medium dense, moist, brown with gray mottles, fine to coarse sand, fine subangular gravel, some brick fragments	13	MC-1B	108	17		
			9	MC-2B	90	29		
	5	**Sandy Lean Clay (CL) [Fill]** stiff, moist, gray and brown mottled, fine to coarse sand, moderate plasticity	5	MC-3A	54	76		
		Fat Clay (CH) [Bay Mud] soft, moist, gray, some organics, trace fine sand, high plasticity	4	MC-4B	50	87		
			2	MC-5B	47	91		
	10			ST-6	49	90		
			3	MC-7B	46	94		
	15	**Sandy Lean Clay (CL)** stiff, moist, bluish gray, fine to medium sand, moderate plasticity						
			13	MC-8B	100	26		
	20	**Silty, Clayey Sand (SC-SM)** medium dense, moist, gray and brown mottled, fine to medium sand, some fine subangular to subrounded gravel						
			28	SPT-9		17		21
	25							

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BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	29–31.5	**Silty, Clayey Sand (SC-SM)** medium dense, moist, gray and brown mottled, fine to medium sand, some fine subangular to subrounded gravel	12	SPT-10		18		23	
	31.5–50	**Clayey Sand (SC)** dense, moist, gray with reddish brown mottles, fine to medium sand	38	SPT-11		19			
			56	MC-12B	109	19		41	
			40	SPT-13		18			
			46	SPT-14		18			
	50	Bottom of Boring at 50.0 feet.							

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment
PROJECT NUMBER 183-5-2
PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

DATE STARTED 5/3/14 DATE COMPLETED 5/3/14
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
LOGGED BY KOH
NOTES

GROUND ELEVATION BORING DEPTH 60 ft.
LATITUDE LONGITUDE
GROUND WATER LEVELS:
AT TIME OF DRILLING 5.5 ft.
AT END OF DRILLING 5.5 ft.

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	4 inches asphalt concrete over 10 inches aggregate base						
		Sandy Lean Clay (CL) [Fill] very stiff, moist, brown and dark gray mottled, fine to coarse sand, some fine subangular gravel, moderate plasticity Liquid Limit = 35, Plastic Limit = 15	26	MC-1B	92	33	20	
			23	MC-2B	105	15		
	5	**Poorly Graded Gravel with Clay and Sand (GP-GC) [Fill]** medium dense, moist, brown and gray, fine to coarse subangular to subrounded gravel, fine to coarse sand	5	MC-3B	49	83		
		Fat Clay (CH) [Bay Mud] soft, moist, gray, some organics, trace fine sand, high plasticity	3	MC-4B	51	84		
	10							
	15		2	MC-5B	46	97		
				ST-6	49	87		
			2	MC-7B	54	76		
	20							
		Clayey Sand (SC) medium dense to dense, moist, bluish gray, fine to medium sand Liquid Limit = 24, Plastic Limit = 16	17	SPT-8		18	8	
	25	color change to brown, some fine gravel	35	SPT-9		19		

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME Millbrae BART - Transit Oriented Developmment

PROJECT NUMBER 183-5-2

PROJECT LOCATION 300 Millbrae Avenue, Millbrae, CA

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

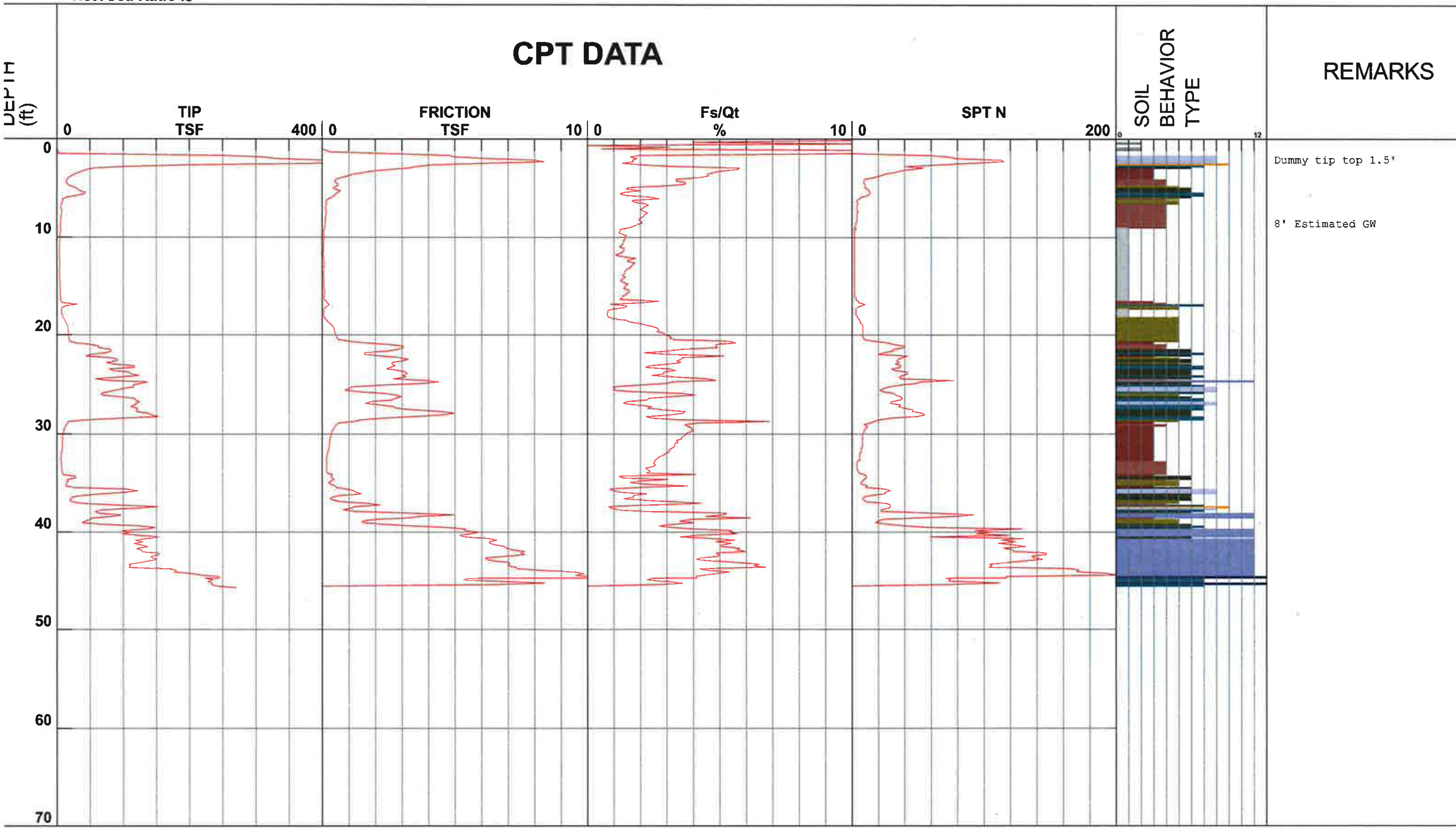
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	30	**Clayey Sand (SC)** medium dense to dense, moist, bluish gray, fine to medium sand	25	MC-10B	72	51		
	35	**Fat Clay (CH)** very stiff, moist, bluish gray with brown mottles, some fine sand, high plasticity						
		Lean Clay with Sand (CL) very stiff, moist, gray, fine sand, low to moderate plasticity	25	SPT-11		27		
	40		21	SPT-12		21		
	45	**Clayey Sand (SC)** dense, moist, gray with reddish brown mottles, fine to medium sand Liquid Limit = 28, Plastic Limit = 17	25	SPT-13		21	11	
		Clayey Sand with Gravel (SC) dense, moist, brown, fine to coarse sand, fine subangular to subrounded gravel	67	MC-14B	126	14		
	50	**Clayey Sand (SC)** dense, moist, light gray brown, fine to medium sand	33	SPT-15		18		
	55		32	SPT-16		18		43
		Clayey Sand with Gravel (SC) dense, moist, brown, fine to coarse sand, fine subangular to subrounded gravel	57	MC-17B	123	13		
	60	Bottom of Boring at 60.0 feet.						

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(246).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-01	Date and Time	4/26/2014 9:18:00 AM	Maximum Depth	45.77 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA

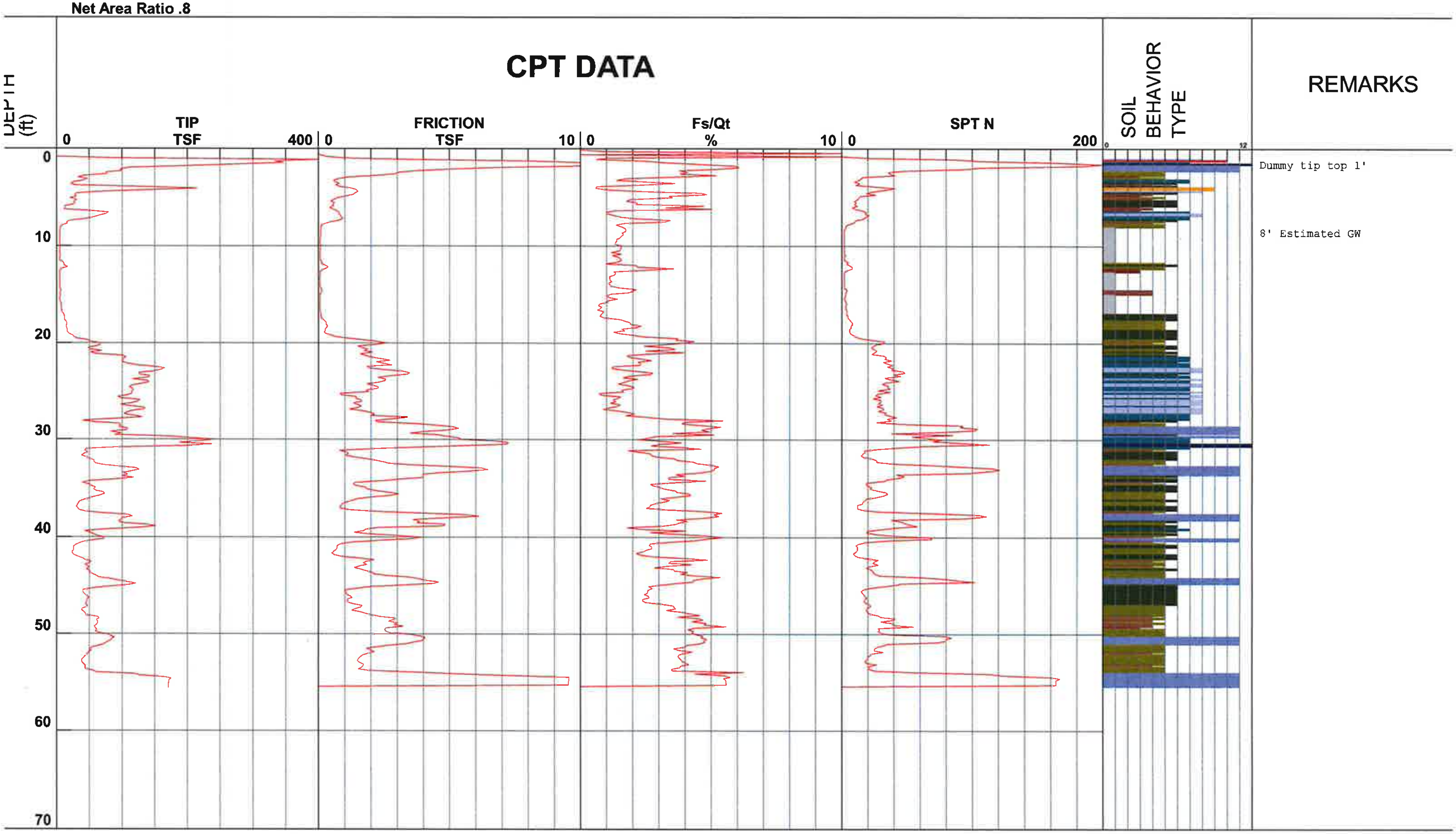


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(244).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-02	Date and Time	4/26/2014 7:48:27 AM	Maximum Depth	55.61 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

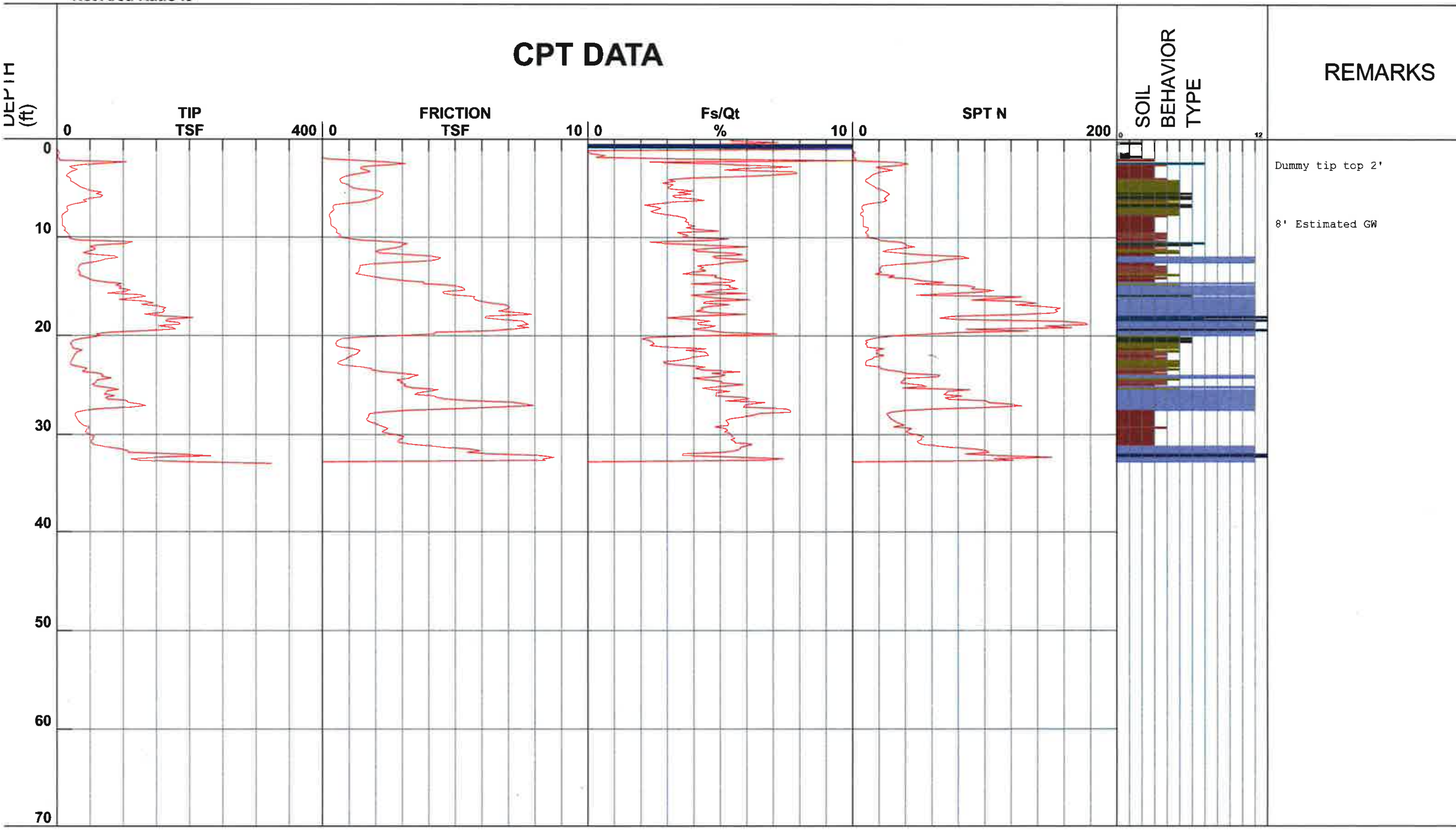


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(254).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-03	Date and Time	4/27/2014 8:06:16 AM	Maximum Depth	32.97 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

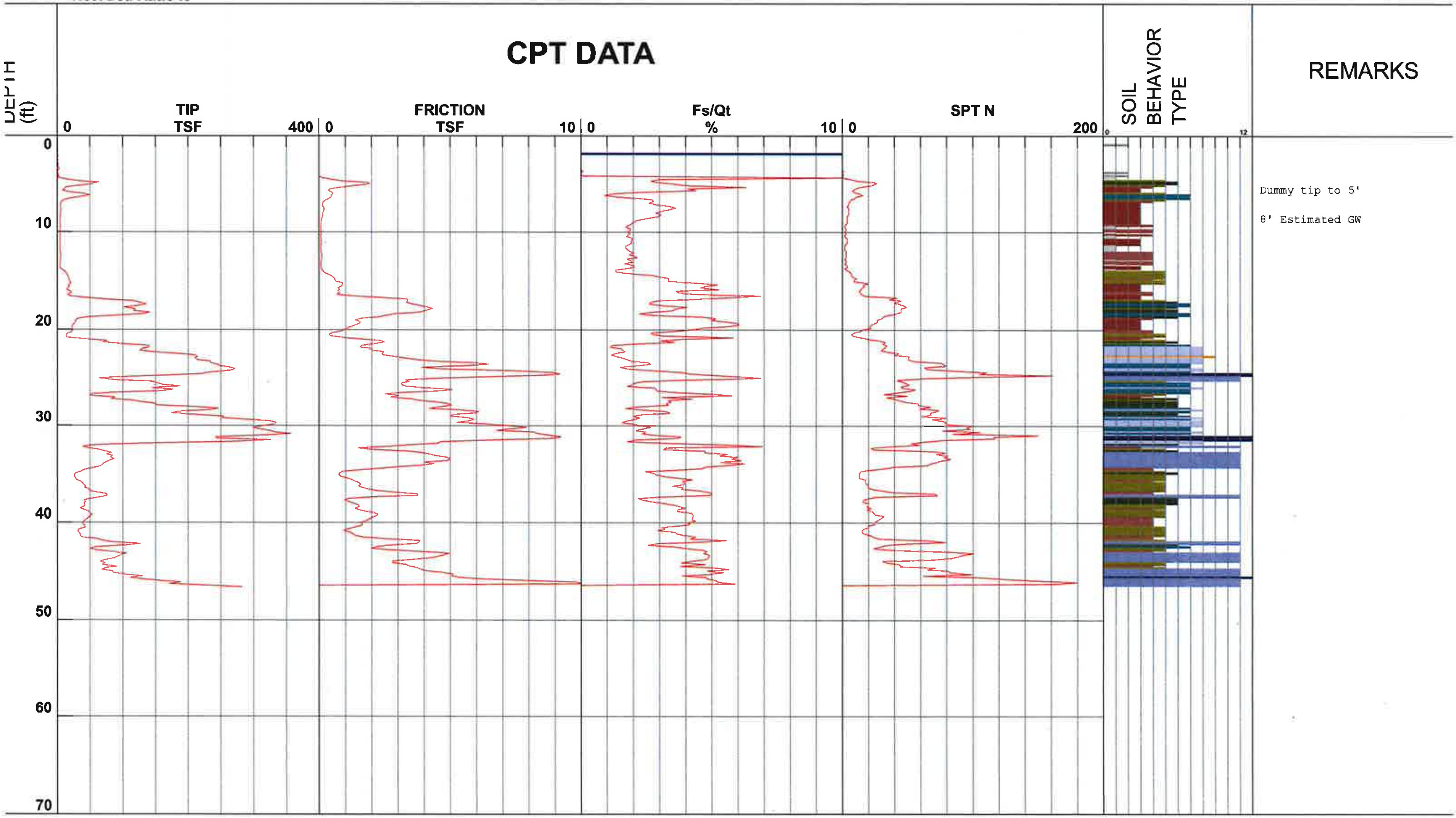


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(251).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-04	Date and Time	4/26/2014 1:10:09 PM	Maximum Depth	46.59 ft
EST GW Depth During Test	0.00 ft				

Net Area Ratio .8

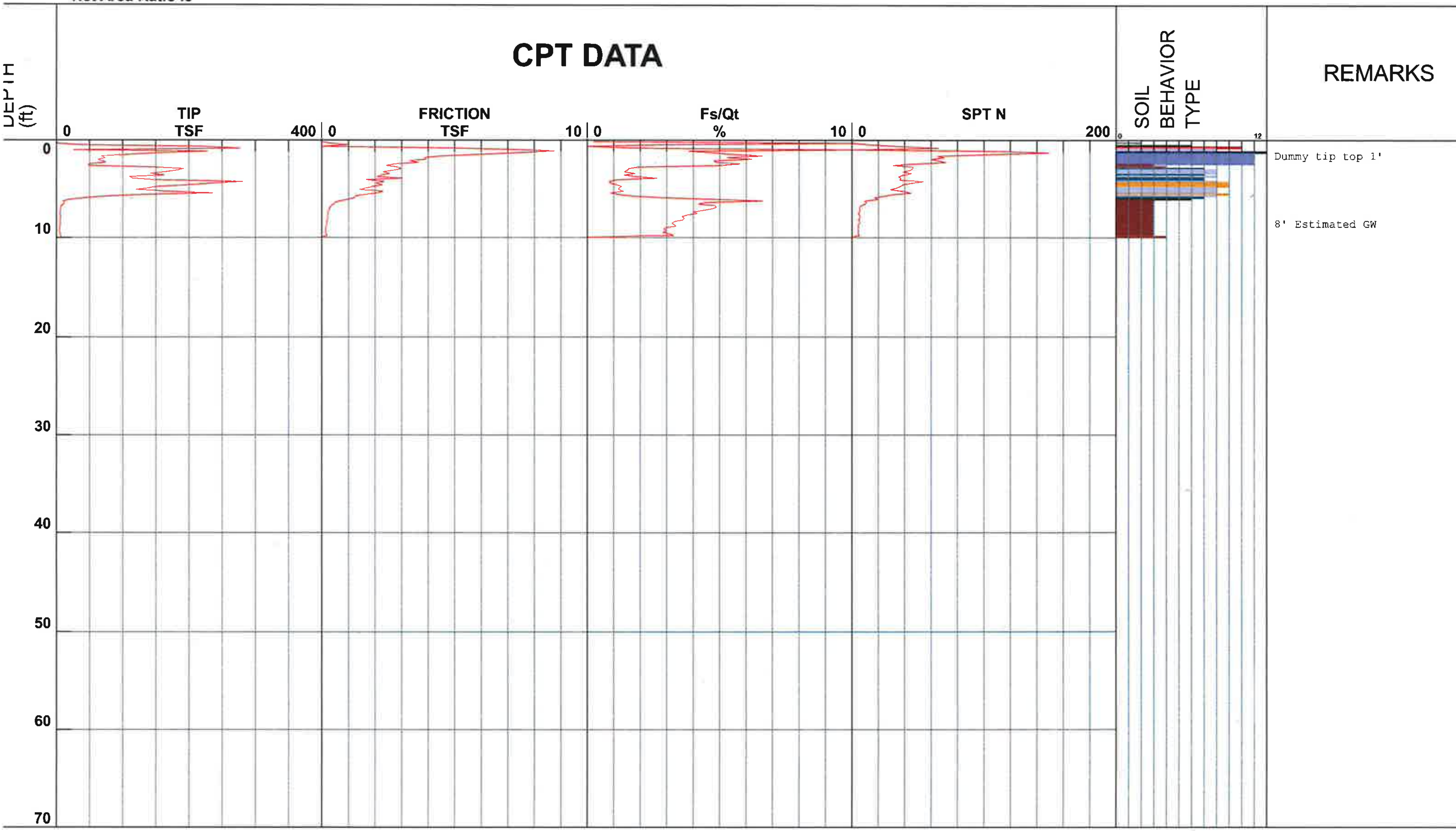


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(247).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-05	Date and Time	4/26/2014 10:12:59 AM	Maximum Depth	10.17 ft
EST GW Depth During Test	0.00 ft				

Net Area Ratio .8



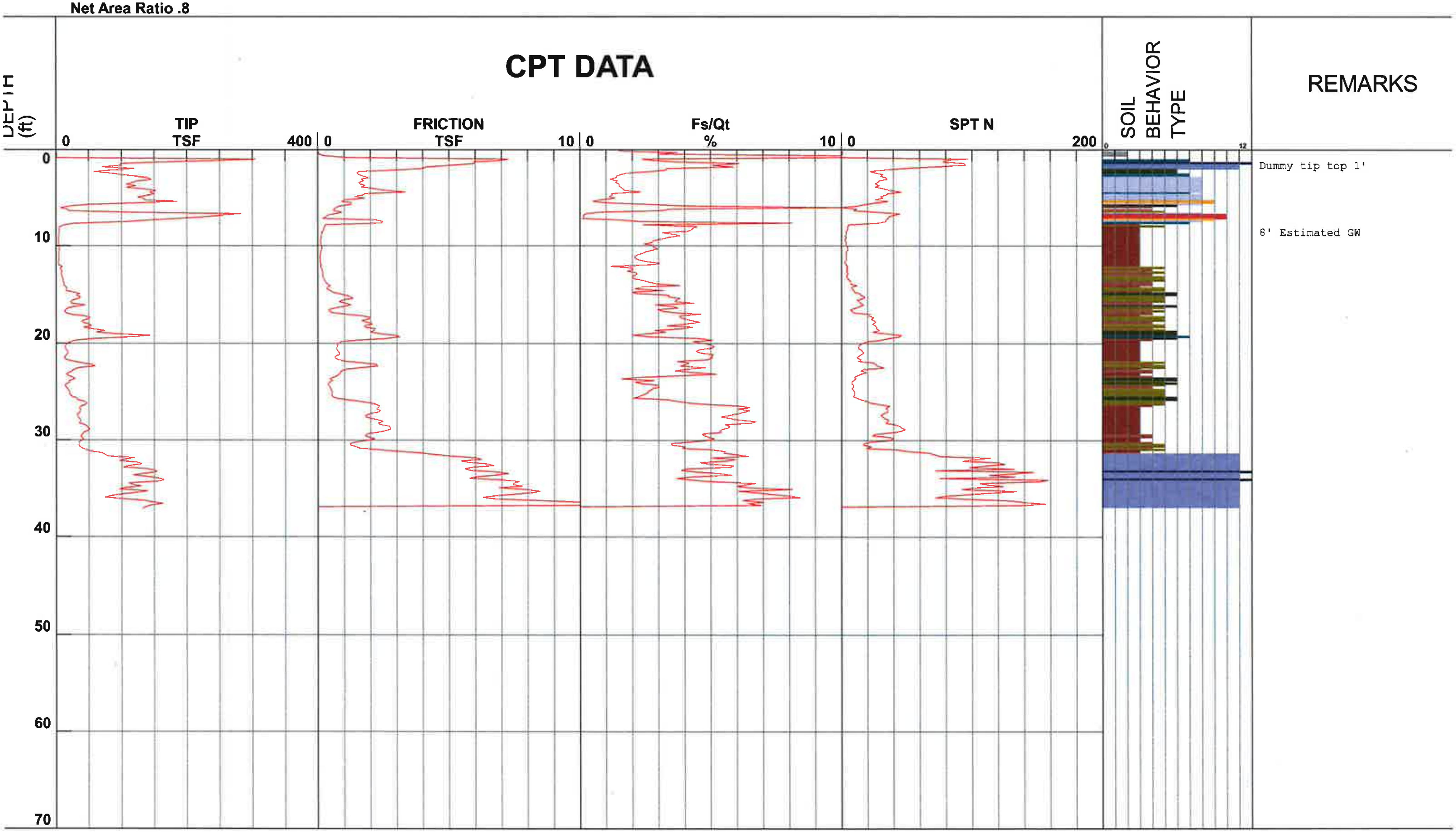
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(248).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-05A	Date and Time	4/26/2014 10:35:42 AM	Maximum Depth	37.07 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA

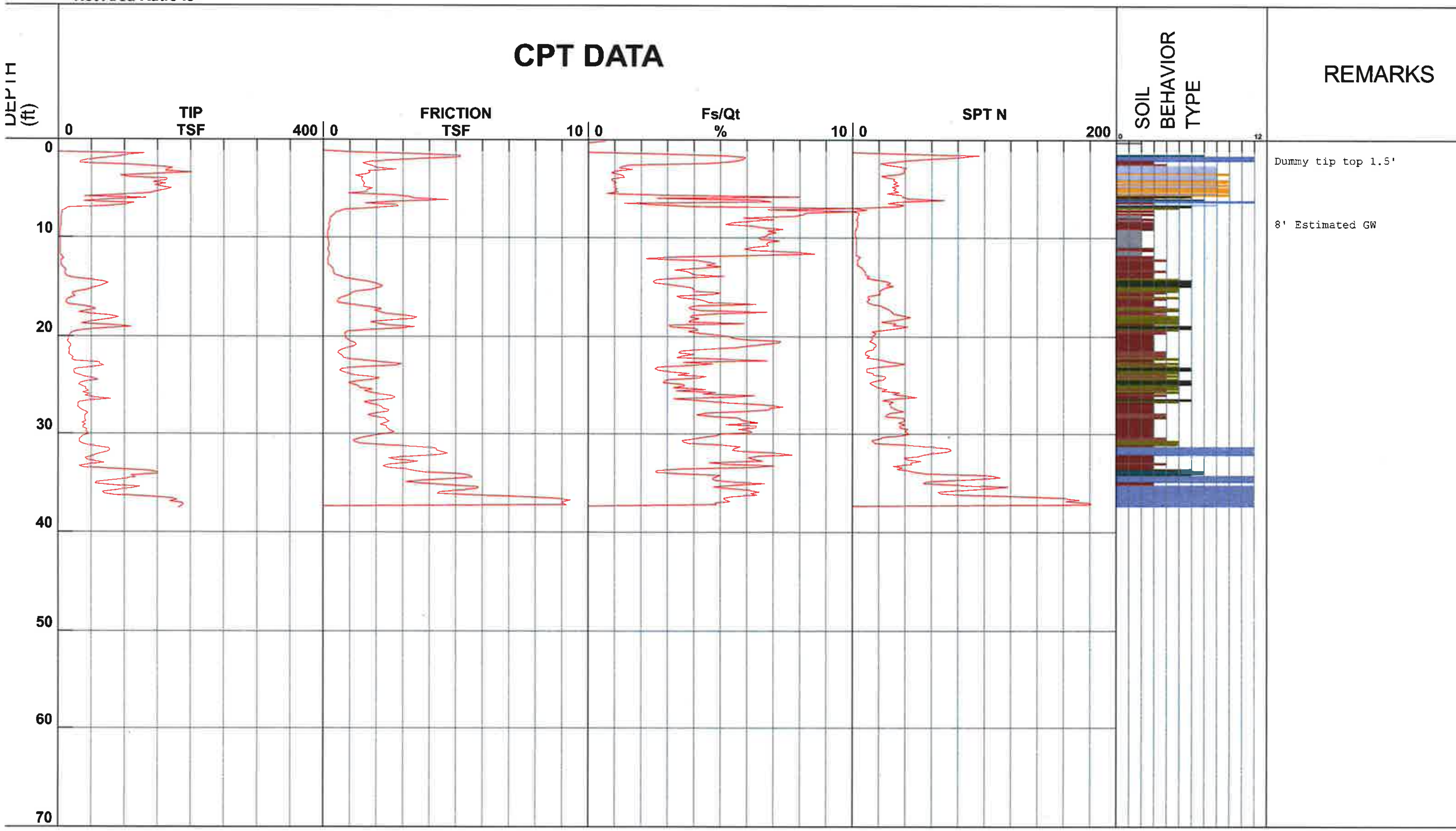


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(249).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-05B	Date and Time	4/26/2014 11:25:42 AM	Maximum Depth	37.57 ft
EST GW Depth During Test	0.00 ft				

Net Area Ratio .8

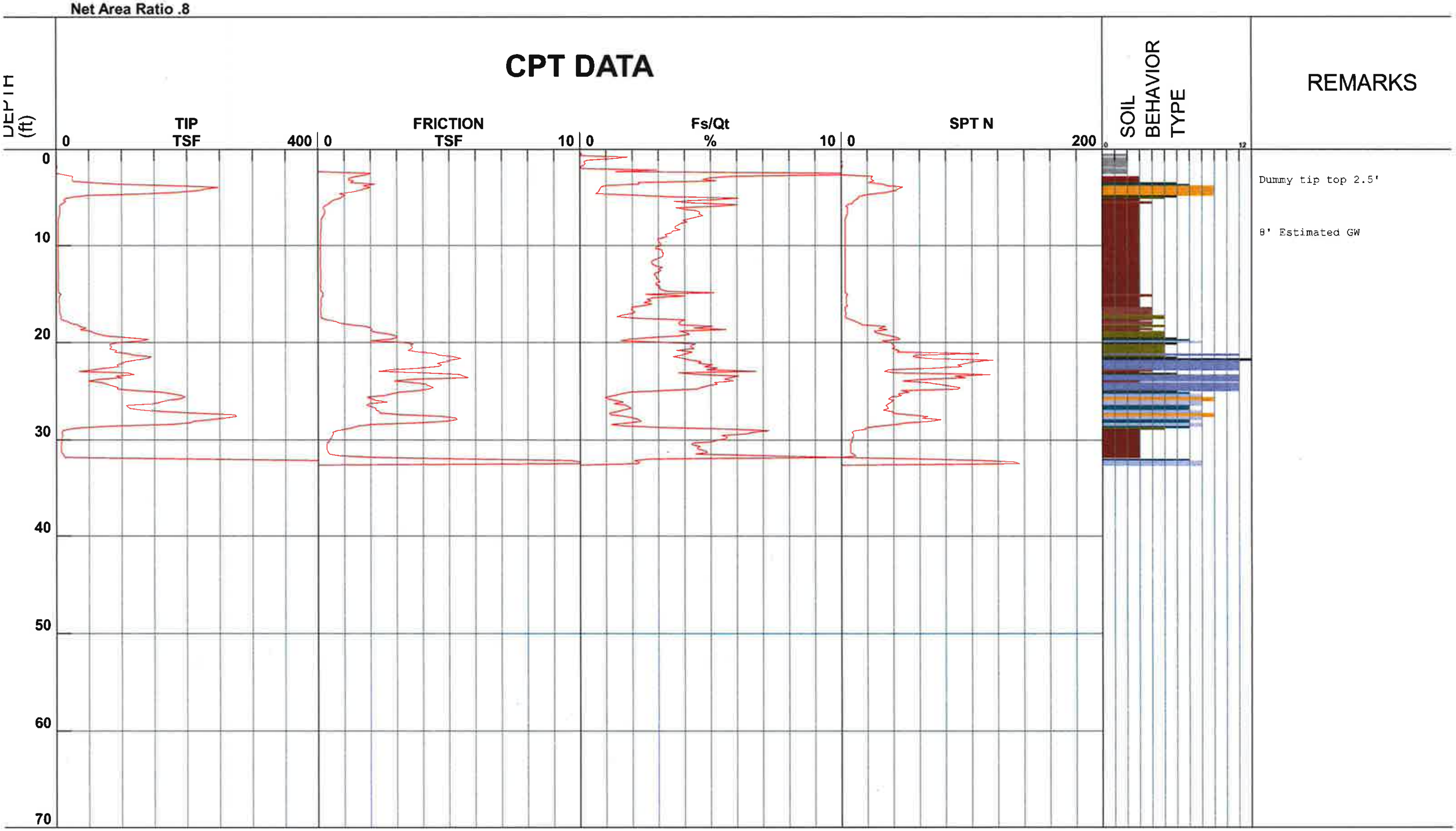


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

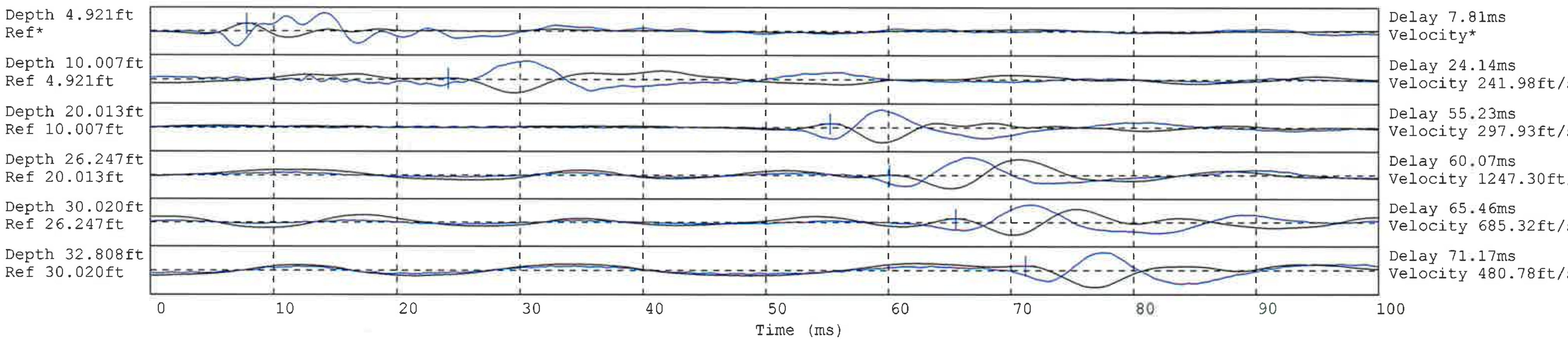
Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(252).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-06	Date and Time	4/26/2014 2:01:12 PM	Maximum Depth	32.81 ft
EST GW Depth During Test			0.00 ft		

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)



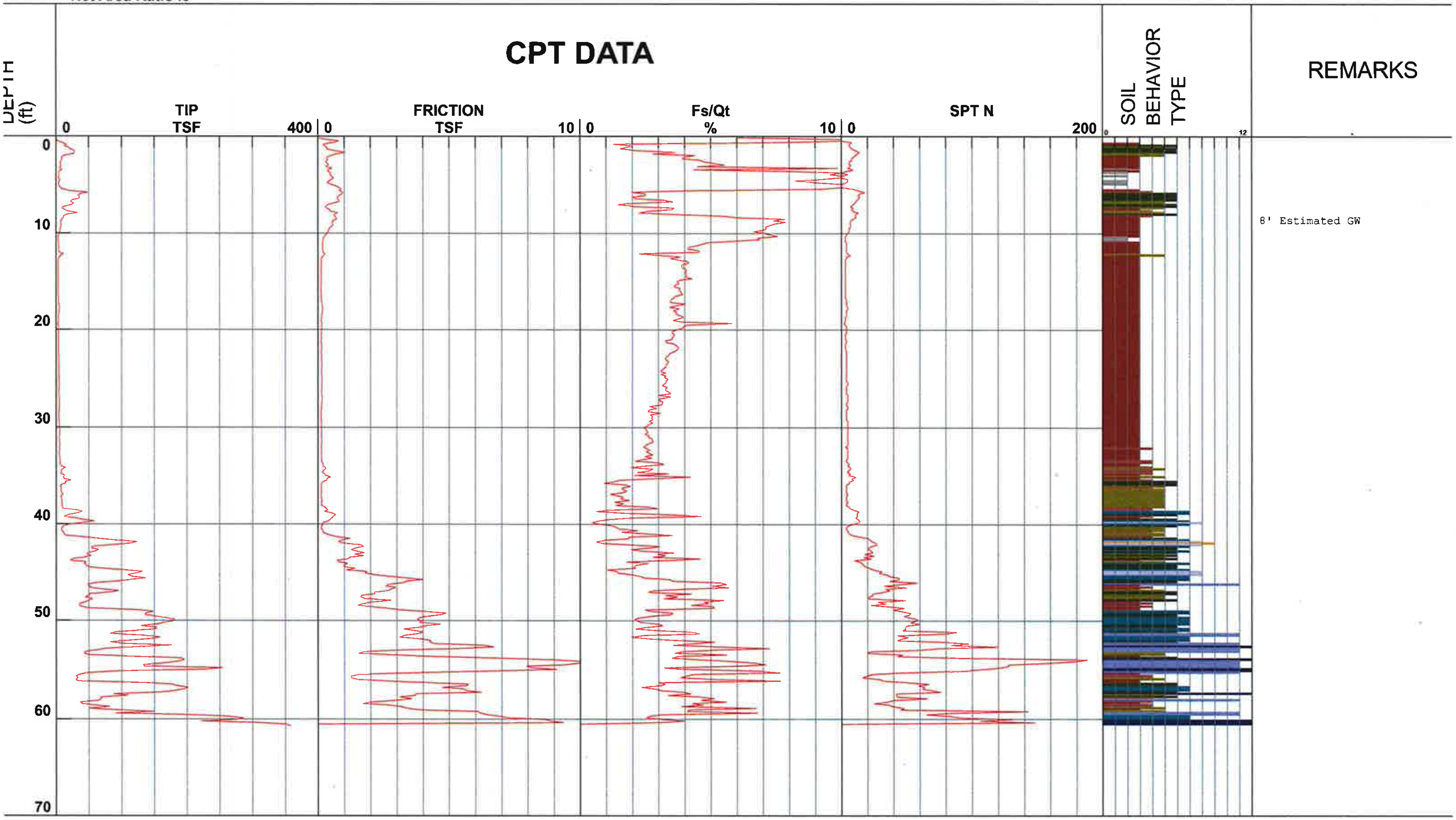
Hammer to Rod String Distance 1.778 (m)
* = Not Determined

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(257).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-07	Date and Time	4/27/2014 10:58:11 AM	Maximum Depth	60.69 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



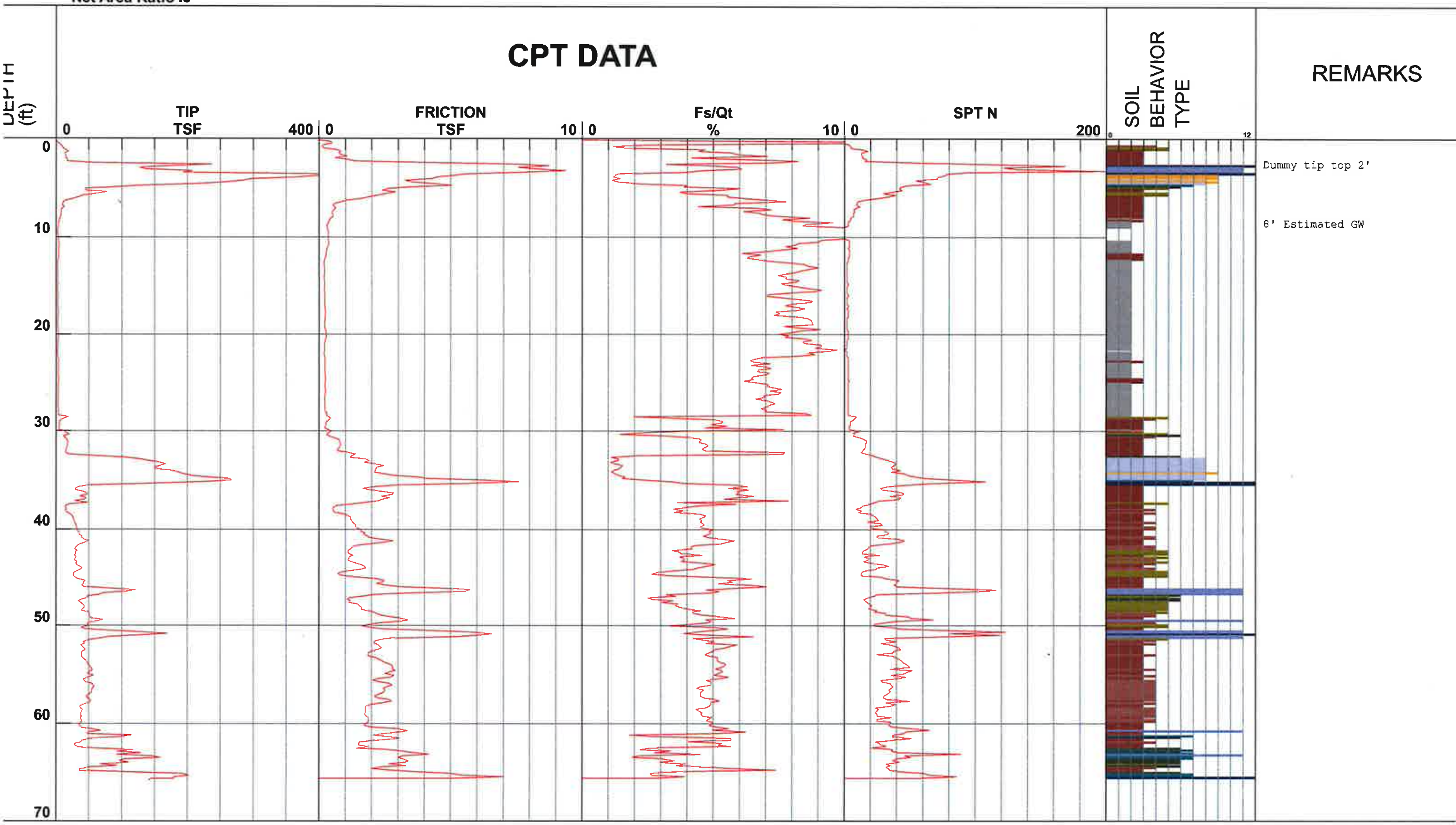
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(256).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-08	Date and Time	4/27/2014 9:45:43 AM	Maximum Depth	65.78 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



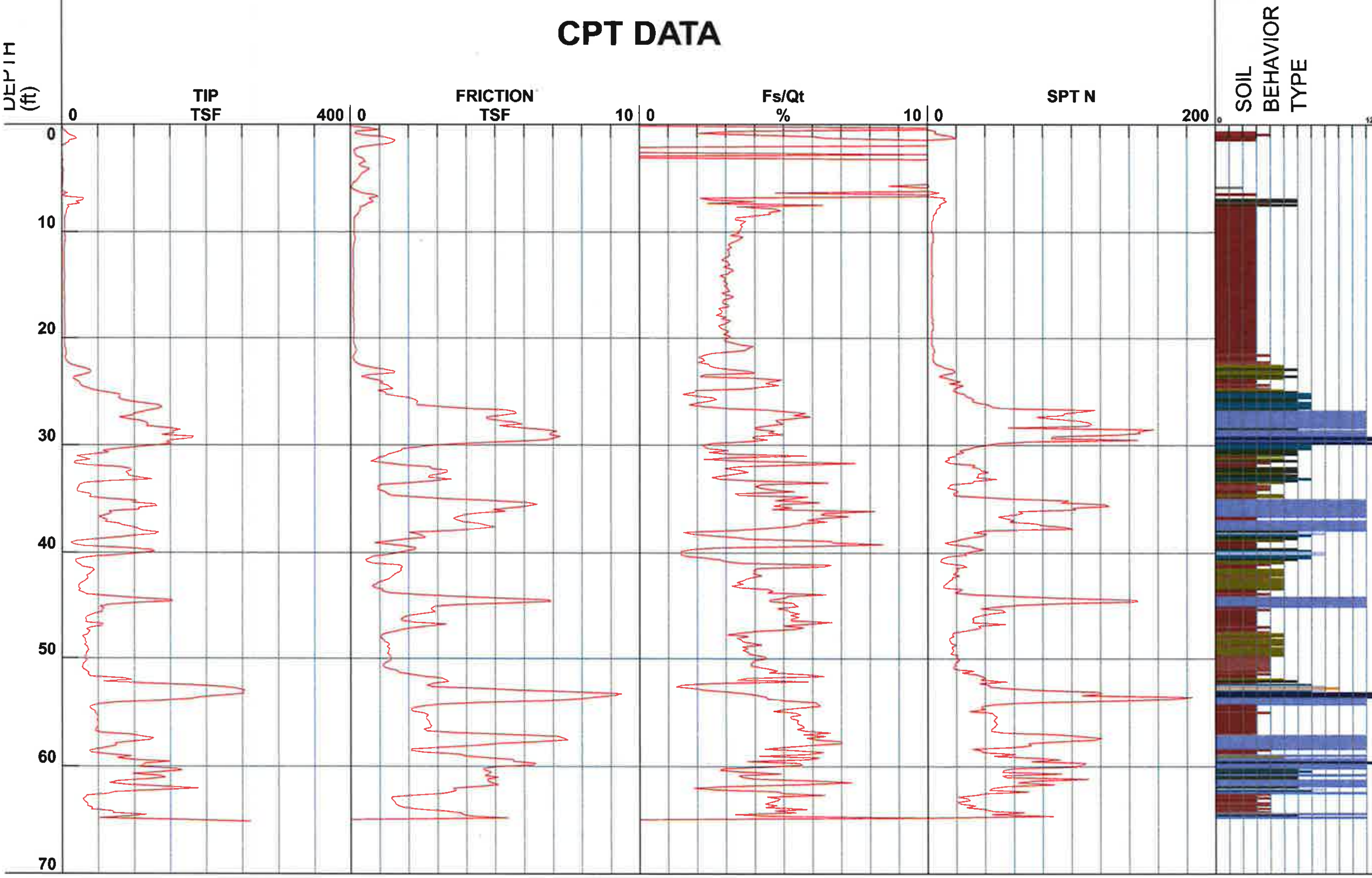
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(258).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-09	Date and Time	4/27/2014 12:04:41 PM	Maximum Depth	65.12 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



REMARKS

Dummy tip top 6'
8' Estimated GW

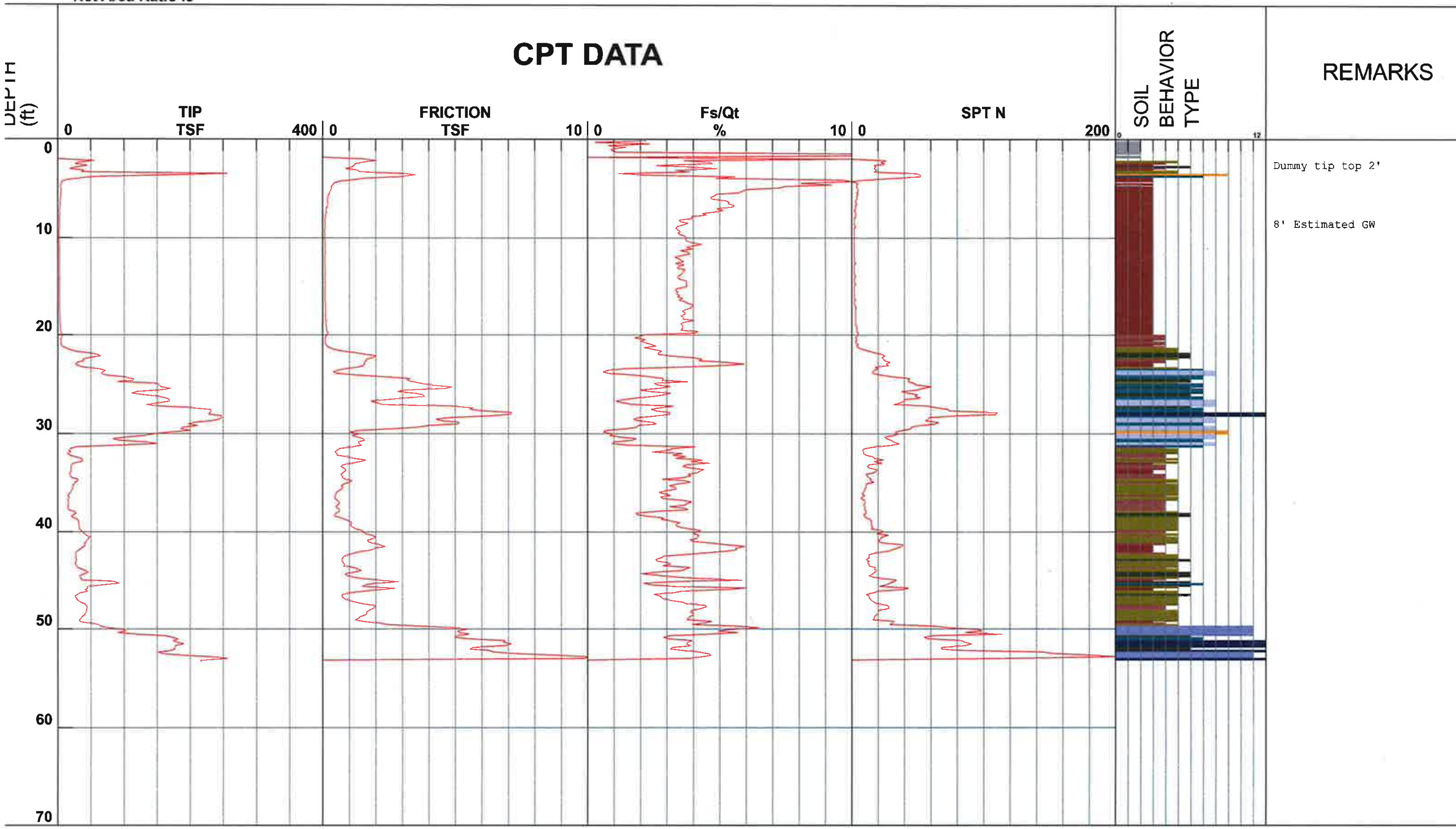
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(253).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-10	Date and Time	4/26/2014 2:59:32 PM	Maximum Depth	53.31 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA

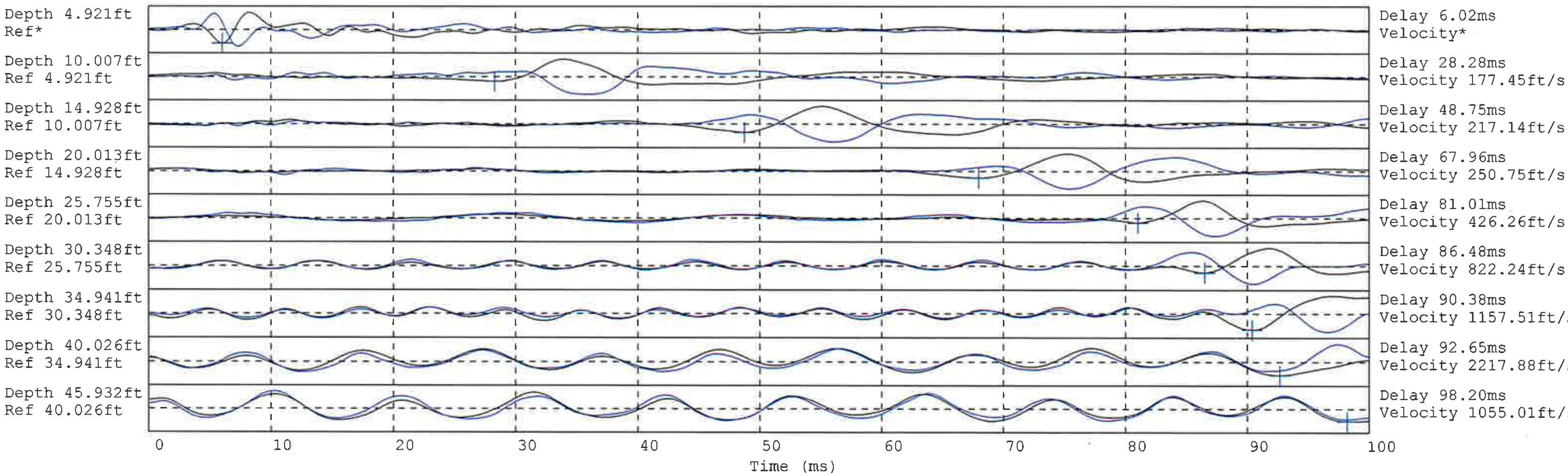


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cornerstone Earth Group

CPT-10

Millbrae TOD



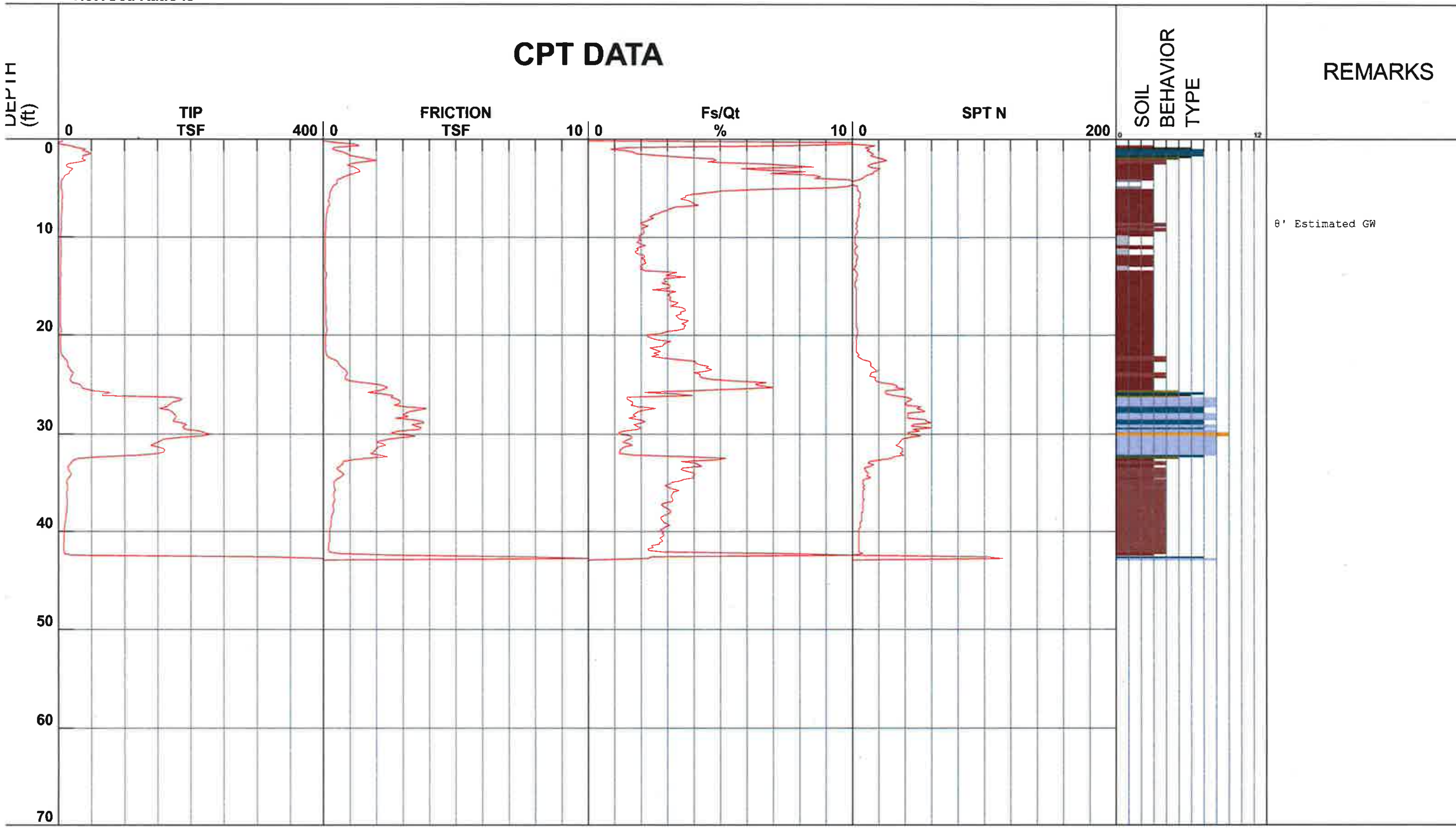
Hammer to Rod String Distance 1.778 (m)
* = Not Determined

Cornerstone Earth Group

Project	Millbrae TOD	Operator	CB/MM	Filename	SDF(255).cpt
Job Number	183-5-2	Cone Number	DDG1298	GPS	
Hole Number	CPT-11	Date and Time	4/27/2014 9:00:05 AM	Maximum Depth	43.14 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on ninety-seven samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on fifty-five samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

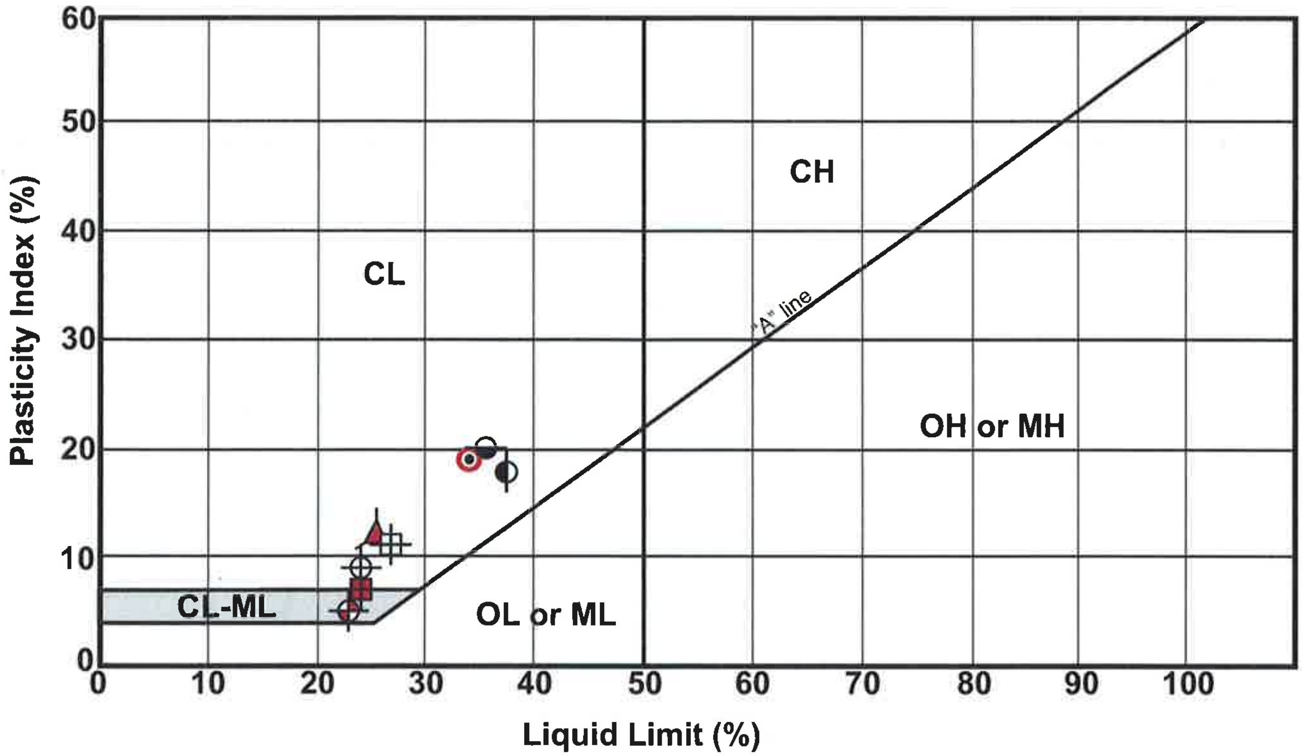
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on seventeen samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Ten Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Undrained-Unconsolidated Triaxial Shear Strength: The undrained shear strength was determined on ten relatively undisturbed samples by unconsolidated-undrained triaxial shear strength testing (ASTM D2850). The results of this test are included as part of this appendix.

Consolidation: Five consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation tests are presented graphically in this appendix.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
	EB-1	20.5	19	23	18	5	32	Silty, Clayey Sand (SC-SM)
	EB-1	34.0	20	24	17	7	39	Silty, Clayey Sand (SC-SM)
	EB-2	2.0	16	34	15	19	—	Sandy Lean Clay (CL) [Fill]
	EB-2	29.5	15	34	15	19	47	Clayey Sand (SC)
	EB-3	2.0	12	25	13	12	—	Clayey Sand (SC) [Fill]
	EB-3	14.5	22	37	19	18	18	Fat Clay with Sand (CH) [Fill]
	EB-4	38.5	30	determined non-plastic			24	Silty Sand (SM)
	EB-6	2.0	33	35	15	20	—	Sandy Lean Clay (CL) [Fill]
	EB-6	23.5	18	24	16	8	—	Clayey Sand (SC)
	EB-6	44.0	21	28	17	11	—	Clayey Sand (SC)

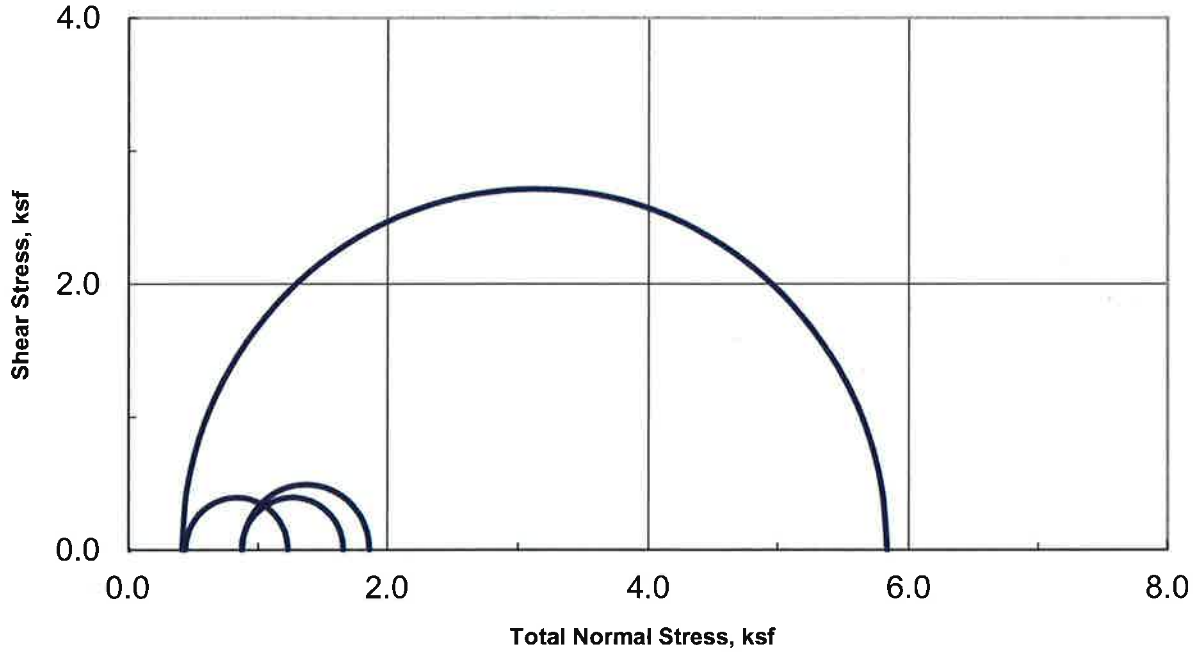
Samples prepared in accordance with ASTM D421

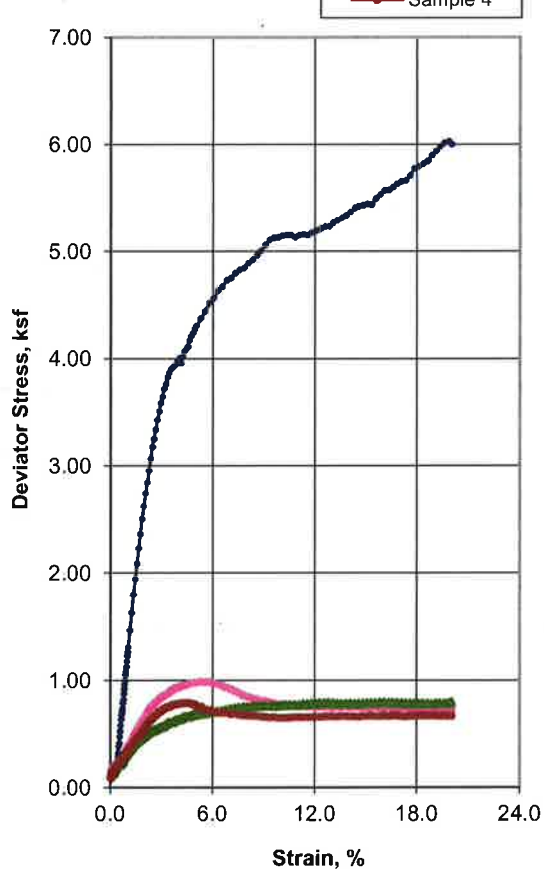
937 Commercial Street
Palo Alto, CA 94303

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data	1	2	3	4
Moisture %	23.2	90.0	76.1	89.5
Dry Den,pcf	99.7	48.0	54.1	48.6
Void Ratio	0.690	2.512	2.114	2.468
Saturation %	90.6	96.7	97.2	97.9
Height in	5.08	6.10	5.02	6.09
Diameter in	2.40	2.86	2.41	2.87
Cell psi	2.9	6.1	3.1	6.1
Strain %	15.00	5.29	15.00	4.54
Deviator, ksf	5.424	0.985	0.787	0.789
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.051	0.061	0.050	0.061
Job No.:	640-673			
Client:	Cornerstone Earth Group			
Project:	Millbrae BART-TOD - 183-5-2			
Boring:	EB-1	EB-1	EB-5	EB-5
Sample:	3B	5	3A	6
Depth ft:	6.0	10(Tip-10")	5.5	10(Tip-10")

Visual Soil Description

Sample #	
1	Dark Gray SILT (slightly plastic)***
2	Gray CLAY w/ pockets of Silty Sand (Bay Mud)
3	Gray Organic CLAY (Bay Mud)
4	Gray Organic CLAY (Bay Mud) w/ pockets of Si Sand

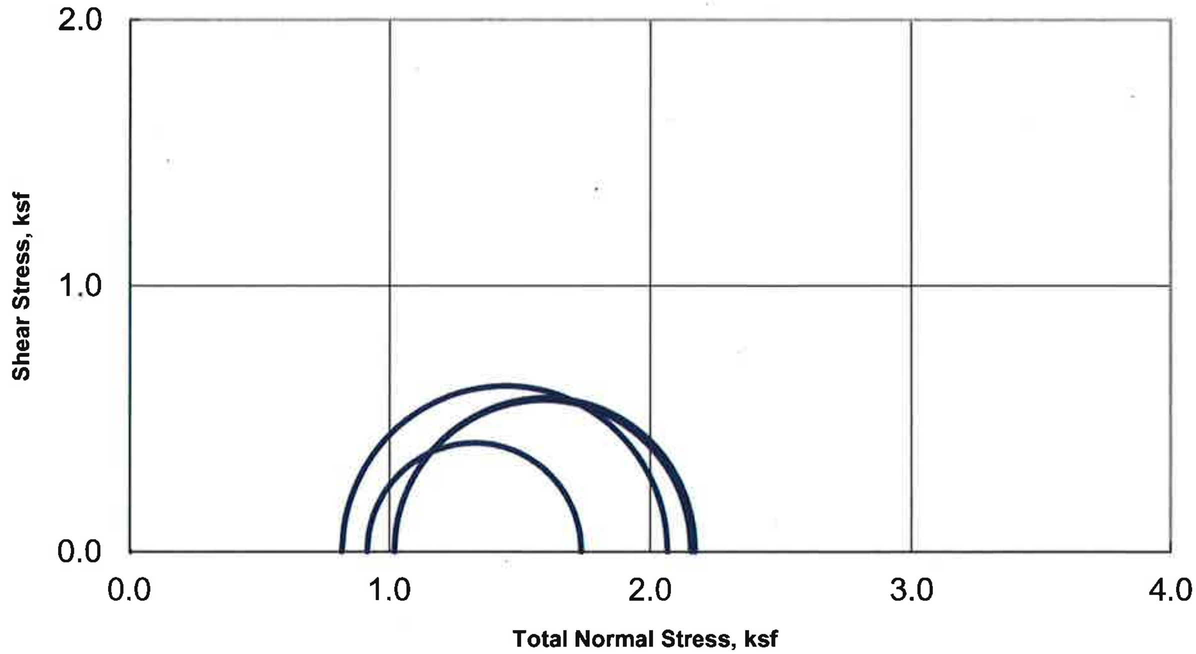
Remarks:***change to Gray CLAY w/ Gravel (Weathered Rock)

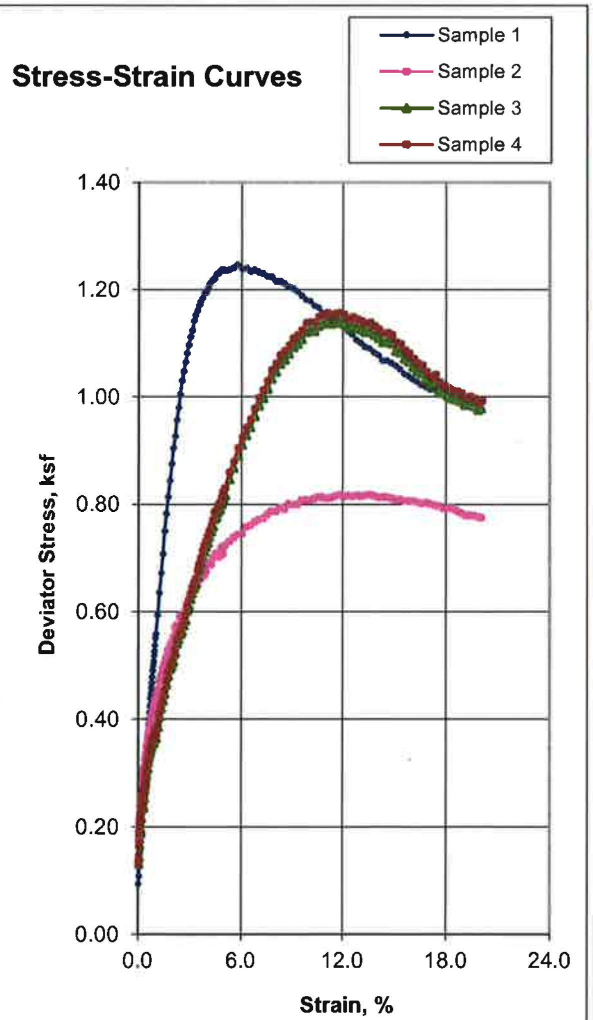
Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

937 Commercial Street
Palo Alto, CA 94303

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data	1	2	3	4
Moisture %	77.9	101.1	83.0	52.7
Dry Den,pcf	50.8	44.2	50.8	70.5
Void Ratio	2.318	2.817	2.315	1.478
Saturation %	90.7	96.9	96.8	99.8
Height in	6.08	5.02	5.01	4.99
Diameter in	2.86	2.39	2.41	2.39
Cell psi	5.7	6.4	7.1	7.1
Strain %	5.78	13.56	11.06	11.10
Deviator, ksf	1.246	0.818	1.140	1.157
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.060	0.050	0.050	0.050
Job No.:	640-678a			
Client:	Cornerstone Earth Group			
Project:	Millbrae TOD - 183-5-2			
Boring:	EB-4	EB-4	EB-4	EB-4
Sample:	5	7B	8A	10B
Depth ft:	12(Tip-5")	19.5	24	34.5

Visual Soil Description	
Sample #	
1	Greenish Gray Organic CLAY (Bay Mud)
2	Dark Gray Organic CLAY (Bay Mud)
3	Dark Gay Organic CLAY (Bay Mud)
4	Dark Gray CLAY w/ organics (Bay Mud)

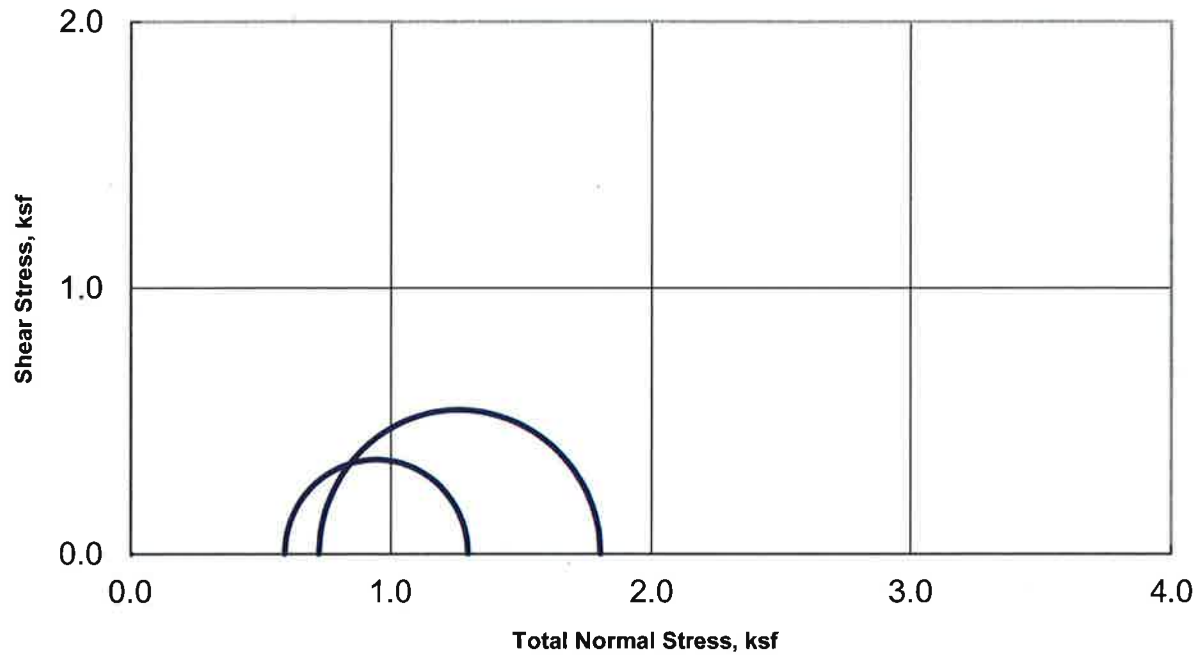
Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

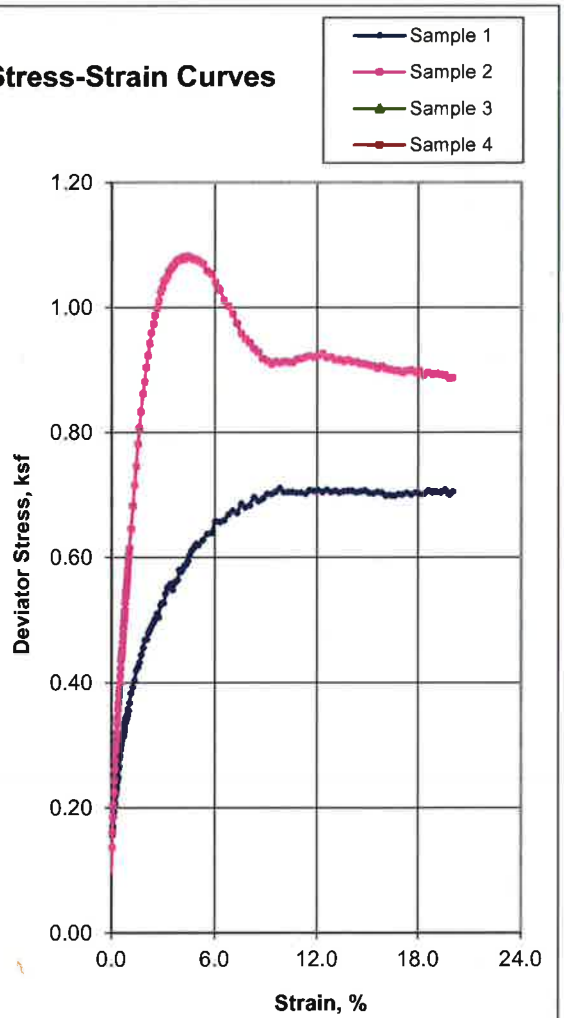
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test

ASTM D2850



Stress-Strain Curves



Sample Data	1	2	3	4
Moisture %	83.7	89.1		
Dry Den,pcf	50.8	49.3		
Void Ratio	2.320	2.422		
Saturation %	97.4	99.3		
Height in	5.01	6.09		
Diameter in	2.41	2.87		
Cell psi	4.1	5.0		
Strain %	9.80	4.44		
Deviator, ksf	0.710	1.082		
Rate %/min	1.00	1.00		
in/min	0.050	0.061		
Job No.:	640-678b			
Client:	Cornerstone Earth Group			
Project:	Millbrae TOD - 183-5-2			
Boring:	EB-6	EB-6		
Sample:	4B	6		
Depth ft:	9.5	15(Tip-5")		

Visual Soil Description

Sample #	
1	Gray Organic CLAY (Bay Mud)
2	Greenish Gray CLAY (Bay Mud)
3	
4	

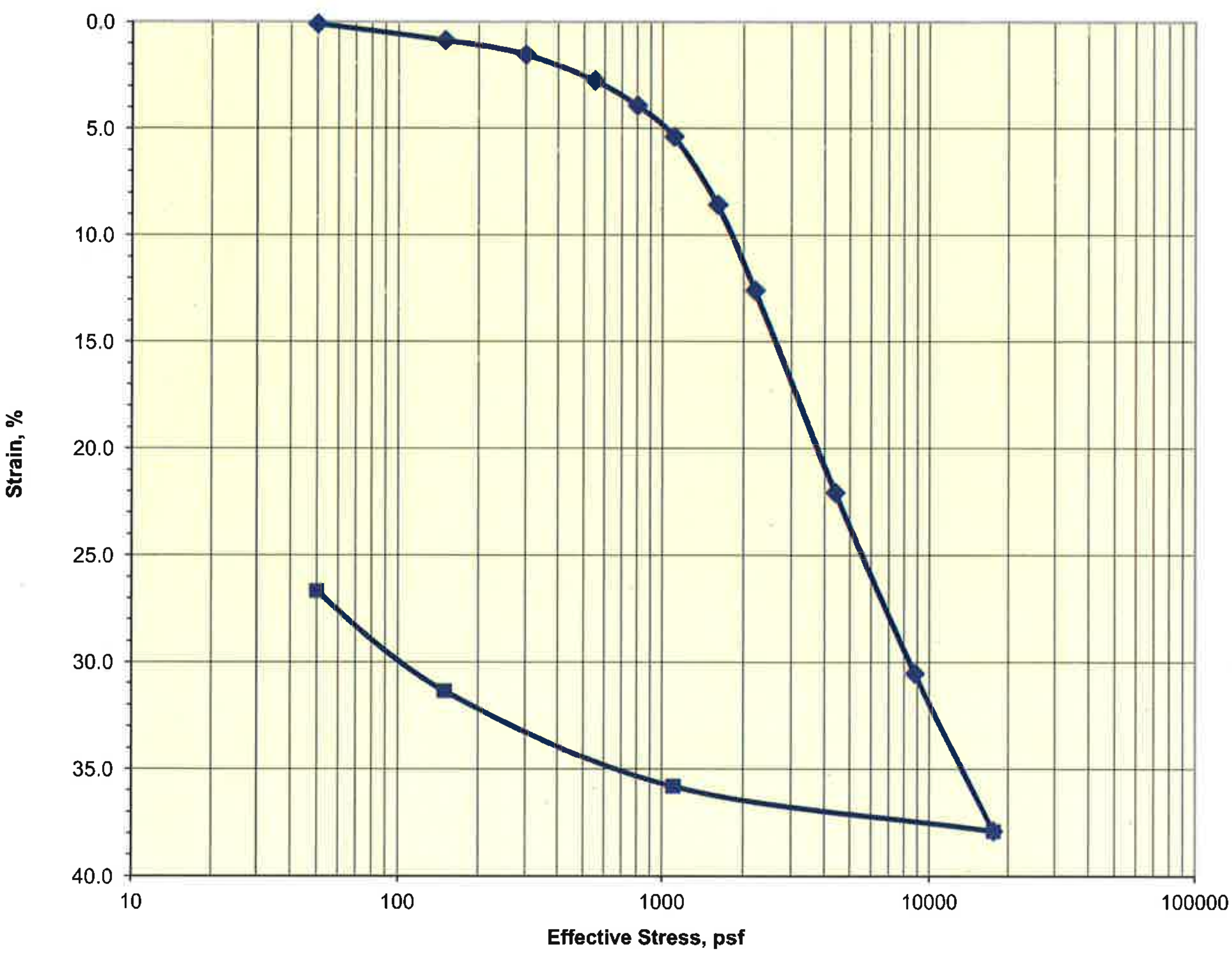
Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

Consolidation Test
ASTM D2435

Job No.:	640-673	**Boring:**	EB-1	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	5	**Reduced:**	PJ
Project:	Millbrae BART-TOD - 183-5-2	**Depth, ft.:**	10(Tip-9")	**Checked:**	PJ/DC
Soil Type:	Gray CLAY w/ pockets of Silty Sand (Bay Mud)			**Date:**	5/15/14

Strain-Log-P Curve



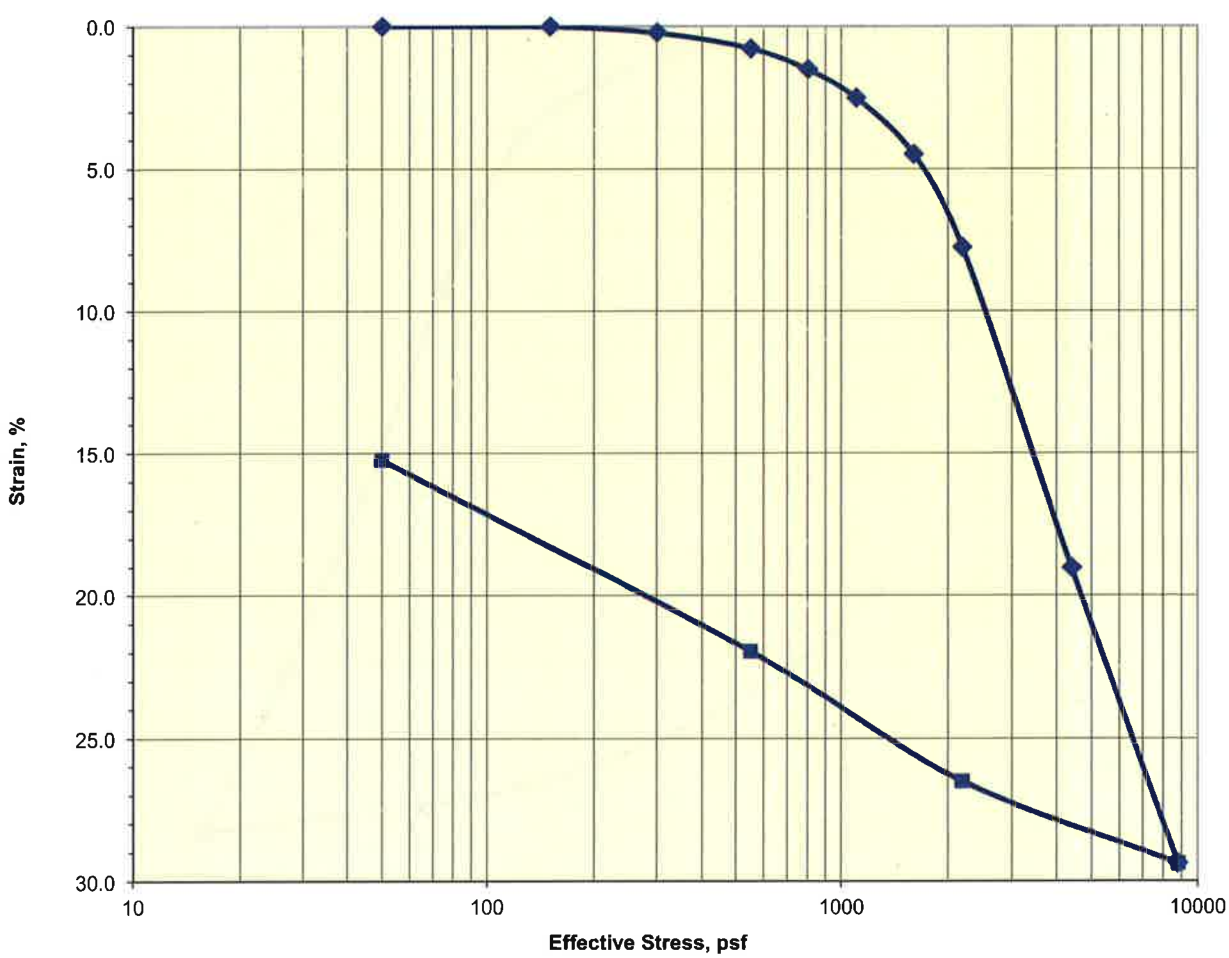
Assumed Gs 2.75	Initial	Final
Moisture %:	78.2	49.9
Dry Density, pcf:	53.6	72.3
Void Ratio:	2.204	1.373
% Saturation:	97.6	100.0

Remarks:

Consolidation Test
ASTM D2435

Job No.:	640-678	**Boring:**	EB-4	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	5	**Reduced:**	PJ
Project:	Millbrae TOD - 183-5-2	**Depth, ft.:**	12-14(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray Organic CLAY (Bay Mud)			**Date:**	5/22/14

Strain-Log-P Curve



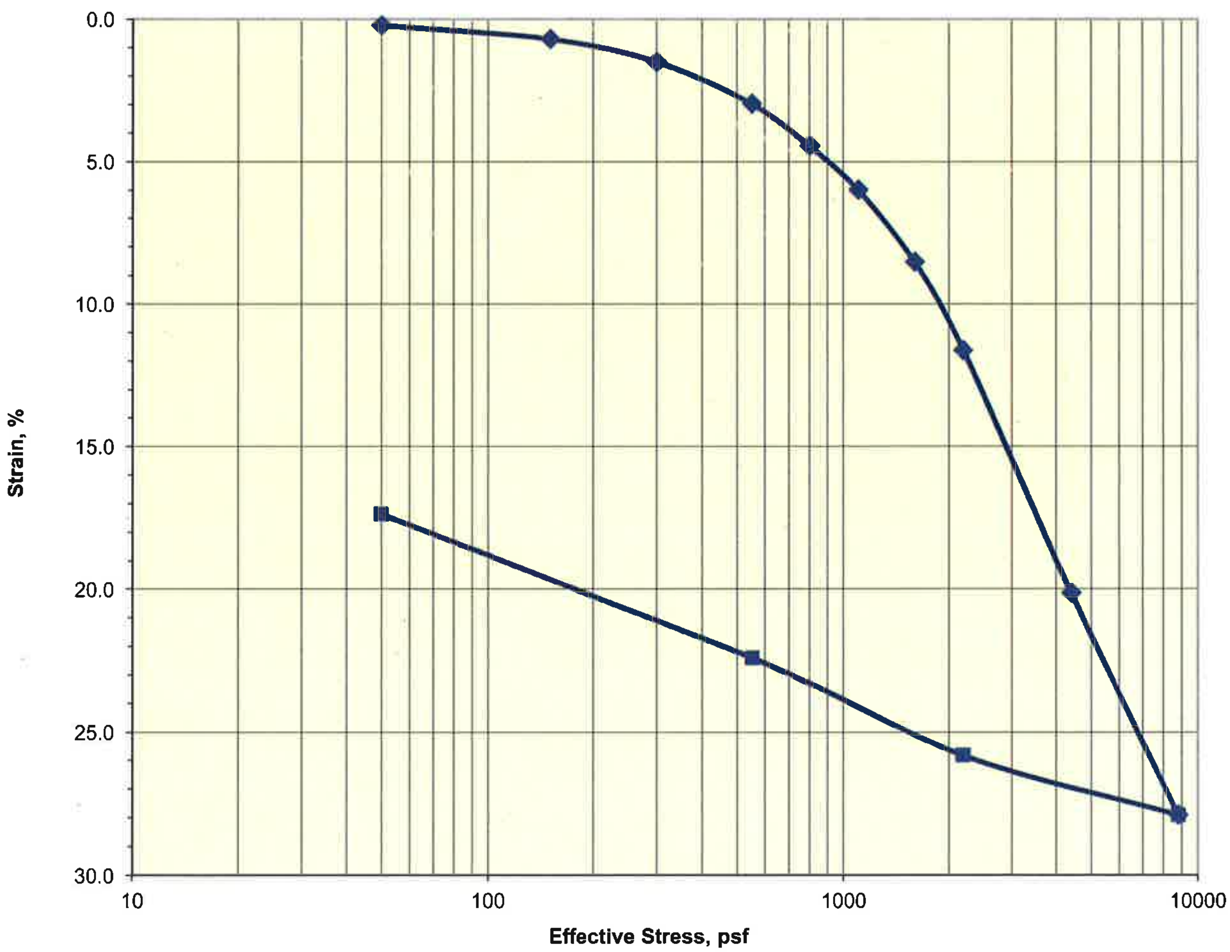
Assumed Gs 2.65	Initial	Final
Moisture %:	108.6	87.9
Dry Density, pcf:	42.5	49.7
Void Ratio:	2.896	2.330
% Saturation:	99.3	100.0

Remarks:

Consolidation Test
ASTM D2435

Job No.:	640-678	**Boring:**	EB-4	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	8b	**Reduced:**	PJ
Project:	Millbrae TOD - 183-5-2	**Depth, ft.:**	24.5	**Checked:**	PJ/DC
Soil Type:	Dark Gray Organic CLAY (Bay Mud)			**Date:**	5/22/14

Strain-Log-P Curve

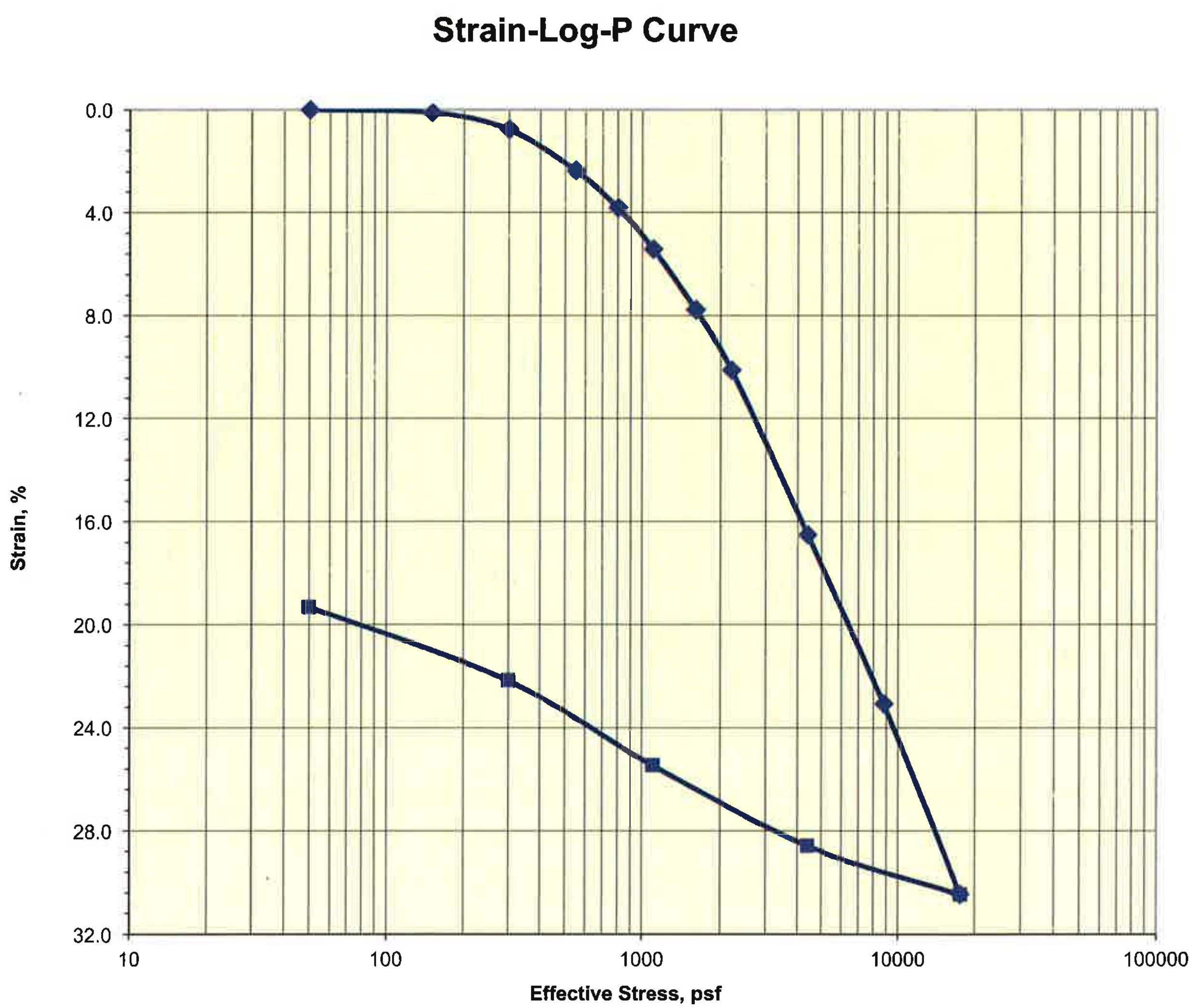


Assumed Gs 2.7	Initial	Final
Moisture %:	74.7	56.5
Dry Density, pcf:	55.6	66.7
Void Ratio:	2.032	1.527
% Saturation:	99.3	100.0

Remarks:

Consolidation Test
ASTM D2435

Job No.:	640-673	**Boring:**	EB-5	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	3B	**Reduced:**	PJ
Project:	Millbrae BART-TOD - 183-5-2	**Depth, ft.:**	6	**Checked:**	PJ/DC
Soil Type:	Gray Organic CLAY (Bay Mud)			**Date:**	5/19/14



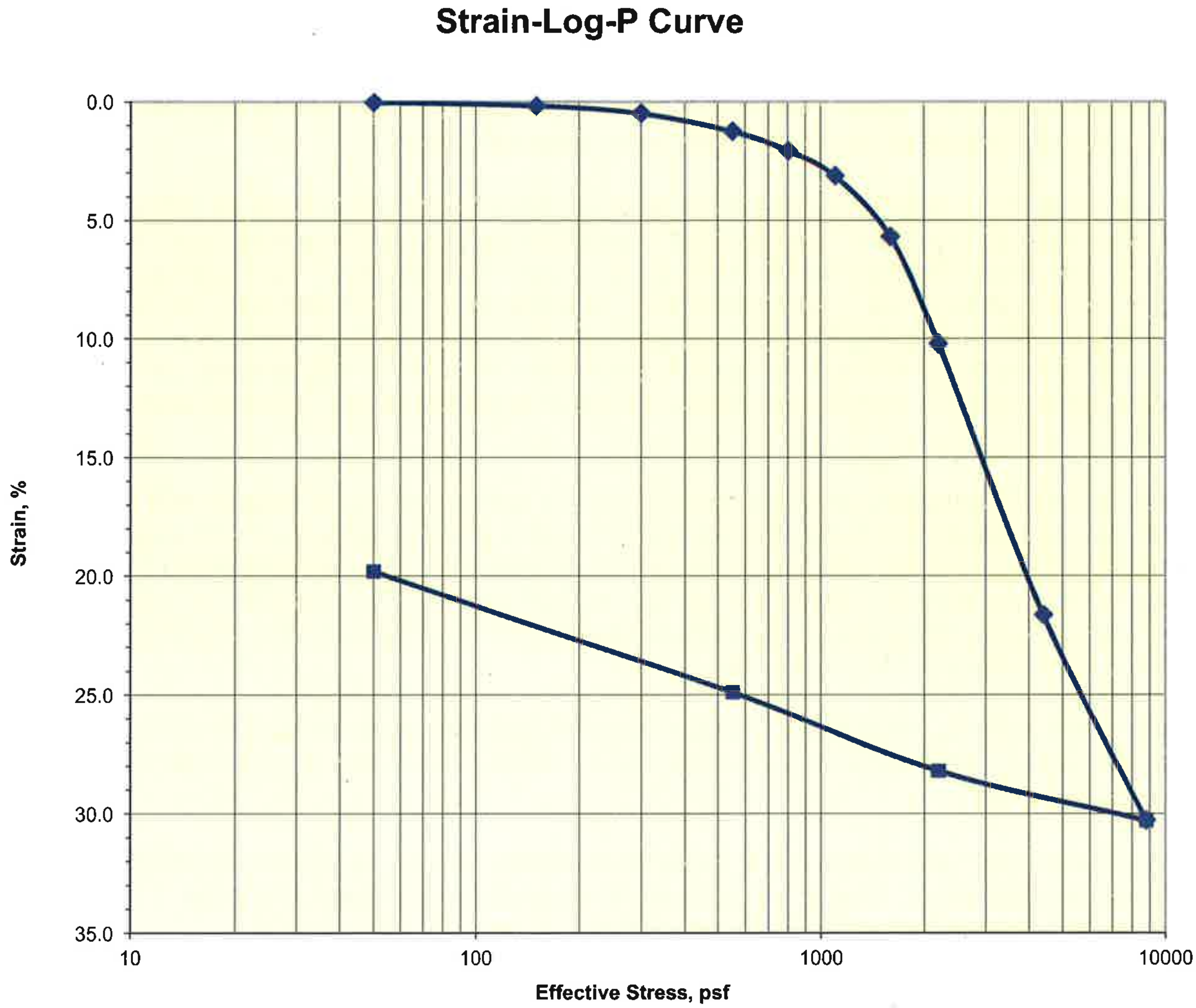
Assumed Gs 2.65	Initial	Final
Moisture %:	80.8	59.9
Dry Density, pcf:	51.8	63.9
Void Ratio:	2.191	1.588
% Saturation:	97.7	100.0

Remarks:

Consolidation Test

ASTM D2435

Job No.:	640-678	**Boring:**	EB-6	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	6	**Reduced:**	PJ
Project:	Millbrae TOD - 183-5-2	**Depth, ft.:**	15-17(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY (Bay Mud)			**Date:**	5/20/14



Assumed Gs 2.8	**Initial**	**Final**
Moisture %:	97.9	71.4
Dry Density, pcf:	46.7	58.3
Void Ratio:	2.744	1.998
% Saturation:	99.9	100.0

Remarks:

APPENDIX C: CONSTRUCTION GUIDELINES ON BAY MUD

INTRODUCTION

The purpose of this document is to provide general observations and guidelines for earthwork construction in the area and to highlight some of the more difficult aspects of earthwork on sites underlain by Bay Mud. It is presented as a supplement to the standard plans and technical specifications normally provided for the project.

GENERAL SOIL AND GROUND WATER CONDITIONS

The site is a former marshland that has been filled to allow development. Fill in the area is generally about 3 to 6½ feet thick and consists of generally medium stiff to very stiff lean clay with varying amounts of sand and loose clayey sand with gravel.

The existing fill is underlain by estuarine deposits consisting of soft to medium stiff fat clay, known locally as Bay Mud. The Bay Mud is generally medium stiff to soft at depth, and over-consolidated due to historic wetting and drying cycles. The Bay Mud "crust" appears to be approximately 2 feet thick in our explorations. The Bay Mud crust is underlain approximately 3 to 6 feet of soft, moderately to highly compressible fat clay to depths ranging from 7 to 11½ feet. Moisture contents of the Bay Mud typically range from about 44 to 199 percent, indicating very high organic content. Interbedded zones of organic peat zones were observed within the Bay Mud.

The Bay Mud is generally underlain by older alluvial soils consisting of medium stiff to very stiff lean clay with varying amounts of sand, medium dense to very dense sand with varying amounts of fines and gravel to the maximum depth explored in our explorations to 110 feet. A couple Plasticity Index (PI) tests were performed from borings EB-1 and EB-2 at 15½ and 10 feet, respectively, below the ground surface resulting in PIs of 34 and 15, indicating low to high expansion potential of the alluvial soils.

Ground water exists within the Bay Mud although it generally does not appear as free water. Instead, ground water tends to seep slowly out of the Bay Mud. The ground water often appears brackish.

Ground water can also collect in the upper fill materials, where it perches upon the lower clayey fills and Bay Mud crust. Due to the granular fill materials encountered at the site, it is possible that seasonally perched water will be encountered in the upper fill materials.

For design purposes, ground water is generally considered to be at about 2½ to 9½ feet below current grades.

TRENCHES AND EXCAVATIONS

Trenching specifications for Bay Mud sites are usually restrictive about excavation methods and shoring requirements. All contractors should carefully review the technical specifications provided for the project. The following observations and guidelines are presented as a supplement to the technical specifications:

1. The fill should be segregated when performing excavations. This can be accomplished by placing the fill on one side of the trench and excavated Bay Mud on the other.

2. Bay Mud requires significant drying and processing time to be reused as compacted fill. Bay Mud must be spread in thin layers and disked or turned to facilitate drying such that the material may be properly compacted.

3. Trenches that extend into the Bay Mud should be backfilled as soon as possible after placement of utilities to prevent base heave or trench sloughing. Significant lateral movements of excavation walls can occur if the trenches are left open for extended periods.

4. The contractor should carefully read and understand city and project trench backfill specifications in regard to fill materials and compaction requirements.

5. Glory hole excavations and V-trenching should not be performed. They result in large quantities of heavy backfill that can cause long-term differential settlements.

HEAVY EQUIPMENT LIMITATIONS

Due to the underlying soft Bay Mud and the relatively thin layer of compacted fill, construction equipment should be limited to medium to lightweight size to reduce the potential for subgrade damage, pipe breakage, or slope failures. The thin layer of fill and Bay Mud crust is subject to high deflections under heavy wheel loads. The following observations and guidelines are presented regarding construction equipment:

1. Avoid the use of large earthwork equipment for mass grading operations. The following are suggested maximum equipment sizes over areas where granular materials are present in the upper 2 feet:

   - Compactors: Cat 815 or equivalent
   - Scrapers: Cat 613 or equivalent
   - Track-wheeled loaders: Cat 963 or equivalent
   - Blade: Cat 12G

2. In areas where the thickness of the existing fill has been reduced, even lighter equipment should be considered. Open excavations in soft Bay Mud cannot support rubber-tired equipment although light dozers with mud tracks, such as a Cat D4 or equivalent, are sometimes used.

3. Haul routes for trucks and scrapers should be kept level and smooth to prevent equipment from bouncing and imposing very high dynamic loads on the fill.

4. Heavy equipment should not be allowed to travel at high speeds as this can cause serious subgrade damage and rutting. This has been a particular problem for loaded scrapers.

5. All operators of heavy earthwork equipment should be informed of these guidelines and be aware of the general soil conditions.

PUMPING SUBGRADE

Under repeated wheel loads and/or excessive moisture, the existing fill can become rutted and difficult to repair. Soft or "pumping" areas can develop from heavy earthwork equipment. Careful attention should be given to the construction operations to limit traffic over areas that have become wet or show signs of surface cracking due to pumping subgrade soils. We present the following observations and guidelines regarding pumping subgrade soils:

1. Subgrade damage most often occurs where repeated heavy wheel loads are imposed on the soil.

2. Increased subgrade moisture content resulting from ponded water can also lead to subgrade damage under even lightweight earthwork equipment.

3. No water should be allowed to pond in traveled areas.

4. Concrete trucks should not be allowed to wash out in traveled areas.

5. Repair of damaged subgrade is most often accomplished by over-excavation to depths ranging from 8 to 18 inches, placement of geotextile fabric or geogrids, and careful compaction of select fill or aggregate base up to the former subgrade level.

WET WEATHER PROBLEMS

Earthwork and construction operations can be severely hampered by wet weather. We present the following observations and guidelines:

1. Construction through the wet weather months usually results in some required repair of damaged roadway subgrade and building pad fills.

2. Surface drainage of rainfall is generally limited by low grades and limited discharge points.

3. Wherever possible, the ground surface should be sloped to drain rainfall and prevent ponding of water.

4. Rainfall and ponded water tend to infiltrate and accumulate at the base of the granular fill.

5. If cuts are made for roadways, the thickness of select fill is often reduced such that shallow water can accumulate very near the roadway subgrade.

6. Forklifts, concrete trucks, and backhoes often cause severe rutting of surficial soils when used over rain-soaked soils.

7. Repeated vehicle and equipment traffic over wet areas can cause a thick mud slurry to accumulate during wet weather months.

8. Temporary construction haul routes constructed of crushed rock with or without an underlying geotextile membrane are often helpful in reducing subgrade damage and preventing damage to shallow utility pipes.

9. Gradall-type forklifts carrying construction materials can easily damage wet subgrade soils.

SHALLOW UTILITIES

Underground utilities can be subject to damage from heavy equipment. For gravity flow utilities such as sewer and storm drain, the pipes usually become shallower toward the rear of the sites, furthest from the main connections on roadways. Where pipes have relatively thin soil cover, they can be damaged from heavy earthwork equipment under certain conditions. When backfill has been properly placed and compacted over shallow pipes, the risk of damage from normal equipment wheel loads is usually small. If the fill is allowed to become saturated from ponded water, however, such as rainfall or wash out from concrete trucks, significant rutting can occur. We present the following observations and guidelines:

1. Shallow pipes should be adequately backfilled to the subgrade level in roadway areas and should not be left low.

2. Heavy equipment such as loaded trucks, forklifts, or concrete trucks should not be allowed to drive through areas where the subgrade soils have become wet. Damage to pipes by forklifts and concrete trucks may not be discovered until months later.

3. Where heavy equipment cannot be avoided, sewer laterals should be clearly marked to prevent damage.

SOIL AND AGGREGATE STOCKPILES

Stockpiling soil and crushed concrete and asphalt can cause non-uniform compression or bearing failure of the underlying Bay Mud. Stockpiles should be 5 feet or less in height. In addition, stockpiles should not be left in place for long periods (weeks) at time. The

Geotechnical Engineer should review and approve the proposed location and lateral extent of soil stockpiles greater than 5 feet high prior to construction.