CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation and Geologic Hazards Evaluation
PROJECT NAME	Charter Square K-5 School
LOCATION	1050 to 1088 Shell Boulevard Foster City, California
CLIENT	Westlake Urban, LLC
PROJECT NUMBER	826-2-1
DATE	August 1, 2017

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SECTION 1: INTRODUCTION

This geotechnical investigation and geologic hazards evaluation report was prepared for the sole use of Westlake Urban, LLC for the Charter Square K-5 School project located at 1050 to 1088 Shell Boulevard in Foster City, California. The location of the site is shown on the Vicinity Map, Figure 1. The site is located at Latitude 37.54976°N and Longitude -122.26469°W.

For our use, we were provided the following documents:

- A topographic survey titled “1050 – 1064 Shell Boulevard, Foster City, California,” prepared by Lea & Braze Engineering, Inc., dated November 29, 2006.
- A partial set of civil plans including a cut fill map, grading plans, and a stormwater control plan titled “Charter Square School,” Sheets C5.0, C5.1, C5.2, and C6.0, prepared by BKF, dated July 6 and 7, 2017.

1.1 PROJECT DESCRIPTION

The project will consist of constructing a new K-5 school for the SMFC School District. The site is currently occupied by a complex of one-story and two-story buildings and surrounding parking lots, plaza areas, and landscaping. Based on our discussions with you and review of the civil plans provided, the new school will consist of three one-story buildings including a classroom building, a classroom/administration building, and a multipurpose building. The buildings will generally be in the central portion of the site extending north to south with the multipurpose building on the north end of the site, the classroom building extending roughly two-thirds the length of the site north to south, and the classroom/administration building toward the southern end of the site. The multipurpose building and the classroom/administration building will be located in areas of the existing parking lots. The northern approximate fourth of the classroom building will also be located within the existing parking lot while the southern three-quarters of the building will extend across areas currently occupied by three of the existing buildings.

The classroom/administration building will total about 8,351 square feet, the classroom building about 28,460 square feet, and the multipurpose building about 5,775 square feet. The finished floor elevation of the multipurpose building will be Elevation 103.75 feet. The finished floor elevation of the classroom/administration building will be Elevation 105.17 feet in the approximate eastern third and Elevation 105 feet in the western two-thirds. To follow more closely to existing site grades and limit cuts and fills, the classroom building will also have two finished floor elevations with the northern third at Elevation 104.25 feet and the southern two-thirds at Elevation 105.75 feet. We understand the proposed buildings will be of wood frame construction.

An open field, pavement basketball courts and play areas, and playgrounds will be located within the western and northwestern portion of the site. School loading and dropoff areas, school entrances, and parking will be located on the eastern and southern portion of the site. Utilities, landscaping, and other pertinent improvements necessary for the school development are also planned.

Preliminary exterior and interior wall dead and live loads were provided by Crosby Group, Inc. For the multipurpose building, exterior wall dead and live loads are 640 pounds per lineal foot (plf) and 550 plf, respectively, and interior wall dead and live loads are 900 plf and 830 plf, respectively. For the classroom and classroom/administration building, exterior wall dead and live loads are 470 plf and 300 plf, respectively, and interior wall dead and live loads are 630 plf and 600 plf, respectively.

Based on the cut and fill map provided, cut and fills within the building pads are to range from about 1½ feet of cut to about ½ foot of fill. We understand that the plans will be reworked to limit the fills to ¼ foot. Cut and fills for areas surrounding the proposed buildings and throughout the rest of the site are to range from about 2 feet of cut to about 1 foot of fill.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated April 15, 2016 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

Our field exploration consisted of performing a geologic site reconnaissance, drilling eight exploratory borings on May 26 and 27, 2016 with truck-mounted, hollow-stem auger drilling equipment, and advancing nine Cone Penetration Tests (CPTs) with track- and truck-mounted CPT equipment on May 23 and 25 and June 8, 2016. The borings were drilled to depths of 20 to 55 feet; the CPTs were advanced to depths of approximately 93 to 128 feet. Practical refusal was encountered at CPT-4, CPT-5, CPT-7, and CPT-8. Seismic shear wave velocity measurements were collected from CPT-8. Boring EB-8 was advanced adjacent to CPT-8 for direct evaluation of physical samples to correlated soil behavior.

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

The approximate locations of our exploratory borings, CPTs, and building layouts are shown on the Site Plan and Geologic Map, 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, earthwork recommendations, and seismic ground deformation estimates. Testing included moisture contents, dry densities, a Plasticity Index test, triaxial compression tests, and consolidation tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The San Francisco peninsula is a relatively narrow band of rock at the north end of the Santa Cruz Mountains separating the Pacific Ocean from San Francisco Bay. This represents one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70- to 200-million years old) rocks of the Franciscan Complex. Locally these basement rocks are capped by younger sedimentary and volcanic rocks. Most of the Coast Ranges are covered by still younger surficial deposits that reflect geologic conditions of the last million years or so.

Movement on the many splays within the San Andreas Fault system has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west. The San Andreas Fault system and its major branch faults are about 40 miles wide in the Bay area and extends from the San Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the Regional Fault Map, Figure 3. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

2.2 REGIONAL SEISMICITY

The significant earthquakes that occur in the Bay area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system (see Figure 3), which regionally trend in a northwesterly direction. The San Andreas Fault, which generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989, passes about 5.1 miles southwest of the proposed campus. Three other major active faults in the area are the San Gregorio Fault, located about 12.8 miles southwest of the campus, the Hayward Fault, located about 13.2 miles northeast, and the Calaveras fault, located about 20.6 miles northeast. Another potentially active fault in the site vicinity is the Monte Vista – Shannon Fault, located about 7.8 miles to the southeast.

Table 1 lists all known active faults in order of increasing distance within 100 kilometers (62 miles) of the site. The fault distances presented in Table 1 are based in the 2008 USGS fault model (Petersen, et al., 2008) and were determined from the computer program EZ- Frisk (Risk Engineering, 2015). The tabulated distances represent the closest distance to the seismogenic source and may differ from the surface expression of the fault that is shown on published geological maps and on-line resources such as Google Earth. The seismic characteristics of some faults vary along its length so different segments of the same fault could be listed separately in the table.

Table 1: Approximate Fault Distances within 100-Kilometers

Fault Name	Distance (miles)	Distance (kilometers)
Northern San Andreas	5.1	8.2
Monte Vista-Shannon	7.8	12.5
San Gregorio Connected	12.8	20.6
Hayward-Rodgers Creek	13.2	21.3
Calaveras	20.6	33.2
Mount Diablo Thrust	25.7	41.3
Green Valley Connected	28.6	46.0
Greenville Connected	32.2	51.8
Zayante-Vergeles	35.6	57.3
Great Valley 7	37.8	60.8
Great Valley 5	40.0	64.4
Point Reyes	41.3	66.4

Table 1 Continues

Table 1: Approximate Fault Distances within 100-Kilometers (Continued)

Fault Name	Distance (miles)	Distance (kilometers)
West Napa	42.4	68.3
Monterey Bay-Tularcitos	44.1	71.0
Great Valley 4b	49.2	79.1
Great Valley 8	56.3	90.6
Ortigalita	57.3	92.2

2.3 HISTORICAL EARTHQUAKES

Figure 5 shows the epicenters of historical earthquakes within approximately 100 kilometers of the site. We also performed a catalogue search of known historical earthquakes of magnitude 5 or greater within approximately 100-kilometer radius of the site from 1906 to present using the USGS computer program located at http://earthquake.usgs.gov/earthquakes/search/. The results generated from that search are listed in Table 2.

Table 2: Magnitude 5 or Larger Earthquakes within 100 km

Date	Magnitude
4/18/1906	7.7
10/22/1926	6.3
9/5/1955	5.8
3/22/1957	5.7
11/28/1974	5.2
8/6/1979	5.8
1/24/1980	5.8
1/27/1980	5.4
4/24/1984	6.2
3/31/1986	5.7
6/13/1988	5.3
6/27/1988	5.3
8/8/1989	5.4
10/18/1989	6.9
10/18/1989	5.1

Table 2 Continues

Table 2: Magnitude 5 or Larger Earthquakes within 100 km (Continued)

Date	Magnitude
4/18/1990	5.4
9/3/2000	5.0
5/14/2002	5.0
10/31/2007	5.5
8/24/2014	6.0

2.4 FUTURE EARTHQUAKE PROBABILITIES

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults.

During such an earthquake, the danger of fault surface rupture at the site is slight, but very strong ground shaking would occur. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes centered in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

SECTION 3: SITE CONDITIONS

3.1 GEOMORPHOLOGY AND RECENT HISTORY

The site and surrounding area within Foster City was historically part of an extensive estuarine and marshland environment along the margins of the San Francisco Bay. Foster City was developed as a result of filling over a former marsh along the margins. A map showing the historic shoreline of San Mateo County indicates a slough trended through the western two-thirds of the site (Nichols and Wright, 1971; Pampeyan, 1981).

Aerial photographs listed in the References show the site vicinity at different times spanning the period from 1946 to 2012. Historic topographic quadrangle maps covering the area were also reviewed that include the years of 1905, 1949, 1957, 1961, 1975, 1981, and 1999. The 1905 topographic map shows the western ¾ of the site occupied by a slough named at that time

"Angel Creek". The 1946 aerial photos and the 1949 topographic map provides greater and shows that the active creek channel is located within the eastern portion of the sough. We infer that the slough represents the meandering path of the creek channel throughout the Holocene and Pleistocene epochs. By the time of the 1956 aerial photos it is apparent the former slough has been drained and the land on the east edge is being cultivated for row crops. By 1968 the land to the east of Shell Boulevard is being developed for commercial purposes. The site at this time (1968) is still undeveloped. Based on review of the topographic maps and the aerial photos, it appears the site was developed sometime in the mid to late 1970's. By 1980 the site was developed with the main building cluster dominating the central region and a parking lot on the north. The buildings in the southeast portion and the northeast portion did not appear until sometime just prior to 1987.

3.2 SURFACE DESCRIPTION

The site is currently occupied by several one-story, wood-framed buildings and a single two-story, wood-framed building and surrounding parking lots, drive aisles, concrete sidewalks, and landscaping areas consisting of plants, shrubs, and mature trees. The site is bounded by residential development to the west, an open space/park area to the north, Shell Boulevard to the east, and Beach Park Boulevard to the south. The site is generally flat with grades ranging from about Elevation 103 to 106 feet (NGVD 29 + 100) according to the topographic survey provided.

Based on our observations of the exterior and interior of some of the existing buildings, minimal distress was noted with only a few areas of cosmetic cracks. We understand that the existing buildings have been constructed on shallow foundations.

Surface pavements generally consisted of 1½ to 3 inches of asphalt concrete over 0 to 6 inches of aggregate base. Based on visual observations, the existing pavements are generally in moderate to poor condition with areas of severe cracking.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The published regional geologic map of Pampeyan (1994) is depicted in Figure 4, Vicinity Geologic Map. Roughly half the San Mateo Quadrangle is covered by Quaternary alluvial sediment shed from the northwest-trending Santa Cruz Mountains that occupy the western portion of the San Mateo quadrangle (Pampeyan, 1994). The site is in an area adjacent to the San Francisco Bay where Holocene age (11,000 years or less before present) alluvial fan, fluvial and estuarine deposits account for the majority of Quaternary sediment deposited in the eastern portion of the San Mateo Quadrangle. Pampeyan's map of 1994 indicates the site is in an area of widespread artificial fill (Qf) that resulted from the previous infilling of an extensive tidal marsh. Artificial fill in the map area consists of various natural and man-made materials emplaced by a variety of methods. The artificial fill is characterized as "poorly consolidated to well-consolidated gravel, sand, silt and fragments in various combinations used in a variety of applications." Pampeyan reports that in the late 1960's and early 1970's hydraulically-placed fill dredged from adjacent marsh lands and sloughs was placed on Brewer Island and vicinity (the site of present Foster City). The mapping by Pampeyan suggests the fill at the site may be underlain by Bay Mud (Qm). The Bay Mud (Qm) is described by Pampeyan as “Very poorly consolidated to well-consolidated, bluish-gray to black, organic clay and silt, with lenses of sand and shells and layers of peat. Bedding ranges from distinct to indistinct. Deposited in brackish to saline water along margin of San Francisco Bay.” The Bay Mud unit typically interfingers with fine- and medium-grained alluvium (Oaf) and is known to be at least 66 feet thick and may be as thick as 280 ft along the bay margin (Hensolt and Brabb, 1990). The Bay Mud in turn is underlain by the Qaf unit [medium grained alluvium (Holocene)]. The Qaf unit is described as “unconsolidated to moderately consolidated, moderately sorted sand and silty to clayey sand chiefly forming alluvial planes in and close to upland areas.” The above-mentioned portion of Pampeyan’s 1994 map was used as the base for our Vicinity Geologic Map, Figure 4. See also the subsurface description below**.**

The site has been completely developed and the geologic units are not exposed anywhere near the site. Our field exploration included the drilling, sampling, and logging of eight exploratory borings and the advancing of nine Cone Penetrometer Tests. Our exploratory borings were drilled to depths of 20 feet (EB-1 through EB-7) to 55 feet (EB-8). The borings encountered a surficial layer of manmade fill generally consisting of medium dense silty sand to depths of 2½ to 5½ feet. Below the fill, Borings EB-6, EB-7, and EB-8 encountered about ½ to 2 feet of residual soil consisting of loose to medium dense silty sand. Beneath the fill in Borings EB-1 to EB-5 and the residual soil in Borings EB-6 to EB-8, Bay Mud (highly compressible clay) was encountered to the maximum depth explored of 20 feet in Borings EB-1 to EB-7 and to a depth of approximately 42 feet in our deeper Boring EB-8. The upper part of the Bay Mud is a 1½- to 3½-foot thick of slightly over-consolidated clay commonly referred to as “Bay Mud Crust”. Below the depth of 42 feet, Boring EB-8 encountered generally lean clays likely belonging to the Qaf geologic mapping unit to the maximum boring depth of 55 feet. A thin, less than 1 foot thick layer of silty sand was encountered at 54 feet. The Bay Mud was found to generally be very soft to soft whereas the older clays below the Bay Mud was found to be generally stiff to hard.

The CPT explorations were advanced deeper than our borings ranging in depth from about 93 feet (CPT-4) to 128 feet (CPT-6). The CPTs encountered similar conditions as the borings to a depth of 55 feet and then generally encountered alternating layers of silts and clays to the maximum depth explored with a few layers of interbedded sands. This material is thought to also belong to the Qaf mapping unit.

Our explorations did not encounter stratigraphic changes that would help to delineate the former tidal slough channel. The very low flow velocities (largely driven by tidal effects) within these slough channels most probably result in the settling out of silts and clays through the water column rather than sands which would require relatively higher flow velocities.

3.3.1 Existing Fill

As previously discussed, the entire area is generally overlain by fills placed historically in the area to develop the prior bay margin site. Our borings encountered approximately 2½ to 5½ feet of undocumented fill blanketing the site. In general, the fills consisted of medium dense silty sand.

3.3.2 Bay Mud

The existing fill is underlain by estuarine deposits consisting of very soft to very stiff fat clay, known locally as Bay Mud. The upper 1½ to 3½ feet of the Bay Mud, typically referred to as Bay Mud "crust" is generally medium stiff to very stiff, over-consolidated due to historic wetting and drying cycles, and is generally considered only moderately compressible under light building loads. The Bay Mud crust is underlain by approximately 33 to 37 feet of soft, highly compressible clay to depths ranging from 40 to 43½ feet.

Moisture contents of the Bay Mud typically range from about 39 to 54 percent for the crust, and from 85 to 118 percent for the soft clay beneath the crust.

3.3.3 Older Alluvial Soils

Beneath the Bay Mud our borings and CPTs encountered alluvial deposits generally consisting of medium stiff to very stiff clays and silts with a few interbedded layers of medium dense to very dense sands to a depth of 128 feet, the maximum depth explored. These older alluvial soils, often referred to as Older Bay Clays, are generally over-consolidated and considered to have relatively low potential for compression.

3.3.4 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample of the upper Bay Mud crust material. Our test resulted in a PI of 47, indicating a very high expansion potential to wetting and drying cycles. The fills and residual soils encountered above the Bay Mud are cohesionless silty sands with fines that in our opinion have a low expansion potential to wetting and drying cycles.

3.4 GROUND WATER

Free groundwater was encountered in two of our Borings EB-6 and EB-8 at depths of about 3 and 11 feet below the surface, respectively. Ground water is generally considered to be at or near the top of the Bay Mud, which we encountered at depths ranging from about 3½ to 6 feet below existing grade. Seasonally, the fill located above Bay Mud can become saturated due to perched water from surface infiltration. We used variable design ground water depths of 3¼ to 5 feet below the existing ground surface for our liquefaction analysis. The depth of ground water selected for our liquefaction analysis was one foot above the Bay Mud at each corresponding exploration location. 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.

3.5 CORROSION SCREENING

We tested three samples (one sample within the overlying existing fill, one within the Bay Mud, and one within the older alluvial soil) from our borings for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 3.

Table 3: Summary of Corrosion Test Results

Boring/Sample	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-1/4A	9	8.0	68	14,115	493
EB-3/1A	1½	8.1	5,658	32	39
EB-8/12A	44	8.1	207	2,228	86

Notes: [1] ASTM G51
[2] ASTM G57 - 100% saturation
[3] ASTM D3427/Cal 422 Modified
[4] ASTM D3427/Cal 417 Modified
[5] 1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

Based on the laboratory test results summarized in Table 3, the overlying existing fill is considered moderately corrosive while the Bay Mud and older alluvial soil are considered very severely corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2016 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-14 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, no cement type restriction is required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. The table below summarizes the associated design values and parameters. We would like to note that although the samples tested do not indicated high sulfate exposure and corrosivity to buried concrete, it is our experience and opinion that Bay Mud is generally corrosive to buried concrete structures.

Table 4: ACI Sulfate Soil Corrosion Design Values and Parameters

Category	Water-Soluble Sulfate (SO4) in Soil (% by weight)	Sulfate (S) Class	Exposure Class	Cementitious Materials (2)
S, Sulfate	< 0.10	S0	F0	no type restriction

Notes: (1) above values and parameters are from on ACI 318-14, Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1
(2) cementitious materials are in accordance with ASTM C150, ASTM C595, and ASTM C1157
(3) refer to Table 5 for chloride design values and parameters

In addition to the above, severe chloride levels were determined in the samples below the existing fill. The table below summarizes the associated design values and parameters.

Table 5: Chloride Soil Corrosion Design Values and Parameters (1)

Category	Minimum f'_c (psi)	Class	Maximum *w/cm*	Maximum water-soluble chloride ion (Cl^-) content in concrete, percent by weight of cement	
				Nonprestressed Concrete	Prestressed Concrete
C, Corrosion protection for reinforcement	5,000	C2	0.4	0.15	0.06

Notes: (1) above values and parameters are from on ACI 318-14, Table 19.3.1.1 and Table 19.3.2.1

We have summarized design values and parameters from ACI 318-14, Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.11 in Tables 4 and 5 above for your information. We understand that JDH Corrosion Consultants, Inc. has been retained to provide corrosion recommendations for this project. The information we have provided above is simply for your reference. JDH Corrosion Consultants, Inc. corrosion recommendations should be followed for this project.

SECTION 4: GEOLOGIC HAZARDS

This section presents our geologic hazards review, as per the requirements of the Division of State Architects (DSA), the Office of Regulatory Services (ORS), and the CGS, formerly CDMG, for the Charter Square K-5 School project, located in Foster City, California, at Latitude 37.54976°N and Longitude -122.26469°W. Our comments concerning potential hazards are presented below.

4.1 FAULT RUPTURE

A regional fault map showing known active faults in the region surrounding the school site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone, known formerly as a Special Studies Zone and no surface expression of active faulting was seen on aerial photographs or in the field. Published regional scale mapping does not depict any active or probably active faults or zones near the site (Wentworth et al.,1985; Pampeyan 1994; Brabb et al., 2000; Bryant 2000). An unnamed, inferred (unclassified Quaternary) fault is shown on a few published maps (USGS Fault and Fold Database, USGS Interactive Fault Map) coming within 4,300 feet east of the site. This fault is designated as being less than 1.6 million years old and having a slip rate of less than 0.2 mm/year. The closest active fault to the site is the San Andreas that passes approximately 5.1 miles southwest of the site.

The Cañada/Hermit Fault Zone is also designated as Holocene active on the Alquist-Priolo map (CDMG, 1974b). This fault is mapped 3.5 miles west of the site, however, it has not been encountered in numerous fault trench investigations and will be mostly removed from the Town of Woodside Geologic Map (Wright, 2004). The Cañada/Hermit Fault Zone is not shown on the Active Fault Near-Source Zones maps of the California Geological Survey (CDMG, 1998), nor is it shown on the USGS Quaternary Fault/Fold database.

The Belmont Hill Fault is shown by Pampeyan (1994) concealed by late Pleistocene and Holocene sediments trending northwestward about 1.9 miles southwest of the site (Figure 4). Brabb and Olsen (1986) do not show the fault south of San Carlos and indicate no epicenters have been associated with it. Where the Belmont Hill Fault is exposed in Franciscan rocks in Belmont and San Carlos, it is a vertical to steeply west-dipping reverse fault (Pampeyan, 1993).

4.2 HISTORICAL GROUND FAILURES

Many historical earthquakes have occurred on active faults and fault branches throughout coastal California, but the San Andreas Fault is considered one of the major active faults of the region. It generated significant, damaging earthquakes in 1836 and 1868, as well as the great San Francisco Earthquake of 1906, which had an approximate Richter Magnitude of 8.3, and the Loma Prieta Earthquake of 1989.

Lawson (1908, p. 246) also described damage in San Carlos from the 1906 earthquake: “The railway station at San Carlos, a low 1-story stone building, was badly damaged, some of the walls being partly thrown down, and the rest of the building cracked. A large frame house near the station was shaken from its cement foundations, and the foundation itself was badly cracked... Between San Carlos and Belmont, over four-fifths of the houses lost their chimneys, but no buildings were thrown from their foundations.” His mapping shows the Foster City area area shook at Rossi-Forel intensity VIII (many chimneys fall) to IX (partial or complete collapse of some buildings). Nason (1980) believed the San Carlos area shook at Modified Mercalli Intensity 8 (characterized on the scale as “general fright and alarm approaches panic”) during the 1906 earthquake. There have been no documented historic cases of ground rupture in the immediate area of the campus as a result of seismic shaking in the 1906 (Youd and Hoose, 1978; Knudsen and others, 2000) or 1989 Loma Prieta Earthquake (Loney 1995) and the Modified Mercali scale intensity was estimated and 4 to 6.

4.3 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) of 0.573g was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2016 edition of the California Building Code. Seismic design criteria values are presented in Section 8.2 of this report.

4.4 LIQUEFACTION POTENTIAL

The site is not currently mapped by the State of California but is mapped by the Association of Bay Area Governments as having a high hazard potential for liquefaction due to an earthquake. Our analysis addressed this issue by retrieving samples from the site, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the soil properties.

4.4.1 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.

4.4.2 Analysis and Results

As discussed in the “Subsurface” section above, some sand layers were encountered below our design ground water (varying by exploration location) corresponding to 1 foot above the top of Bay Mud, corresponding to ground water depths ranging from 3¼ to 5 feet below the existing ground surface. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), 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 (i.e. settlement).

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 less reliable 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.

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49 high-quality, cyclically induced, ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data. Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sublayers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to nonliquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

The results of our CPT analyses (CPT-1 through CPT-9) are presented on Figures 8A to 8I of this report.

4.4.3 Summary

Our analyses indicate that some layers could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement ranging up to ½ inch based on the Yoshimine (2006) method. As discussed in Special Publication 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 are anticipated to be on the order of ⅓ inch between independent foundation elements, assumed to occur over a horizontal distance of 30 feet.

4.4.4 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 approximate 3⅔- to 5-foot thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore, the above total settlement estimates seem reasonable.

4.5 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.

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.6 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose to medium dense unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the granular soils above the design ground water level based on the CPT and SPT data using the Robertson and Shao (2010) and the Pradell (1998) procedure. Based on our analysis, the unsaturated granular soils are anticipated to experience less than ¼-inch of total settlement following strong seismic shaking.

4.7 LANDSLIDING

The site and adjacent areas are topographically flat and are located far from any slopes. The regional scale published landslide themed maps show no landslides of debris flow source areas anywhere near the site (Brabb and Pampeyan, 1972; Mark 1972; San Mateo County 2008) nor does it have a potential for earthquake induced slope instability (Wieczorek et al., 1985). Therefore, the potential for landsliding to impact the site is virtually nil.

4.8 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 1 mile inland from the San Francisco Bay shoreline, and is approximately 3 to 6 feet above mean sea level. Although the site is relatively close to the shoreline, the site is above sea level, and is mapped by the State of California Tsunami Inundation Map as not being within an inundation area. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.9 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X (areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood). There is a note indicating that overtopping or failure of any levee system is possible. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

4.10 VOLCANIC ERUPTION

The site is located over 200 miles hundred miles from the nearest potentially or historically active volcano (at Mt. Lassen Park). We believe the volcanic eruption hazard for the school site is very low.

SECTION 5: BAY MUD SETTLEMENT

The site is underlain by up to approximately 33 to 37 feet of highly compressible Bay Mud. Analyses were performed to estimate the potential settlement associated with the placement of new fill and foundation loads. The analysis assumed a design period of 30 years commencing with the completion of construction.

Our settlement analysis was based on the site history, laboratory data, and proposed construction. Given the site history and laboratory data collected, we assumed the Bay Mud crust and Bay Mud was slightly over-consolidated to a depth of approximately 10 feet, and normally consolidated below a depth of 10 feet under the weight of the existing fill and prior to placement of new fills.

As discussed, based on the cut and fill map provided, cuts up to 2 feet and fills up to 6 inches are being proposed to grade the building pads and surrounding site areas. We assumed a total unit weight of 135 pounds per cubic foot for any new fill for our analysis. The results of our settlement analysis indicates that approximately 4 to 6 inches of total consolidation settlement per foot of fill placed will occur after fill placement. This settlement would occur gradually over a 30 year time period. Based on this, we advise to limit filling to 3 inches above existing grades to limit settlement due to fill placement.

5.1 AREAL FILL SETTLEMENT

Where areal fill is placed, the amount of future settlement is primarily dependent on the construction grade and the thickness of Bay Mud. For example, we estimate that continuing settlement due to the existing fills at the site will be limited, and that primary consolidation due to the original construction will likely be less than 1 inch. We estimate that settlement from the placement of new fills will be approximately 4 to 6 inches for every foot of new fill placed. The settlement discussed above is due to the weight of the existing and any additional fills only;

settlement from other loads would be in addition to these estimates. Currently, the plan is to limit new fill placement to 3 inches or less to limit any new settlement from placement of fills.

We also estimated the approximate rate of future settlement during the 30-year period after construction. Based on our analyses, we estimate that approximately 25 percent of the indicated future settlements will occur within about 5 years after construction and approximately 65 percent of estimated future settlements will occur within 30 years after construction.

In addition, we estimate that less than 2-inches of settlement due to secondary consolidation will occur after 30 years from the time of construction. Some of this settlement has occurred.

The estimated settlements should be taken into account in the design of surface drainage and gravity-flow utilities to minimize the potential for grade reversal and joint separation or leakage.

Any underground utility pipes entering the buildings should be designed to accommodate the expected differential settlement between the building and the adjacent ground.

5.2 FLEXIBLE UTILITY CONNECTIONS

Due to the expected total and differential settlements, incorporating the anticipated settlements in the design of new utilities and surface drainage will be necessary. Ball joints and sleeve type or other flexible couplings, as appropriate, should be considered between piping and the buildings and in utility areas in which large differential settlements are expected to occur.

SECTION 6: CONCLUSIONS

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Bay Mud settlement
- Liquefaction-induced settlements
- Shallow ground water
- Differential movement between exterior grades and structures
- Corrosion potential of soils
- Presence of undocumented fill
- Soft soil construction
- Cohesionless soils

6.1 BAY MUD SETTLEMENT

As discussed in Section 5.0, the site is underlain by 33 to 37 feet of highly compressible Bay Mud that will continue to settle under the weight of the existing fill, from any new fills placed, and from any building loads. Even small grade changes can cause additional settlement of the underlying soft clays (Bay Mud). 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.

To mitigate the effects of the anticipated differential settlements, we recommend that gravity flow utilities be designed to account for any future settlement to avoid grade reversal, sags or leakage from joint separation. In addition, even where grades are similar and anticipated settlement is expected to be relatively uniform, differential settlement can occur due to variations in conditions, previous stress history, and other anomalies. Therefore, utilities should be designed for variations in differential settlement, including over-steepening gravity-flow utilities to accommodate such variations.

Currently, we understand the proposed buildings will be one story with typical interior and exterior wall loading as depicted in Section 1.2 above. Based on these loads and our settlement analysis, to limit the total and differential static settlements, the one-story buildings may be supported by a grid of continuous shallow foundations baring the total and differential static settlements due to foundation loads discussed in the “Foundations” section are acceptable. It should also be noted that based on the settlement estimates discussed in Section 5.0, minimal (less than ¼ foot) additional fill should be placed to grade the building pad areas to enable the one-story buildings to be founded on gridded continuous shallow foundations. Based on the cut fill grading map provided, there appears to be areas within the building pads with fills up to about ½ foot. We recommend grades be adjusted to limit fills to ¼ foot or less or light weight fill be utilized. We understand the plans are being revised. Detailed recommendations for shallow foundations are presented in the “Foundations” section.

6.2 LIQUEFACTION-INDUCED SETTLEMENT

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of some 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 ranging up to ½ inch could occur, resulting in differential settlement up to ⅓ inch between independent foundation elements. Foundations should be designed to tolerate the anticipated total and differential settlements. Detailed foundation recommendations are presented in the “Foundations” section.

6.3 SHALLOW GROUND WATER

Free ground water was encountered in two of our borings at depths of approximately 3 and 11 feet below current grades. We anticipate ground water to be at or near the top of the Bay Mud (encountered at depths of about 3½ to 6 feet) and potentially higher up into the overlying existing fill material. Our experience with similar sites in the vicinity 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 DIFFERENTIAL MOVEMENT FROM EXTERIOR GRADES TO STRUCTURE

The amount of static settlement between exterior at-grade improvements and structures supported by shallow continuous strip footings will vary. 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 entrances/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.

- Replacement and maintenance of slabs-on-grades to repair these areas of movement every 5 to 10 years in the building operating costs and performed as needed.

6.5 CORROSION POTENTIAL OF SOILS

As discussed, we performed a preliminary soil corrosion screening based on the results of analytical tests on samples of the soils. In general, based on the test results, the use of sulfate resistant concrete is not required for buried concrete; however, we would like to note that it is our experience and opinion that Bay Mud is generally corrosive to buried concrete structures. Additionally, test results indicate the corrosion potential for buried metallic structures, such as metal pipes, is considered moderately corrosive in the overlying existing fill but very severely corrosive in the Bay Mud. Furthermore, severe chloride levels were determined in the samples below the existing fill. As mentioned, we understand JDH Corrosion Consultants, Inc. have been retained to provide corrosion recommendations for this project. Recommendations provided by JDH Corrosions Consultants, Inc. should be followed for this project.

6.6 PRESENCE OF UNDOCUMENTED FILL

Our borings encountered approximately 2½ to 5½ feet of undocumented fill. Because the existing fill was placed decades ago and is more stable than the Bay Mud, this material is not recompacted on projects in Foster City. To reduce the potential for differential settlement, we recommend the bottom of all shallow foundations be compacted with vibratory equipment before the placement of rebar. To reduce the potential for damage to proposed surface improvements due to differential settlement of the fill material, new surface improvements should be constructed on a properly-prepared subgrade. Detailed recommendations are provided in the "Earthworks" section.

6.7 SOFT SOIL CONSTRUCTION

Since the existing fill is relatively thin over the Bay Mud, grading and installation of utilities over these soft soils will likely require special earthwork considerations. In anticipation of these considerations, we are providing construction guidelines on Bay Mud in Appendix C.

6.8 COHESIONLESS SOILS

As mentioned, the site is blanketed by existing fills consisting of generally cohesionless, silty sands. Due to the cohesionless sandy soils, excavation sidewalls for shallow foundations, utility trenches, etc. may cave in or accumulate a significant amount of slough. The contractor will need to address this issue. We recommend that consideration be given to installing Stay-Form®, or similar, on all excavations including shallow footings and shallow trenches to reduce the potential for sidewall collapse. Deeper trenches will need shoring designed to accommodate sands and Bay Mud.

6.9 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.

6.10 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter widely-spaced 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 SUMMARY

In general, the site may be graded with relatively light-weight grading equipment; however, potential difficulties could arise due to shallow Bay Mud, loose to medium dense existing fill, and shallow ground water. 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 C, Construction Guidelines on Bay Mud.

7.2 SITE DEMOLITION, CLEARING AND PREPARATION

7.2.1 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. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

7.2.2 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.

7.2.3 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas. Slabs, foundations, 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.

7.2.4 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 or to the installation of pile foundations. 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.

7.3 EXISTING FILLS

Our borings encountered approximately 2½ to 5½ feet of undocumented fill. Based on our borings, the fill will generally consist of silty sand at the bottom of new at-grade shallow footing foundations. We recommend the bottom of all shallow foundations be compacted with vibratory equipment before the placement of rebar. A Cornerstone representative should observe the bottom of all foundations prior to the placement of rebar.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and are constructed on a properly-prepared subgrade as discussed in the “Compaction” section below.

7.4 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.

Excavations performed during site demolition and utility removal should be sloped at 3:1 (horizontal:vertical) within the upper about 4 feet below building subgrade. Excavations extending more than about 4 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, braced shoring, slide rail, 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. Use of trench boxes or shields and temporary trench shoring should not be used in Bay Mud or overlying sands.

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 during construction. To help reduce the potential for base heave, once a pipe has been placed in an excavation, the trench should be backfilled the same day. The shoring designer should analyze the potential for base heave.

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 material 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 utilities.

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 recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

Any below-grade excavation, could be located close to, or in, Bay Mud, depending on the excavation depth and the amount of fill in that area. Stabilization of the bottom of these excavations will likely be required. Stabilization should be accomplished with 12 to 18 inches of clean crushed rock depending on the final depth and condition of subgrade. The crushed rock should be underlain by stabilization fabric (Mirafi RS 380i or approved equivalent) as separation between the native clays and the crushed rock. The final thickness of crushed rock needed should be based on the judgment of the contractor and the type of equipment and material loading that is likely to occur. Construction equipment is unlikely to be able to access the bottom of excavations without stabilized access. Destabilized or disturbed areas will require repair using methods approved by the geotechnical engineer. Increased stability could be obtained with the use of fabric or geogrids beneath the stabilization section of crushed rock. Excavations should be in accordance with recommendations for excavations in Bay Mud.

7.6 SHALLOW GROUND WATER

Free ground water was encountered in two of our Borings EB-6 and EB-8 at depths of about 3 and 11 feet below the surface, respectively. We anticipate ground water to be shallow, at or near the top of the Bay Mud (encountered at depths ranging from about 3½ to 6 feet), and potentially higher up into the overlying existing fill material. Therefore, in our opinion, excavations deeper than approximately 3 feet may potentially be impacted by shallow ground water, depending on how long the excavations are left open. This depth might be higher or lower depending on the time of year. In our opinion, provided shallow excavations less than 3 feet deep are completed and backfilled within the same day, they will most likely remain relatively dry. If significant ground water does accumulate, it should be removed from the excavation prior to backfilling.

7.7 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 potentially 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.

7.7.1 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.

7.7.2 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. Please refer to Section 7.5 above for discussion on stabilization close to or in Bay Mud.

7.7.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with cement (for silty sands) 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. Chemical treatment will not be acceptable in planted areas. Removal of chemical treated materials before planting should be anticipated.

7.8 MATERIAL FOR FILL

7.8.1 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 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 oversized 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.

7.8.2 Re-Use of On-Site Site Improvements

Asphalt concrete (AC) grindings and aggregate base (AB) may 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. AC/AB grindings may not be reused within building footprint areas. Laboratory testing will be required to confirm the grindings meet project specifications.

7.8.3 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less. 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.

7.8.4 Controlled Low-Strength Material

Controlled Low-Strength Material (CLSM) may be used as engineered fill. As with all engineered fill, CLSM should be placed on subgrade soils prepared in accordance with “Subgrade Preparation” section above. CLSM should have a minimum 28-day unconfined compressive strength of 50 to 100 pounds per square inch (psi). Unconfined compression testing should be performed in accordance with ASTM D4832. CLSM should be placed and tested in accordance with DSA IR 18-1.

7.9 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 and 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. Where the soil’s PI is 20 or greater, the expansive soil criteria should be used.

Table 6: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	On-Site Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	93	>3
	On-Site Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	On-Site Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum

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)

Table 6 Continues

Table 6: Compaction Requirements (Continued)

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	On-Site Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	On-Site Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	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)

7.9.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

7.10 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.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

Bay Mud underground utility construction means and methods are generally left up to the contractor; however, excavations extending into Bay Mud will require shoring. Bay Mud is very weak, and may fail due to surcharge from equipment, or even under its own weight. Utilities extending into Bay Mud should balance the weight of backfill materials with the weight of materials being removed as recommended in this section. In addition, dewatering will likely be necessary in trenches due to perched water or ground water seepage as discussed in Section 7.11.2, “Construction Dewatering”. We should review the project plans to confirm the backfill will not impose a surcharge on the Bay Mud. Lightweight fill may be needed.

7.11 SITE DRAINAGE

7.11.1 Surface Drainage

Surface water should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 to 3 percent towards suitable discharge facilities; landscape areas should slope at least 3 to 5 percent towards suitable discharge facilities. It is noted that surface drainage should be designed for the estimated Bay Mud settlements. 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.

7.11.2 Construction Dewatering

Construction of underground utilities extending into the fill materials, or into the native Bay Mud, may require excavation dewatering due to perched water or ground water. Because of the relatively low permeability of the Bay Mud, seepage from the native Bay Mud may be minimal; however, existing fill areas could potentially be saturated and excavations extending through them could be subject to seepage. The dewatering system should be designed and implemented by the contractor.

Depending on the ground water quality, on-site retention, off-site disposal, or treatment prior to discharge, either into storm or sanitary sewer, may be required.

7.12 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 have variability in fines content and are expected to have infiltration rates of less than about 0.2 inches per hour. In our opinion, the soils could significantly limit the infiltration of stormwater.

- Ground water is generally considered to be shallow, likely at or near the top of the Bay Mud, which ranged from depths of about 3½ to 6 feet below the surface. Free ground water was encountered in two of our borings at depths of about 3 and 11 feet and seasonal fluctuations can cause the fill located above the Bay Mud to become saturated with perched water. Therefore, ground water is expected to be within 10 feet of the base of the infiltration measure.

- 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.

7.12.1 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.

7.12.1.1 General Bio-retention Basin Design Guidelines

- If possible, avoid placing bio-retention basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bio-retention basins must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with an impermeable membrane to reduce water infiltration into the surrounding soils.

- Bio-retention basins constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bio-retention basins 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 bio-retention basin filter material is above the foundation plane of influence.

- The bottom of bio-retention basin 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.

7.12.1.2 Bio-retention Basin Infiltration Material

- Gradation specifications for bio-retention basin filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bio-retention basin 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 bio-retention basin 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 bio-retention basin 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 bio-retention basins are to be planted.

- If bio-retention basins 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 bio-retention basin 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 bio-retention basin filter materials, long-term settlement of the bio-retention basin medium should be anticipated. To reduce initial volume loss, bio-retention basin 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 bio-retention basin materials.

- It should be noted that the volume of bio-retention basin filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bio-retention basins after the initial exposure to winter rains and periodically over the life of the bio-retention basin areas, as needed.

7.12.1.3 Bio-retention Basin 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 bio-retention basin 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 bio-retention basin such that there is at least 3 feet of horizontal distance between the edge of improvements and the top edge of the bio-retention basin excavation for every 1 foot of vertical bio-retention basin depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bio-retention basin 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.

- The underlying soft Bay Mud will provide little to no support and is subject to static settlement. As discussed, footings for restraining improvements adjacent to bio-retention basins should not extend below a depth of 30 inches below site grades. As a result, basins adjacent to improvements will likely need to be braced to provide lateral restraint.

7.13 LANDSCAPE CONSIDERATIONS

Landscaping fill berms, retaining structures, or other landscaping features that cause abrupt changes in stress on the underlying Bay Mud may adversely affect the development by contributing to differential settlement 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,
- 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, the proposed one-story buildings may be supported by a grid of continuous shallow foundations provided the total and differential static plus seismic settlements are determined acceptable, minimum (less than ¼ foot) additional fills are placed during grading or lightweight fill is used, and the recommendations in the “Earthwork” section and sections below are followed.

8.2 SEISMIC DESIGN CRITERIA

The 2016 California Building Code (CBC) 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.

The shear wave velocity measurement performed for our investigation at CPT-8 resulted in an average shear wave velocity of 405 feet per second (or 123 meters per second) and the site is underlain by 33 to 37 feet of deep soft clays with moisture contents greater than 40 percent and undrained shear strengths of less than 500 pounds per square foot (psf). The Bay Mud does not have a significant organic content based on our explorations in the Foster City area. Therefore, we have classified the site as Site Class E. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS web-based program *U.S. Seismic Design Maps*, located at http://earthquake.usgs.gov/designmaps/us/application.php, based on the site coordinates presented below and the site classification.

In accordance with 2016 CBC Sections 1613A.3.5 and 1616A.1.3, a site-specific seismic design analysis is required when Risk Category I, II, or III structures with a mapped spectral response acceleration parameter at the 1-second period (S_1) is greater than 0.75. In accordance with the above, since the mapped spectral response acceleration at the 1-second period (S_1) is 0.730, a site-specific seismic design analysis is not required and was not performed. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 7: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	E
Site Latitude	37.54976°
Site Longitude	-122.26469°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.589g
1-second Period Mapped Spectral Acceleration[1], S_1	0.730g
Short-Period Site Coefficient – Fa	0.9
Long-Period Site Coefficient – Fv	2.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.430g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.751g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	0.953g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.167g
Mapped MCE Geometric Mean Peak Ground Acceleration – PGA_M	0.573
Site Coefficient Based on PGA and Site Class - F_{PGA}	0.9

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

8.3 SHALLOW FOOTING FOUNDATIONS

Provided the estimated settlements are acceptable to the project structural engineer, the proposed one-story buildings may be founded on a grid of continuous strip footing foundations to aid in limiting total and future differential settlements.

It should be noted that due to the potential settlement discussed below, settlement cracks in the building interior/exterior finishes may occur over time. This type of settlement is normal for this type of project site and the cracks are cosmetic. Therefore, occasional repairs for cosmetic issues should be expected and planned for during the lifetime of the project.

8.3.1 Continuous Strip Footings

Continuous strip footings should bear on existing fill material or engineered fill and be at least 15 inches wide, and extend at least 15 inches below the lowest adjacent grade. Footing widths should not exceed 24 inches. If footing widths exceed 24 inches, we should be consulted to analyze potential settlements on a case-by case basis. Additionally, footing depths should be kept as shallow as possible (i.e. should not exceed 30 inches below lowest adjacent grade) to keep footing bottoms as high above the soft, compressible Bay Mud and limit static settlements. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. As previously discussed, we recommend the bottom of all footings should be compacted with vibratory equipment in accordance with our “Earthwork” section of this report prior to placement of rebar.

We have made estimates of the ultimate bearing capacity for continuous strip footings that are 15 to 18 inches wide and 15 to 30 inches deep. Additionally, we assume the over-strength factor will be on the order of 3 for this project. Our analysis indicates that the ultimate bearing capacity would be on the order of 2,400 pounds per square foot (psf). For the purpose of evaluating the footings for Dead Load plus Live Load plus transient Earthquake Load combinations, we would recommend an allowable bearing capacity of 800 psf. For the purposes of evaluating the footings for Dead Load plus Live Load combinations, we would also recommend an allowable bearing pressure of 800 psf. The allowable bearing pressures listed above have a factor of safety of 3 or greater which is consistent with the over-strength factor of three for the building type which was assumed. The above pressures are net values; the weight of the footing may be neglected for the portion of the footing 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.

8.3.2 Footing Settlement

As mentioned, preliminary exterior and interior dead and live loads were provided for the multipurpose building, classroom, and classroom/administration building. Based on these loads and the allowable bearing pressure presented above, we estimate that the total static footing settlement will be on the order of 1 inch, with about ½-inch of post-construction differential settlement between independent foundation elements. In addition, we estimate that differential seismic movement will be on the order of ⅓-inch between independent foundation elements, resulting in a total estimated differential footing movement of about ¾-inches between independent foundation elements, assumed to be on the order of 30 feet. We recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

As previously mentioned, we recommend minimal (less than ¼ foot) additional fill be added during grading as new fill may cause significant additional settlement. If fills within the building pads can not be limited to less than ¼ foot, light weight fill should be considered.

8.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.35 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 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. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.3.4 Spread Footing Construction Considerations

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.

Sloughing of the sandy soil is considered likely; therefore we recommend that Stay-Form® or similar be placed within the footing excavations as they are excavated during construction of the foundation elements. Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

8.3.5 Alternative Foundation

As an alternative to spread footings or if the estimated settlements exceed the structural requirements, the proposed buildings may be supported on a reinforced concrete mat or by deep foundations. If these foundation alternatives are desired, we should be contacted to provided additional recommendations. We understand that continuous shallow foundations will be used for this project.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE WITH CONTINUOUS STRIP FOOTINGS

As the Plasticity Index (PI) of the surficial soils is 15 or less, proposed slabs-on-grade may be supported directly on subgrade unless moisture protection is required (see below). Subgrade should be prepared in accordance with the recommendations in the “Earthwork” section of this report. 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.

Our experience with tideland reclamation projects has shown that ground water generally stabilizes, through evaporation, to the level of the top of the Bay Mud in uncovered areas. However, when the areas are covered by buildings or slabs-on-grade, evaporation becomes inhibited and ground water rises by capillary action to depths near the bottom of the floor slabs. This can result in damp or wet floors. If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the "Moisture Protection Considerations" section of this report should be considered.

9.2 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 minimum 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 crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 - 10

- 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 and F710 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.3 EXTERIOR PEDESTRIAN CONCRETE FLATWORK

Exterior concrete flatwork and sidewalks should be at least 4 inches thick and underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. We recommend that exterior slabs be isolated from adjacent improvements. In addition, consideration should be given to using closer than normal joint spacing to allow slabs to better conform to the expected settlement. We further recommend that the designer consider the high potential for differential movement in selecting appropriate reinforcing steel, dowels, and cold joints. Hinge connections should be used wherever differential settlements may be detrimental to exterior flatwork, such as at building connections or abrupt changes in grade or surface loads. If concrete flatwork will be subject to wheel loads, they should be designed in accordance with the "Portland Cement Concrete Pavements" section of this report.

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 20 to represent the silty sands at the project. The design R-value was chosen based on engineering judgment considering the variable surface conditions.

Table 10: Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	5.5	8.0
4.5	2.5	7.0	9.5
5.0	3.0	7.0	10.0
5.5	3.0	9.0	12.0
6.0	3.5	9.5	13.0
6.5	4.0	10.5	14.5

*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. We recommend that the civil engineer consider placement of perforated pipes behind or underneath curbs to intercept and control water that may seep into the pavement subgrade.

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). 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 11: PCC Pavement Recommendations

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. We recommend that the civil engineer consider placement of perforated pipes behind or underneath curbs to intercept and control water that may seep into the pavement subgrade.

SECTION 11: RETAINING WALLS

11.1 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, surcharge loads acting behind the wall, and the potential for differential movement due to surcharge of the underlying highly compressible Bay Mud. To limit differential settlement, we recommend site walls be limited to about 2 to 3 feet. Please note the differential loading caused by retaining walls can cause significant differential settlement. 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 12: 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

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 2016 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls greater than 6 feet in height. At this time, we are not aware of any new retaining walls for the project and have not provided seismic earth pressures with this report. Seismic earth pressures can be provided at a later time for walls greater than 6 feet in height, if requested by the project design team. Seismic earth pressures are not required to design minor landscape retaining walls.

11.3 WALL DRAINAGE

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.

11.4 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. If significant differential settlement will occur, lightweight backfill should be considered to limit settlement.

11.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the “Foundations” section of this report. As discussed, footings should not extend more than 30 inches below site grades.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Westlake Urban, LLC specifically to support the design of the Charter Square K-5 School project in Foster City, 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.

Westlake Urban, LLC may have provided Cornerstone with plans, reports and other documents prepared by others. Westlake Urban, 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.

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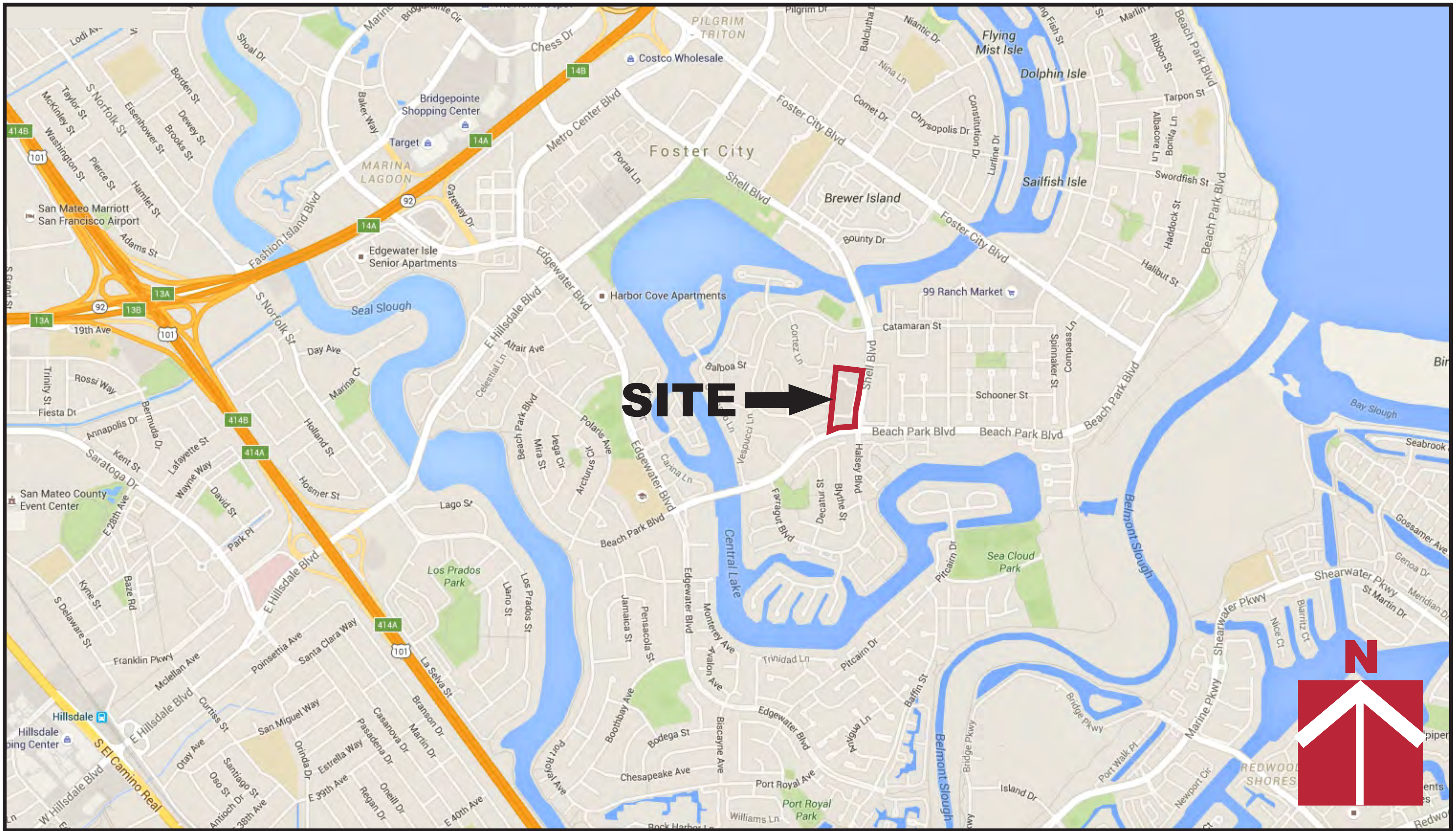
Aerial Photographs Reviewed:

Date	Type
1946	vertical black & white
1956	vertical black & white
1958	vertical black & white
1968	vertical black & white
1980	vertical black & white
1993	vertical black & white
2002	vertical black & white
2005	vertical black & white
2009	vertical black & white
2010	vertical black & white
2012	vertical black & white

Historic USGS Topographic Maps Reviewed:

Date	Type
1905	1:62,500
1949	1:62,500
1957	1:24,000
1961	1:24,000
1975	1:24,000



![Cornerstone Earth Group logo]

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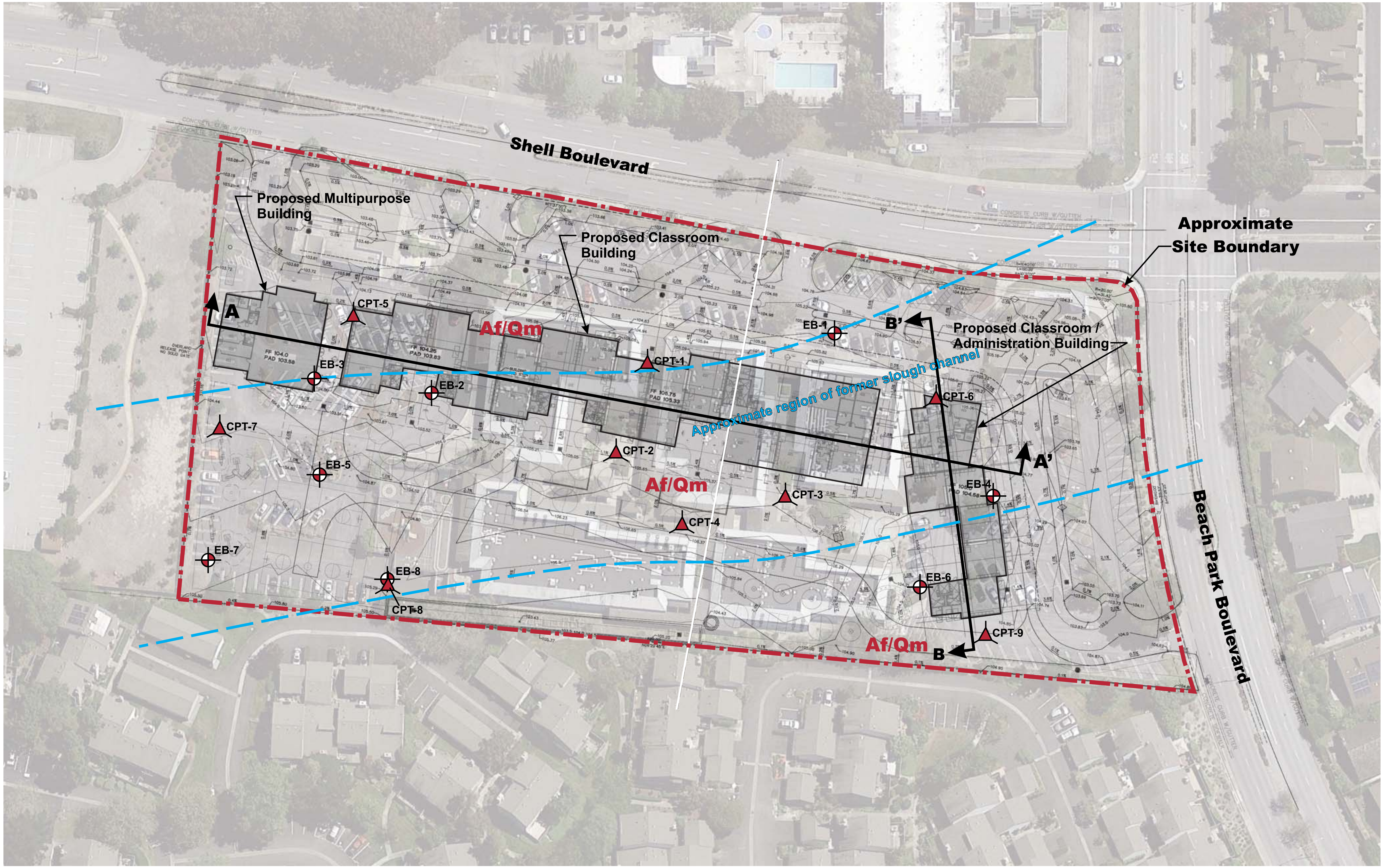
Vicinity Map

Charter Square K-5 School
Foster City, CA

Project Number	826-2-1
Figure Number	Figure 1
Date	June 2016
Drawn By	RRN



Geologic Units

Af	Artificial fill
Qm	Bay Mud (Holocene)

Legend

⊕ Approximate location of exploratory boring (EB)

▲ Approximate location of cone penetration test (CPT)

A A' Approximate location of cross section

— — Approximate limits of slough channel

N

0 80 160

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2016, over lay by BKF, "Grading Plan", dated 7/6/2017.

Project Number 826-2-1

Figure Number Figure 2

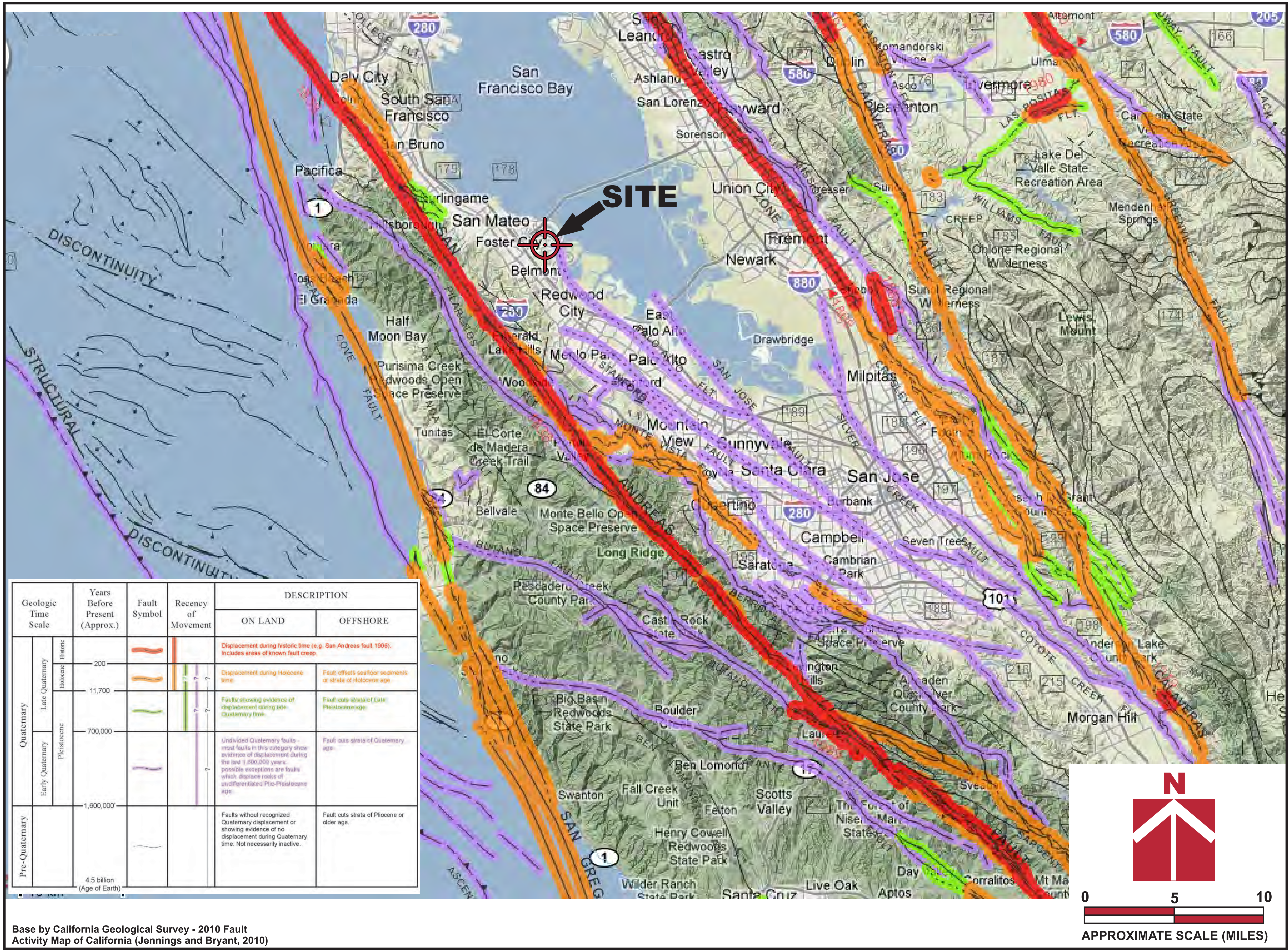
Date July 2017

Drawn By RRN, FLL

Site Plan and Geologic Map

Charter Square K-5 School
Foster City, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number
826-2-1

Figure Number
Figure 3

Date
June 2016

Drawn By
RRN

Regional Fault Map

Charter Square K-5 School
Foster City, CA

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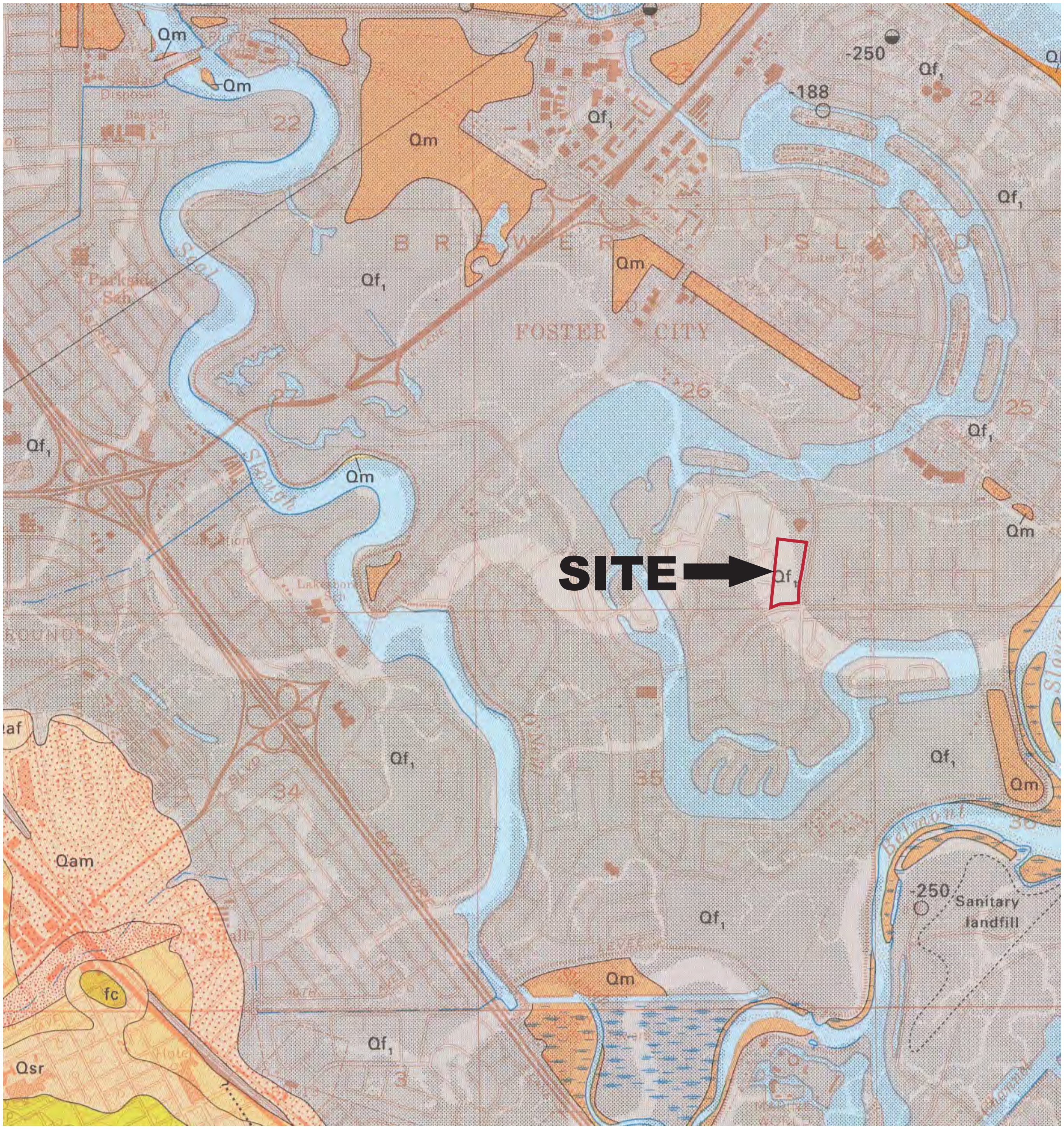

Geologic Units

Qam	Medium-grained alluvium (Holocene)
Qm	Bay mud (Holocene)
Qf$_1$	Artificial fill (Holocene)

Explanation

Contact- dashed where approximate, dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base: Geologic Map of the Montara Mountain and San Mateo 7.5' Quadrangles, San Mateo County, California, by Pampeyan, 1994

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Vicinity Geologic Map

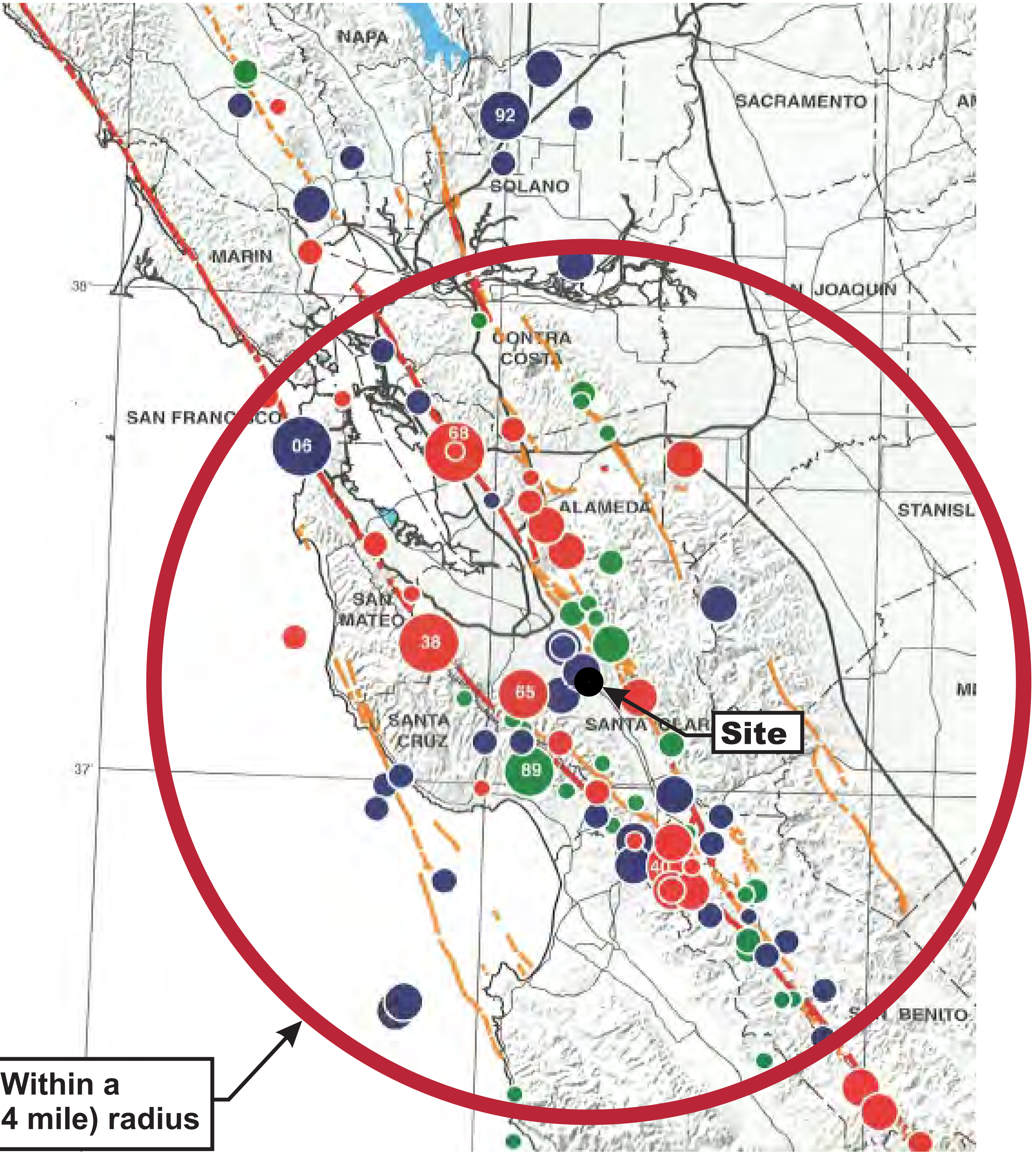
Charter Square K-5 School
Foster City, CA

Project Number 826-2-1

Figure Number Figure 4

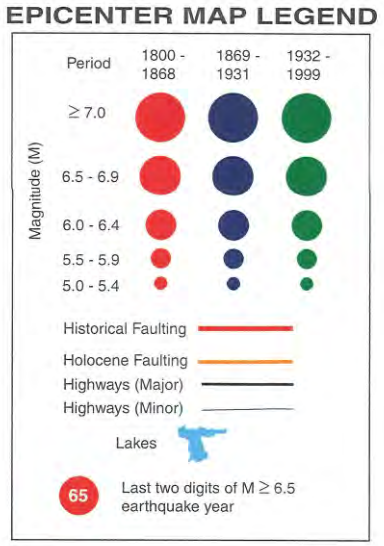
Date June 2016

Drawn By RRN

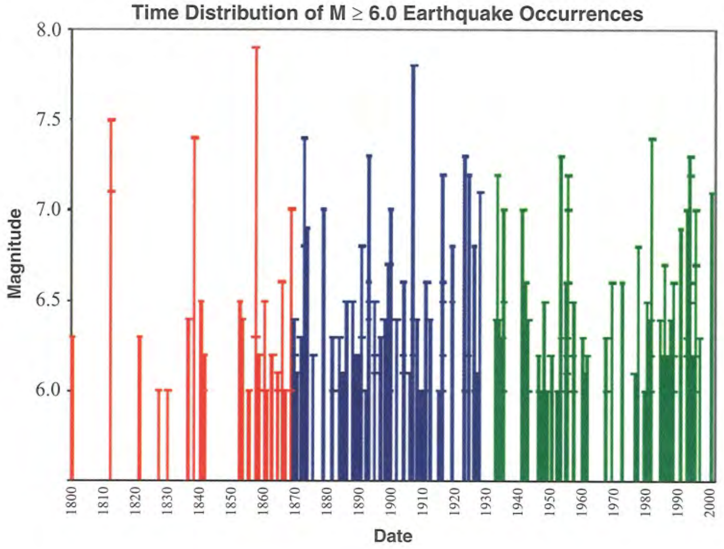


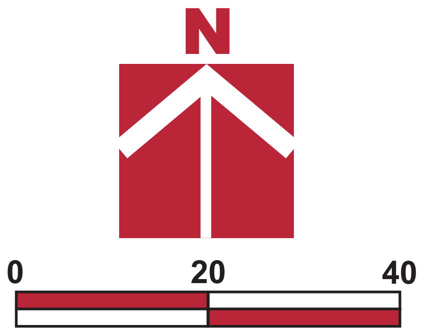
Area Within a 100km (62.4 mile) radius

From: T. Toppozada & Others (2000)



Time Distribution of M ≥ 6.0 Earthquake Occurrences





APPROXIMATE SCALE (MILES)

Project Number 826-2-1

Figure Number Figure 5

Drawn By RRN

Date June 2016

Historical Earthquakes Map

Charter Square K-5 School
Foster City, CA

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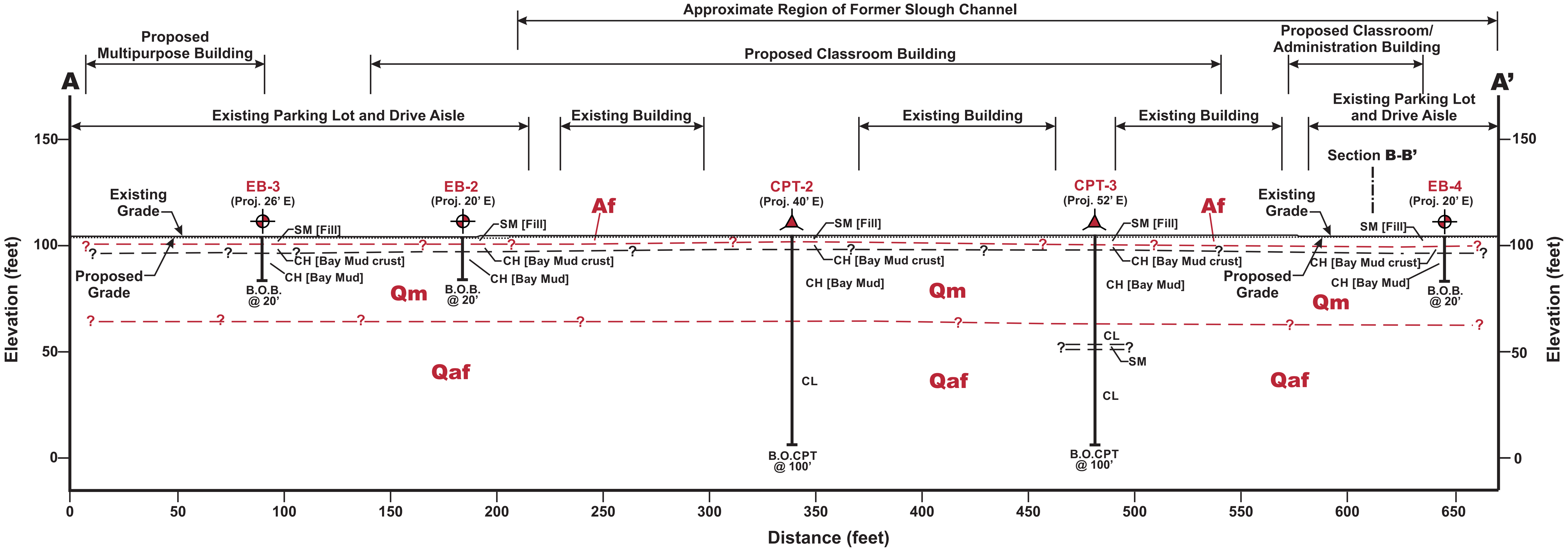

Section A-A'

(View Looking East)
1"=50' H:V

Explanation

Geologic Units

Af	Artificial fill
Qm	Bay Mud (Holocene)
Qaf	Fine-grained alluvium (Holocene)
— — —	Geologic contact

Symbols

CL	Lean Clay/Silty Clay
CH	Fat Clay
SM	Silty Sand
⌖	Approximate location of exploratory boring (EB)
▲	Approximate location of cone penetration test (CPT)

Notes:
1) Existing site elevations based on existing topographic map (Lea & Braze Engineering, Inc., Topographic Survey - Sheet No. 1, 11/26/2006.
2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
4) See Figure 2 for location of cross section.

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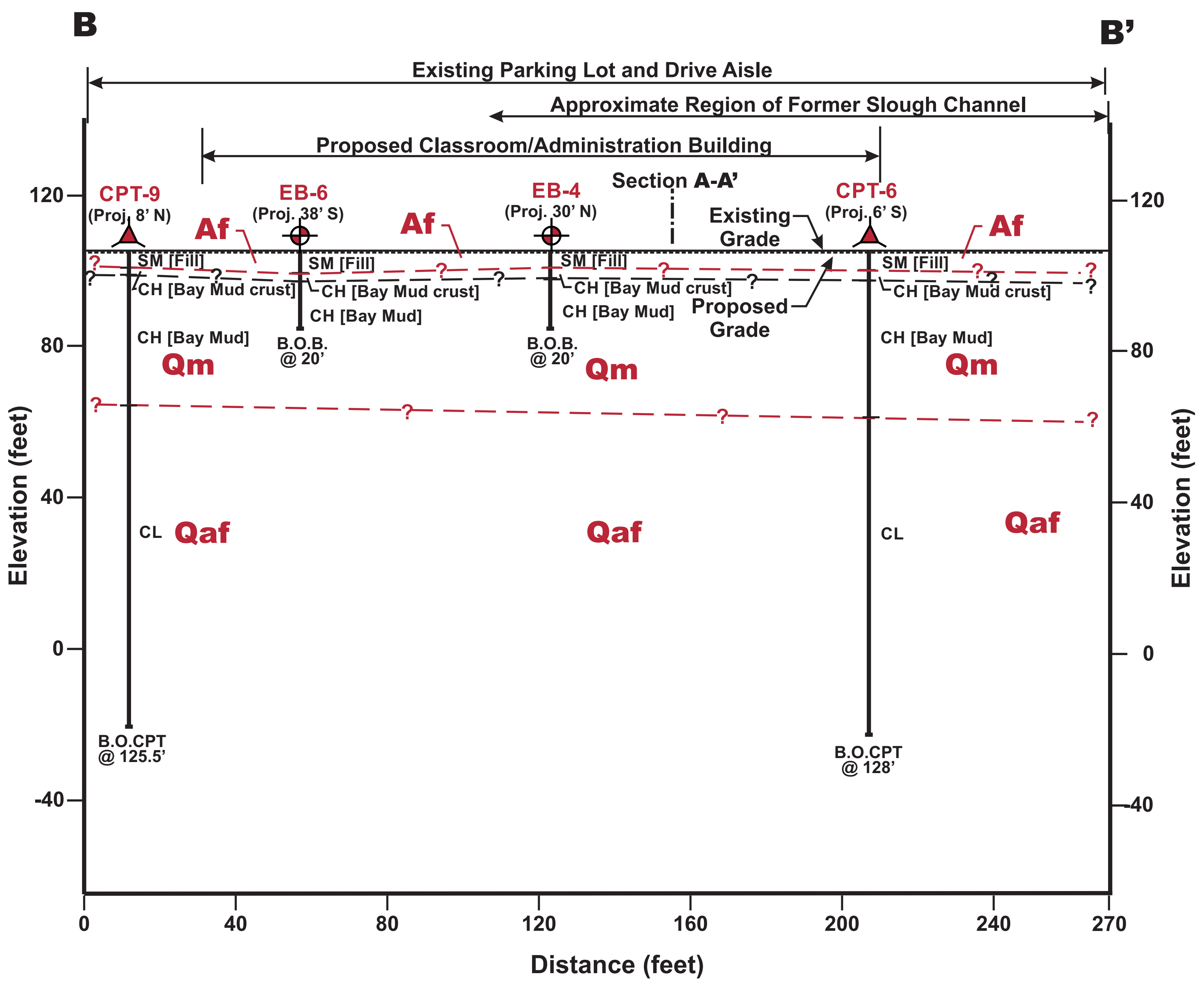
Geologic Cross Section A-A'

Charter Square K-5 School
Foster City, CA

Project Number
826-2-1

Figure Number
Figure 6

Date July 2017
Drawn By RRN/ESY



Section B-B'

(View Looking North)
1"=40' H:V

Explanation

Geologic Units

Af	Artificial fill
Qm	Bay Mud (Holocene)
Qaf	Fine-grained alluvium (Holocene)
– – –	Geologic contact

Symbols

CL	Lean Clay/Silty Clay
CH	Fat Clay
SM	Silty Sand
⊕	Approximate location of exploratory boring (EB)
▲	Approximate location of cone penetration test (CPT)

Notes:
1) Existing site elevations based on existing topographic map (Lea & Braze Engineering, Inc., Topographic Survey - Sheet No. 1, 11/26/2006.
2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
4) See Figure 2 for location of cross section.

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Geologic Cross Section B-B'

Charter Square K-5 School
Foster City, CA

Project Number
826-2-1

Figure Number
Figure 7

Date
July 2017

Drawn By
FLL, ESY

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FIGURE 8A

CPT NO. 1

PROJECT/CPT DATA

Project Title	**Charter Square K-5 School**
Project No.	**826-2-1**
Project Manager	**MJS**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.573**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**5**
Design Water Depth (feet)	**4**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.32 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.23 **L/H** 118.5

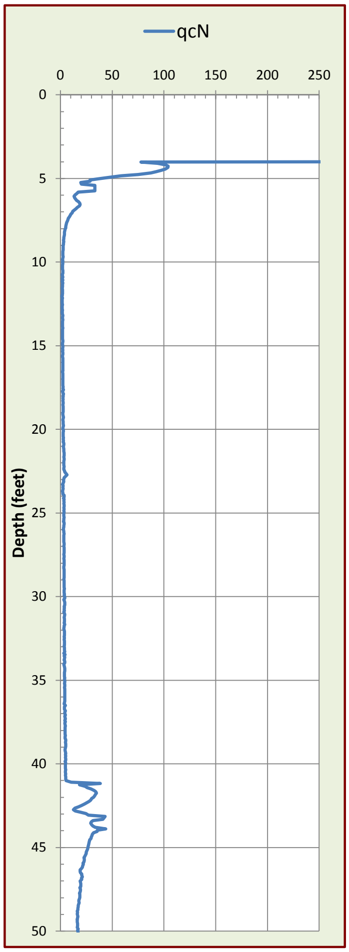
LDI$^1_{\text{Corrected for Distance}}$ 0.03 (4 < L/H < 40)

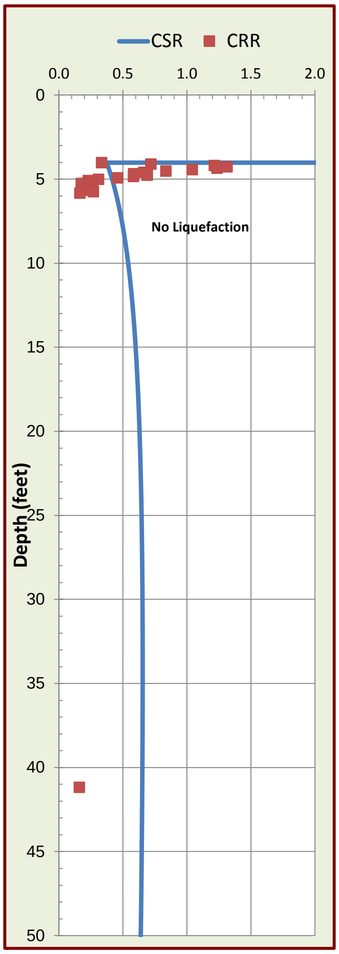
EXPECTED RANGE OF DISPLACEMENT

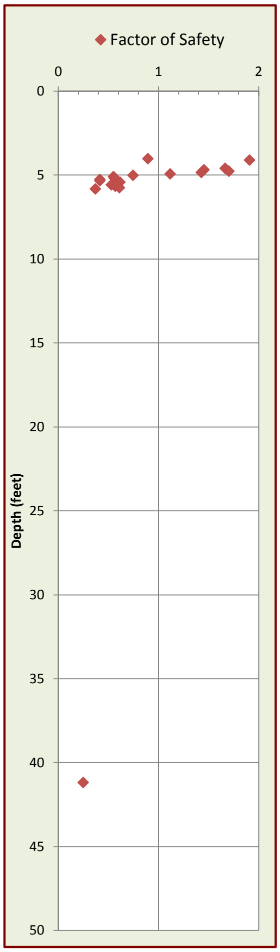
0.0 **to** **0.1** **feet**

[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.







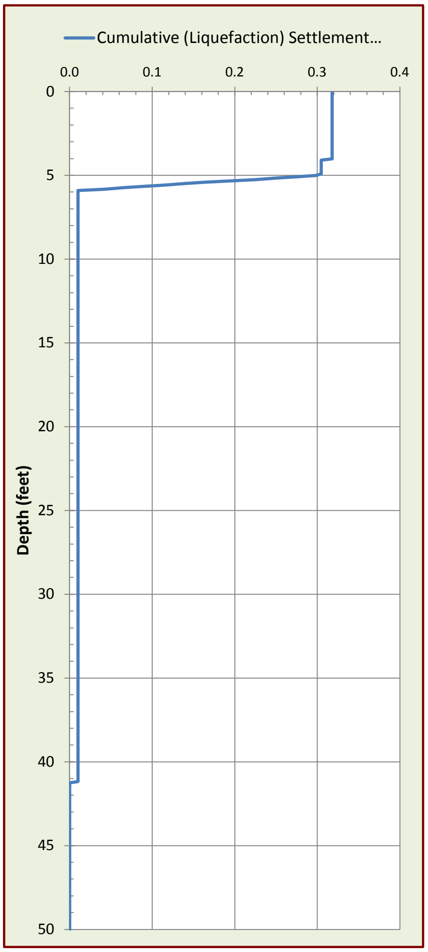


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FIGURE 8B

CPT NO. 2

PROJECT/CPT DATA

Project Title **Charter Square K-5 School**

Project No. **826-2-1**

Project Manager **MJS**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.573** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **5**

Design Water Depth (feet) **4**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.39 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.35 **L/H** 118.0

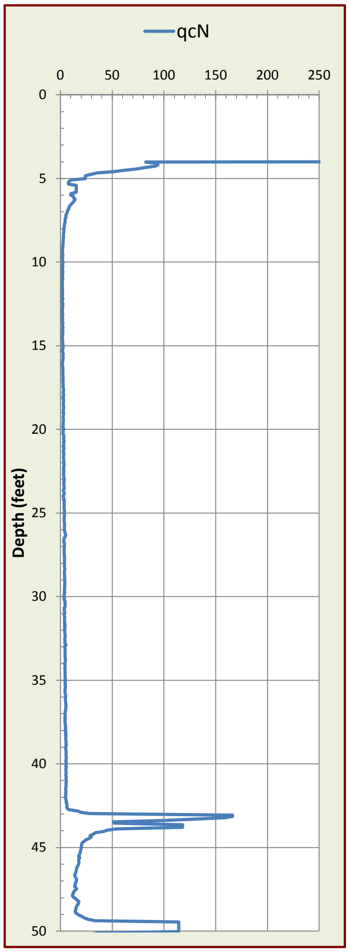
LDI^1$_{\text{Corrected for Distance}}$ 0.05 (4 < L/H < 40)

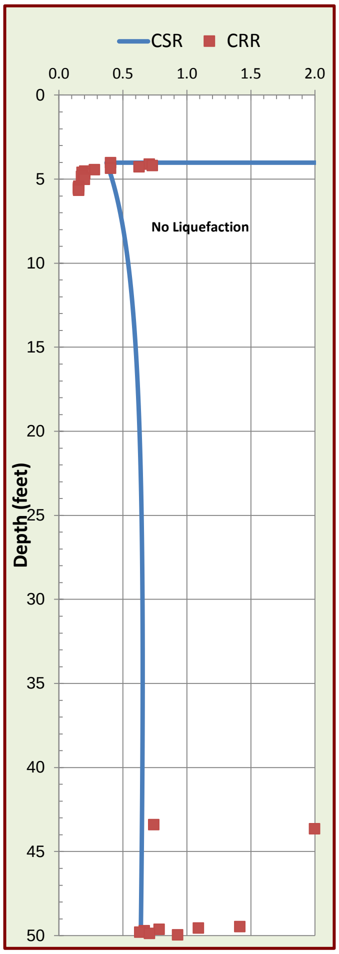
EXPECTED RANGE OF DISPLACEMENT

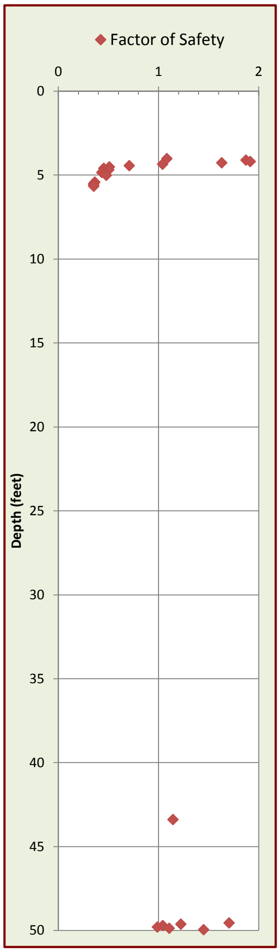
0.0 **to** **0.1** **feet**

[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.







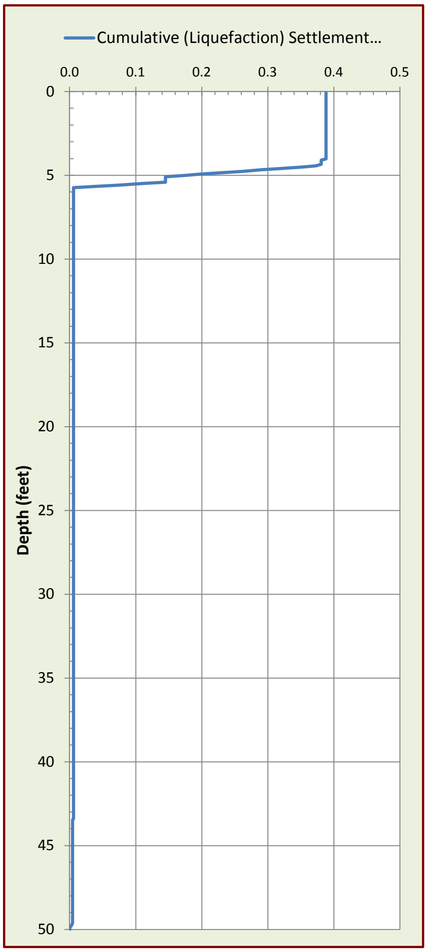


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FIGURE 8C

CPT NO. 3

PROJECT/CPT DATA

Project Title **Charter Square K-5 School**

Project No. **826-2-1**

Project Manager **MJS**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.573** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **5.75**

Design Water Depth (feet) **4.75**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4.75 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.19 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.13 **L/H** 116.5

LDI[1]$_{\textbf{Corrected for Distance}}$ 0.02 (4 < L/H < 40)

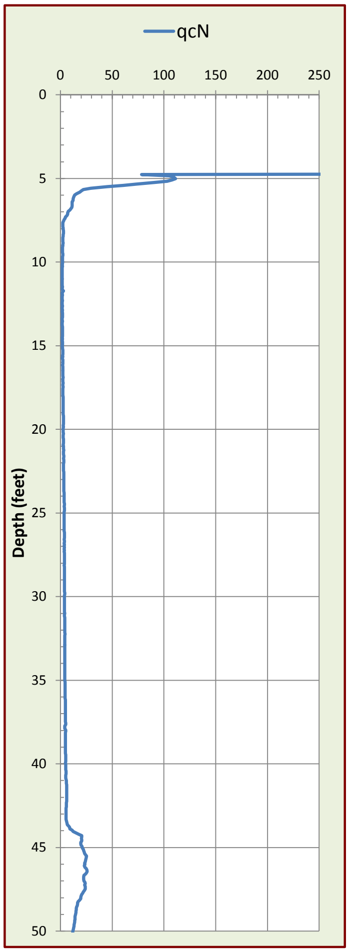
EXPECTED RANGE OF DISPLACEMENT

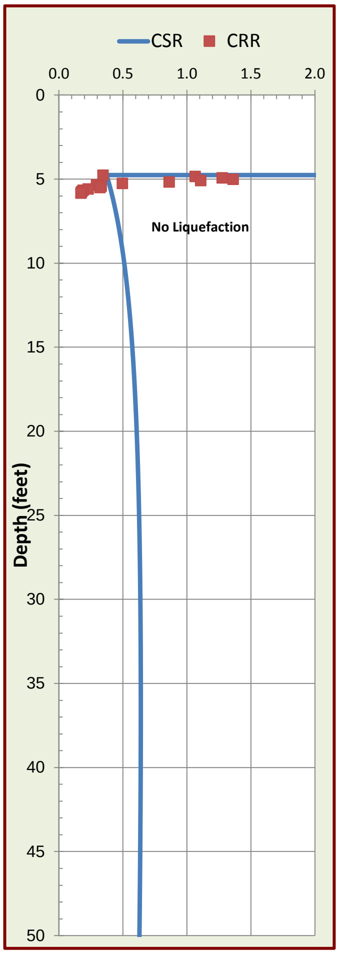
0.0 **to** **0.0** **feet**

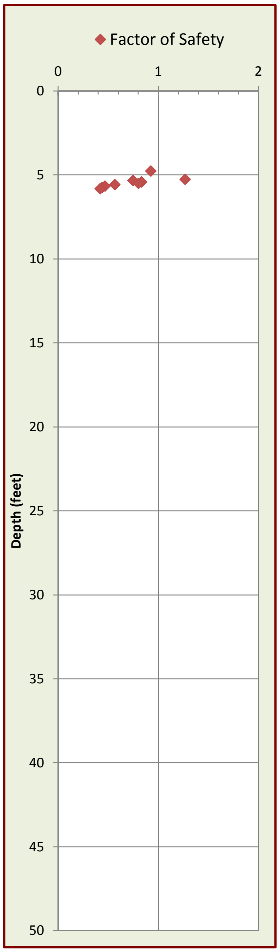
---

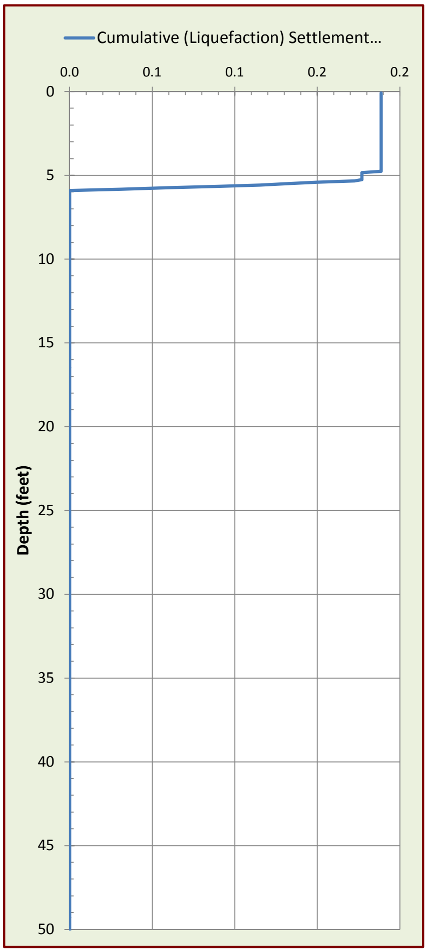
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 8D

CPT NO. 4

PROJECT/CPT DATA

Project Title **Charter Square K-5 School**

Project No. **826-2-1**

Project Manager **MJS**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.573** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **6**

Design Water Depth (feet) **5**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.30 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.25 **L/H** 124.0

LDI[1]**$_{\text{Corrected for Distance}}$** 0.03 (4 < L/H < 40)

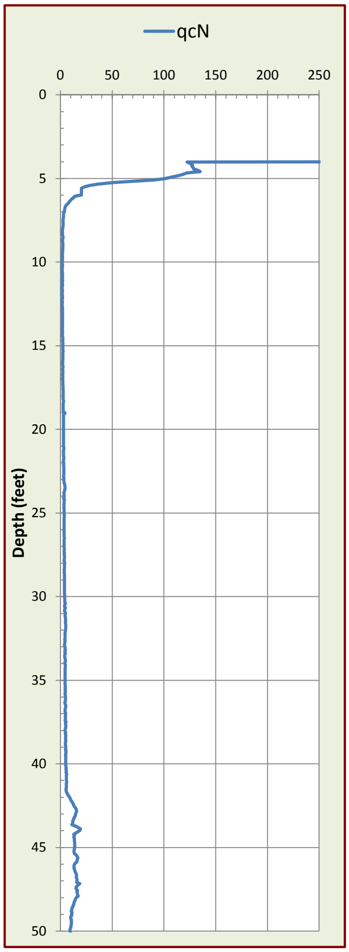
EXPECTED RANGE OF DISPLACEMENT

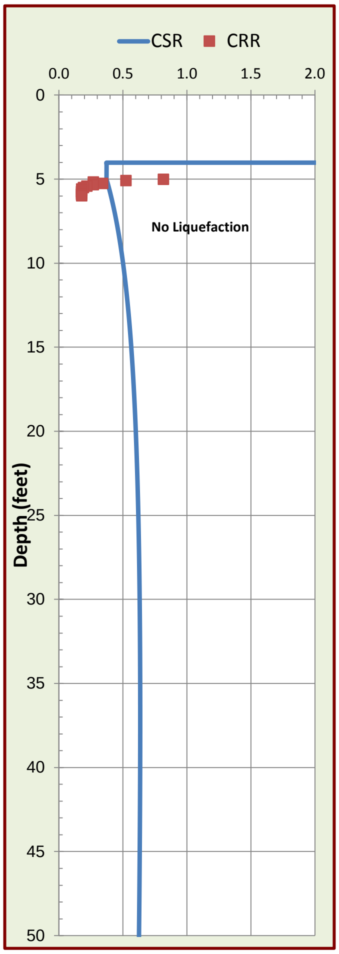
0.0 **to** **0.1** **feet**

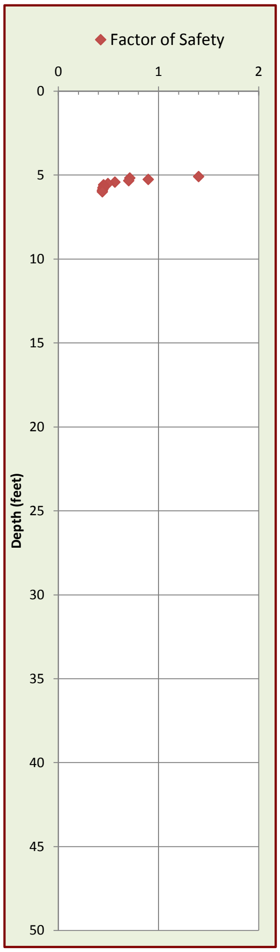
---

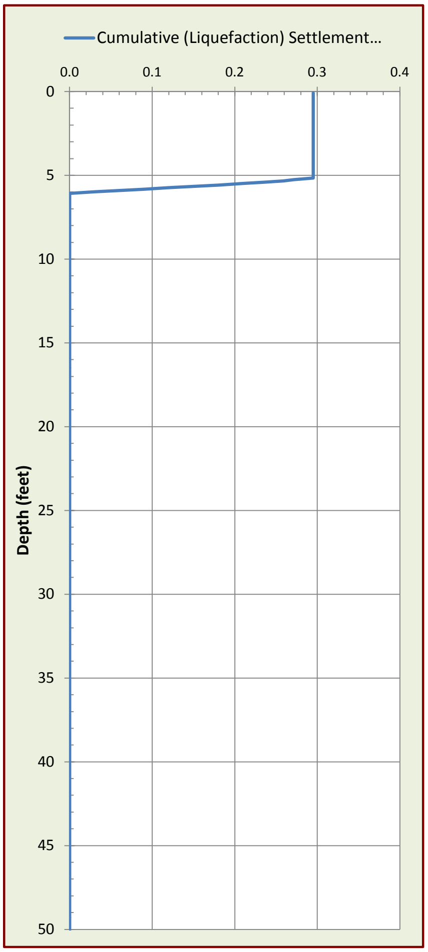
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 8E

CPT NO. **5**

PROJECT/CPT DATA

Project Title **Charter Square K-5 School**

Project No. **826-2-1**

Project Manager **MJS**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.573** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **4.5**

Design Water Depth (feet) **3.5**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 3.5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.24 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.10 **L/H** 98.0

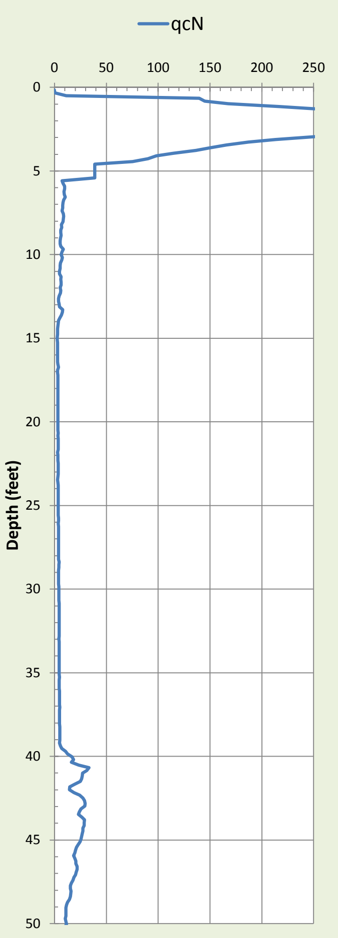
LDI[1]$_{\text{Corrected for Distance}}$ 0.01 (4 < L/H < 40)

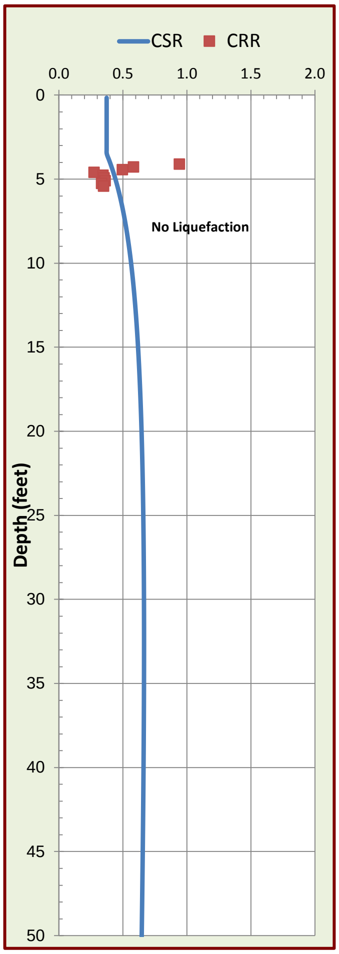
EXPECTED RANGE OF DISPLACEMENT

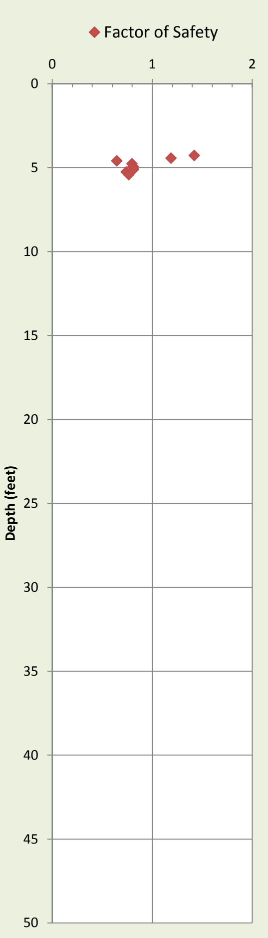
0.0 **to** **0.0** **feet**

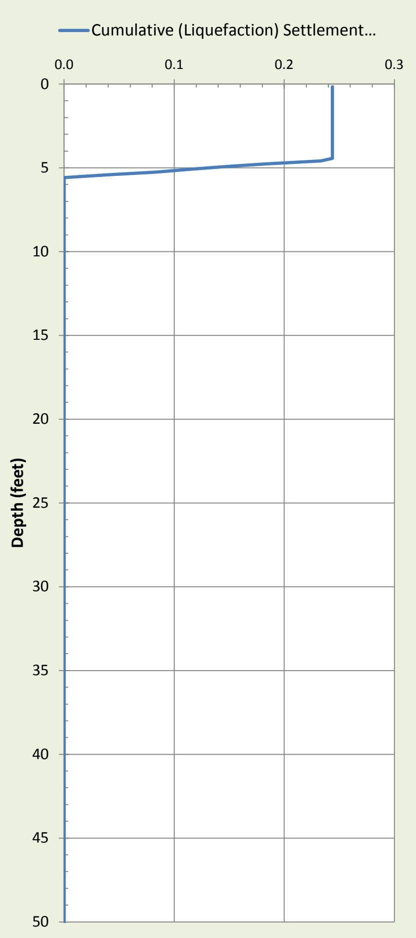
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 8F

CPT NO. 6

PROJECT/CPT DATA

Project Title **Charter Square K-5 School**

Project No. **826-2-1**

Project Manager **MJS**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.573** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **4.5**

Design Water Depth (feet) **3.5**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 3.5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.24 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.22 **L/H** 101.0

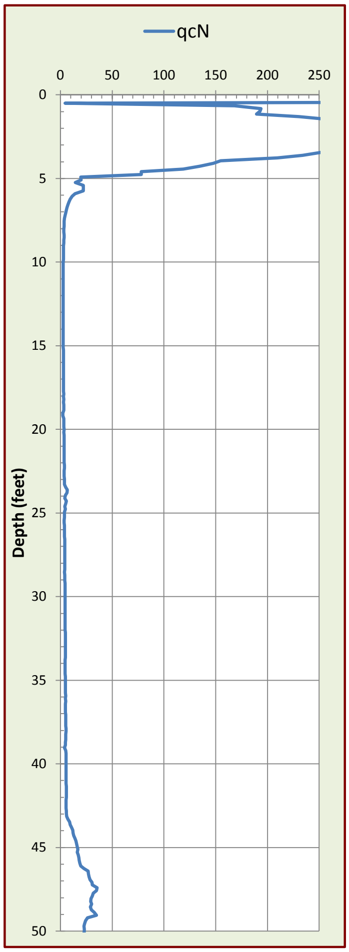
LDI[1]$_{\text{Corrected for Distance}}$ 0.03 (4 < L/H < 40)

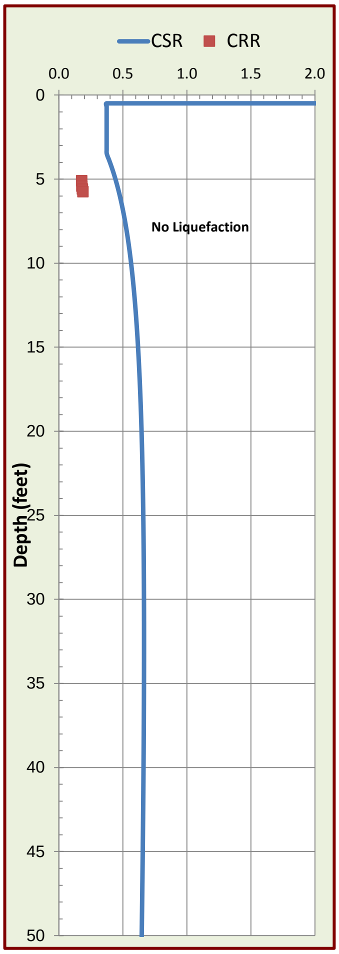
EXPECTED RANGE OF DISPLACEMENT

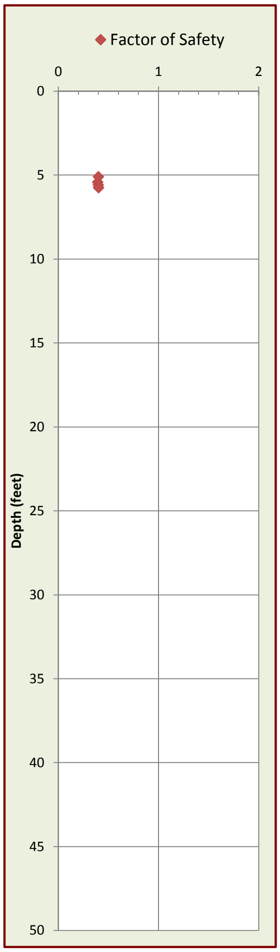
0.0 **to** **0.1** **feet**

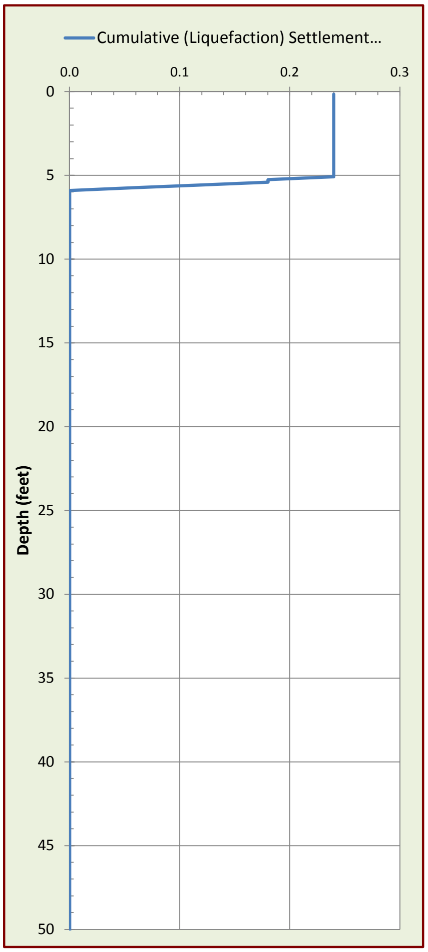
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 8G

CPT NO. 7

PROJECT/CPT DATA

Project Title	**Charter Square K-5 School**
Project No.	**826-2-1**
Project Manager	**MJS**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.573**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**4.25**
Design Water Depth (feet)	**3.25**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 3.25 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.51 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.37 **L/H** 86.0

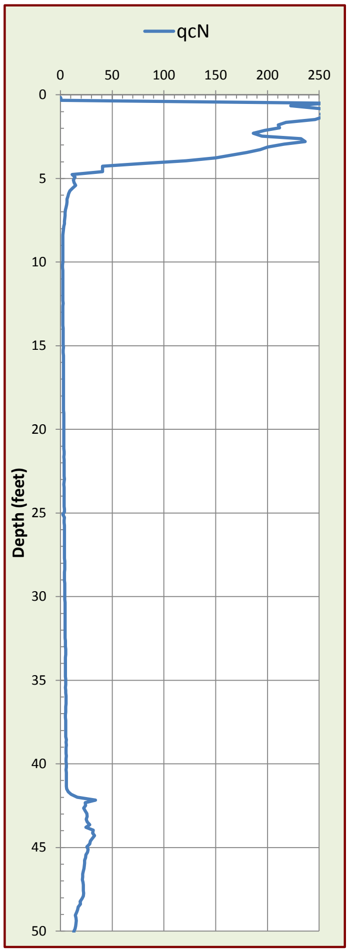
LDI[1]**Corrected for Distance** 0.06 (4 < L/H < 40)

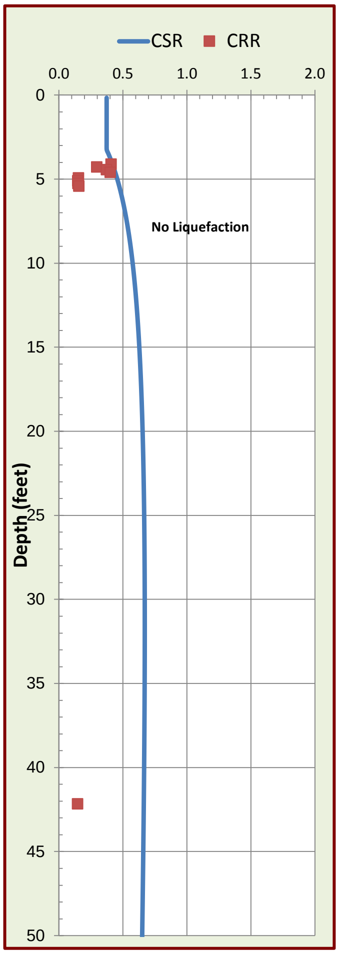
EXPECTED RANGE OF DISPLACEMENT

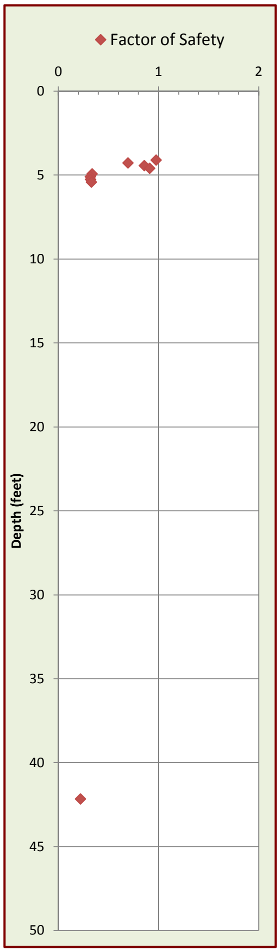
0.0 **to** **0.1** **feet**

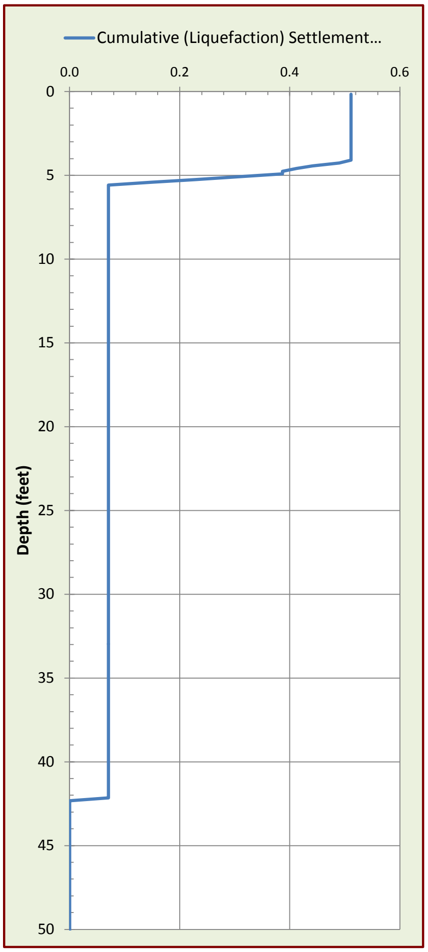
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE **8H**

CPT NO. **8**

PROJECT/CPT DATA

Project Title	**Charter Square K-5 School**
Project No.	**826-2-1**
Project Manager	**MJS**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.573**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**4.25**
Design Water Depth (feet)	**3.25**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 3.25 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.39 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.34 **L/H** 101.0

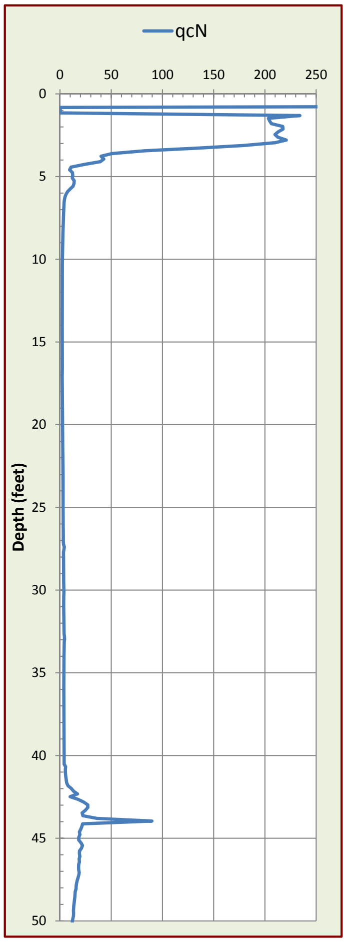
LDI[1]**Corrected for Distance** 0.05 (4 < L/H < 40)

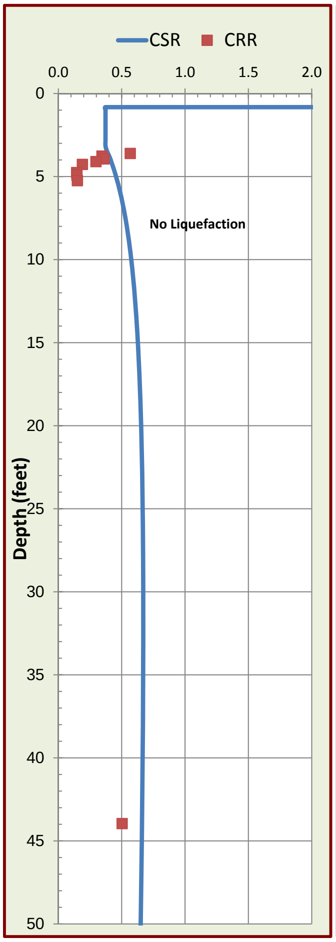
EXPECTED RANGE OF DISPLACEMENT

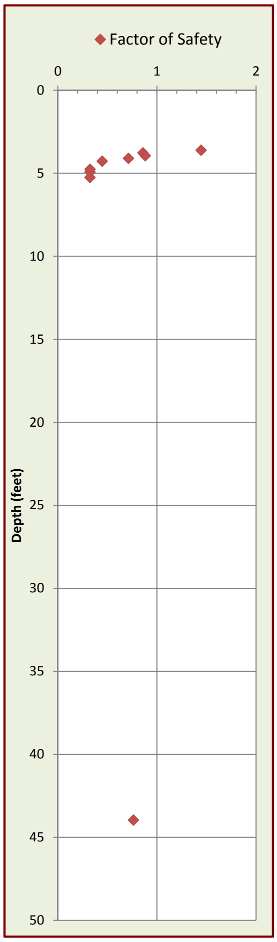
0.0 **to** **0.1** **feet**

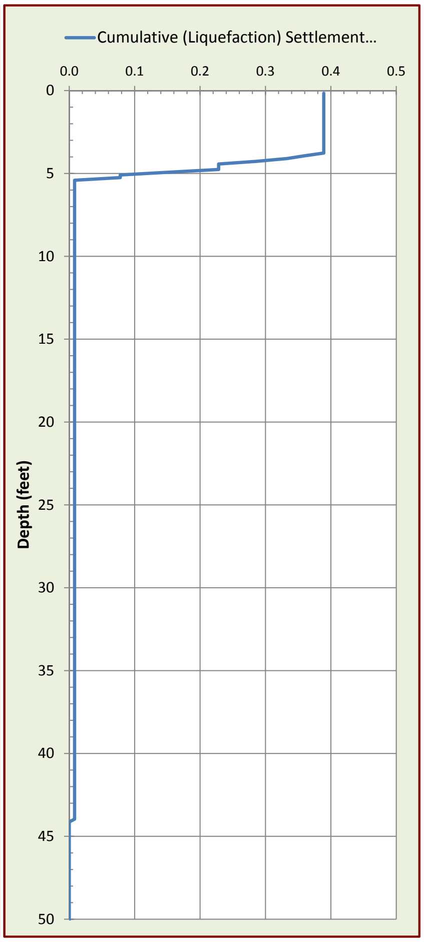
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 8I

CPT NO. 9

PROJECT/CPT DATA

Project Title	**Charter Square K-5 School**
Project No.	**826-2-1**
Project Manager	**MJS**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.573**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**5**
Design Water Depth (feet)	**4**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**115**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.00 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.0 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 110.0

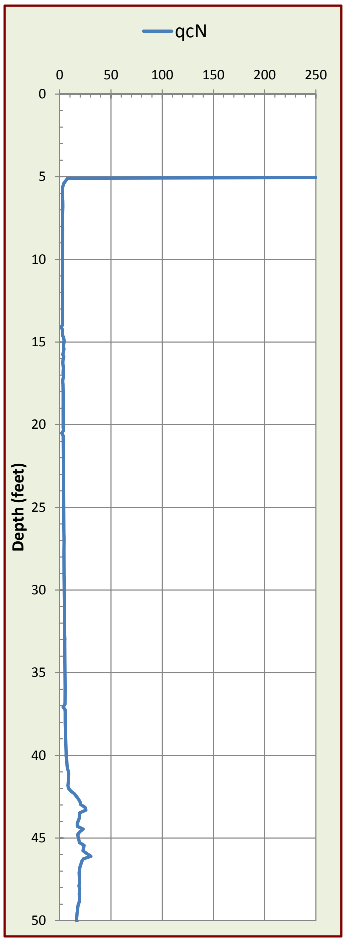
LDI[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

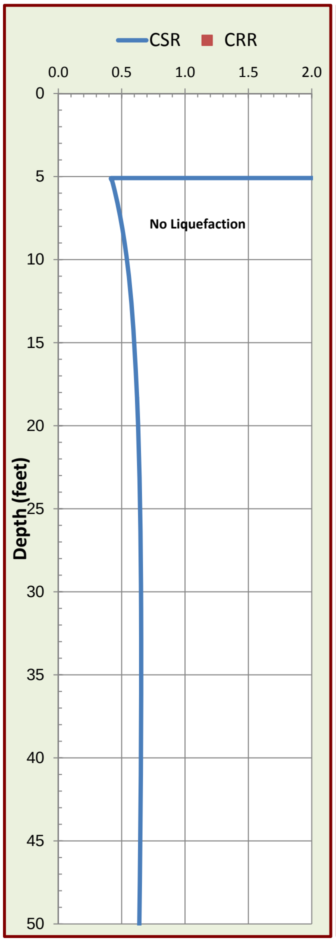
EXPECTED RANGE OF DISPLACEMENT

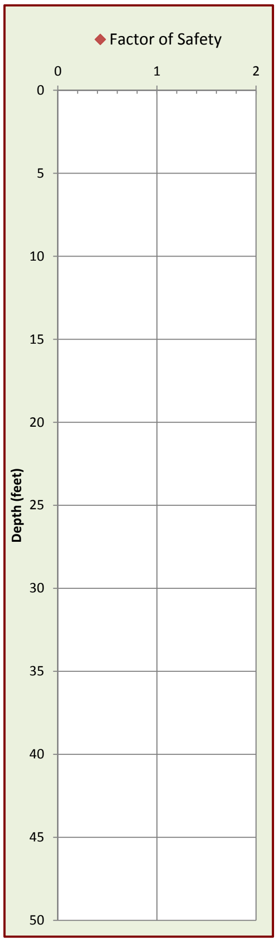
0.0 **to** **0.0** **feet**

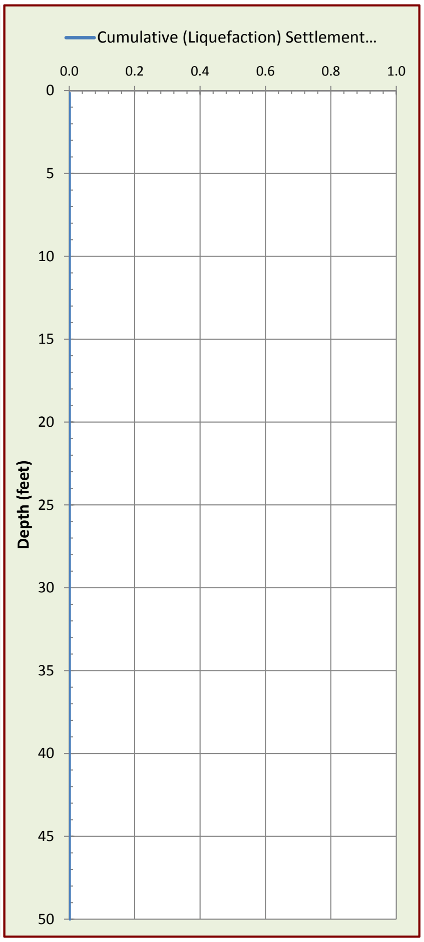
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









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, 20-ton truck-mounted Cone Penetration Test equipment, and track-mounted Cone Penetration Test equipment. Eight 8-inch-diameter exploratory borings were drilled on May 26 and 27, 2016 to depths of 20 to 55 feet. Nine CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on May 23 and 25 and June 8, 2016, to depths ranging from approximately 93 to 128 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan and Geologic Map, 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 based on interpolation of plan contours. The locations and elevations 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 are 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-2487-98)

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

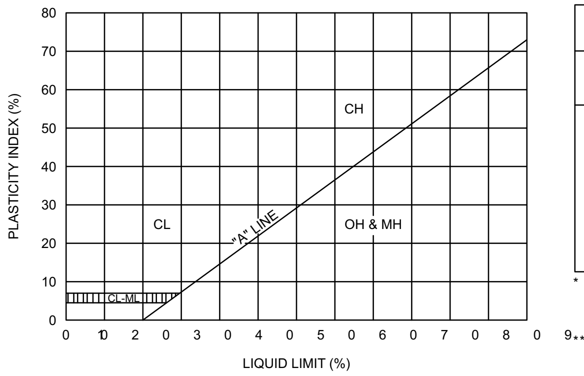
SAMPLER TYPES

☒	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)		-	
RV	-	R-VALUE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
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

BORING NUMBER EB-1

PAGE 1 OF 1

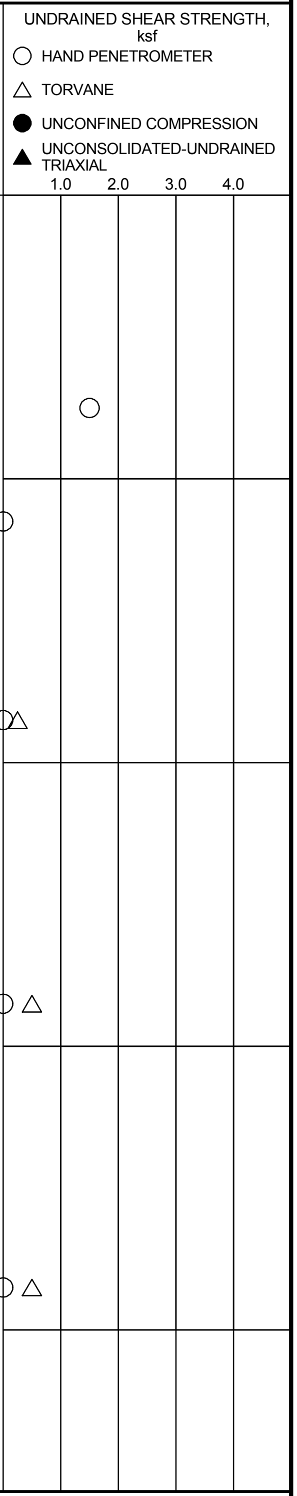
CORNERSTONE
EARTH GROUP

PROJECT NAME Charter Square K-5 School
PROJECT NUMBER 826-2-1
PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA
DATE STARTED 5/26/16 **DATE COMPLETED** 5/26/16
GROUND ELEVATION 105 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.54941° **LONGITUDE** -122.26436°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION
105.0 104.8	0	■	3 inches asphalt concrete
		▨	**Silty Sand (SM) [Fill]** medium dense, moist, gray brown, fine to medium sand, some shell fragments
101.0		▨	**Fat Clay (CH) [Bay Mud Crust] [Qm]** stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity
99.0		▨	**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity
85.0	20		Bottom of Boring at 20.0 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
38	MC-1B	108	17		
16	MC-2B	99	19		
	ST-3	66	54		
3	MC-4B	46	90		
2	MC-5B	46	94		
3	MC				



BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME Charter Square K-5 School
PROJECT NUMBER 826-2-1
PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA
DATE STARTED 5/27/16 **DATE COMPLETED** 5/27/16
GROUND ELEVATION 105 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.55030° **LONGITUDE** -122.26453°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
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
105.0 104.8	0	2 inches asphalt concrete over 6 inches aggregate base							
104.3		**Silty Sand (SM) [Fill]** medium dense, moist, gray brown, fine to medium sand, some shell fragments		GB					
				GB					
100.5	5	**Fat Clay (CH) [Bay Mud Crust] [Qm]** stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity	12	MC-3B	68	53			○
97.5		**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity	2	MC-4B	40	118			○△
	10								
			3	MC					△
	15								
			4	MC-6B	48	90			○
85.0	20	Bottom of Boring at 20.0 feet.							

BORING NUMBER EB-3

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Charter Square K-5 School

PROJECT NUMBER 826-2-1

PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA

DATE STARTED 5/26/16 **DATE COMPLETED** 5/26/16

GROUND ELEVATION 104 FT +/- **BORING DEPTH** 20 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.55056° **LONGITUDE** -122.26449°

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

GROUND WATER LEVELS:

LOGGED BY OL

∇ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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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
104.0	0	2 inches asphalt concrete						
103.8		**Silty Sand (SM) [Fill]** medium dense, moist, gray brown, fine to medium sand, some shell fragments		1A		19		
			47	MC				
				1B	106	18		
100.5		**Fat Clay (CH) [Bay Mud Crust] [Qm]** medium stiff to stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity	11	MC-2B	89	46		
	5		12	MC-3B	78	44		
97.0		**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity						
			2	MC-4B	44	98		
	10							
			2	MC-5B	43	94		
	15							
			3	MC				
84.0	20	Bottom of Boring at 20.0 feet.						

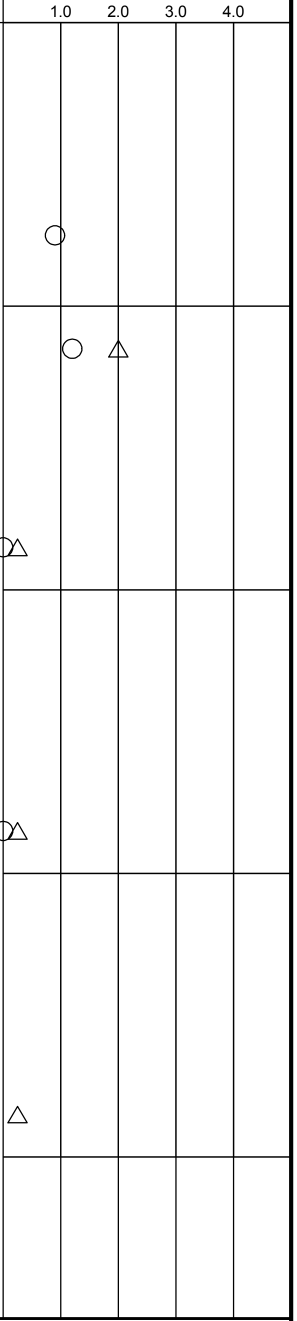
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Charter Square K-5 School
PROJECT NUMBER 826-2-1
PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA
DATE STARTED 5/26/16 **DATE COMPLETED** 5/26/16
GROUND ELEVATION 104 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.54906° **LONGITUDE** -122.26482°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

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 (1.0 2.0 3.0 4.0)
104.0	0	2 inches asphalt concrete							
103.8		**Silty Sand (SM) [Fill]** medium dense, moist, gray brown, fine to medium sand, some shell fragments							
			36	MC-1B	104	17			
			29	MC					○ 2.5
99.8		**Fat Clay (CH) [Bay Mud Crust] [Qm]** very stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity							
99.0	5	**Silty Sand (SM)** loose, moist, gray, fine sand							
98.5		**Fat Clay (CH) [Bay Mud Crust]** very stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity	14	MC-3B	75	43			○ 2.5
97.0		**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity							
			3	MC-4B	49	88			○ 0.0 △ 0.25
	10			ST-5	45	100			▲ 0.3
			4	MC					○ 0.0 △ 0.5
	15								
			3	MC-7B	44	99			△ 0.25
84.0	20	Bottom of Boring at 20.0 feet.							

BORING NUMBER EB-5

PAGE 1 OF 1

PROJECT NAME Charter Square K-5 School
PROJECT NUMBER 826-2-1
PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA
DATE STARTED 5/26/16 **DATE COMPLETED** 5/26/16
GROUND ELEVATION 104 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.55055° **LONGITUDE** -122.26477°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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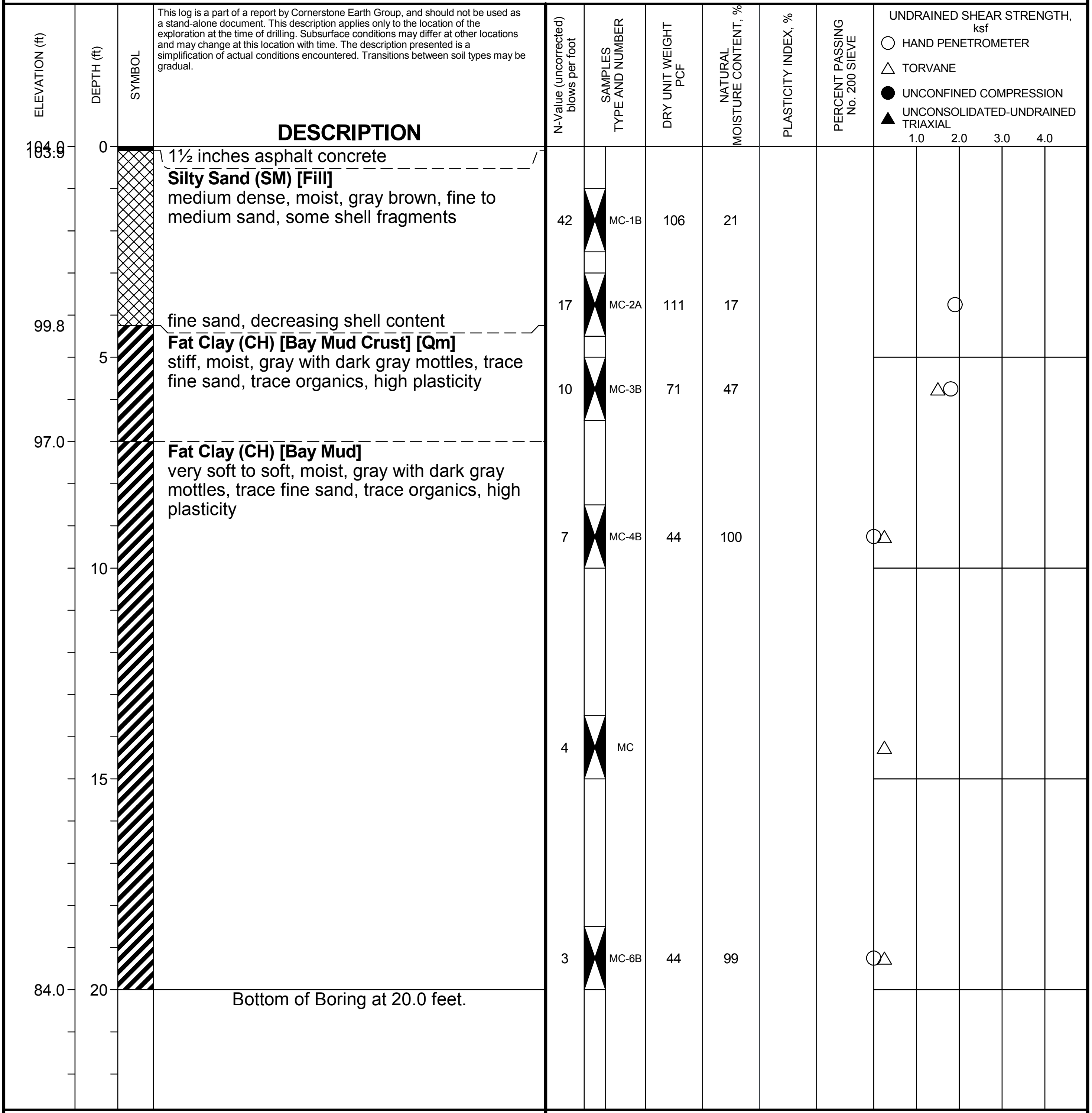

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
104.0 103.9	0	1½ inches asphalt concrete						
		Silty Sand (SM) [Fill] medium dense, moist, gray brown, fine to medium sand, some shell fragments	42	MC-1B	106	21		
			17	MC-2A	111	17		
		fine sand, decreasing shell content						
99.8		**Fat Clay (CH) [Bay Mud Crust] [Qm]** stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity						
	5		10	MC-3B	71	47		
97.0		**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity						
			7	MC-4B	44	100		
	10							
			4	MC				
	15							
			3	MC-6B	44	99		
84.0	20	Bottom of Boring at 20.0 feet.						

BORING NUMBER EB-6

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Charter Square K-5 School

PROJECT NUMBER 826-2-1

PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA

DATE STARTED 5/27/16 **DATE COMPLETED** 5/27/16

GROUND ELEVATION 105 FT +/- **BORING DEPTH** 20 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.54922° **LONGITUDE** -122.26507°

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

GROUND WATER LEVELS:

LOGGED BY OL

▽ **AT TIME OF DRILLING** 3.1 ft.

NOTES

▼ **AT END OF DRILLING** 3.1 ft.

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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
105.0 104.8	0		2 inches asphalt concrete							
			Silty Sand (SM) [Fill] medium dense, moist, gray brown, fine to medium sand, some shell fragments	32	MC-1B	103	17			
		▼	becomes loose, wet, abundant shell fragments	15	MC-2B	99	21			
99.5	5		**Silty Sand (SM) [Residual Soil]** loose, moist, gray, fine sand	10	MC-3B	74	39			○
99.0			**Fat Clay (CH) [Bay Mud Crust] [Qm]** medium stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity							
97.0			**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity	3	MC-4B	47	88			○△
	10									
				3	MC-5B	43	101			○△
	15									
			contains shells							
				3	MC					△
85.0	20		Bottom of Boring at 20.0 feet.							

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 6/30/16 13:22 - P:\DRAFTING\GINT FILES\826-2-1 CHARTER SQUARE K-5 SCHOOL.GPJ

BORING NUMBER EB-7

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Charter Square K-5 School
PROJECT NUMBER 826-2-1
PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA
DATE STARTED 5/26/16 **DATE COMPLETED** 5/26/16
GROUND ELEVATION 105 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.55080° **LONGITUDE** -122.26500°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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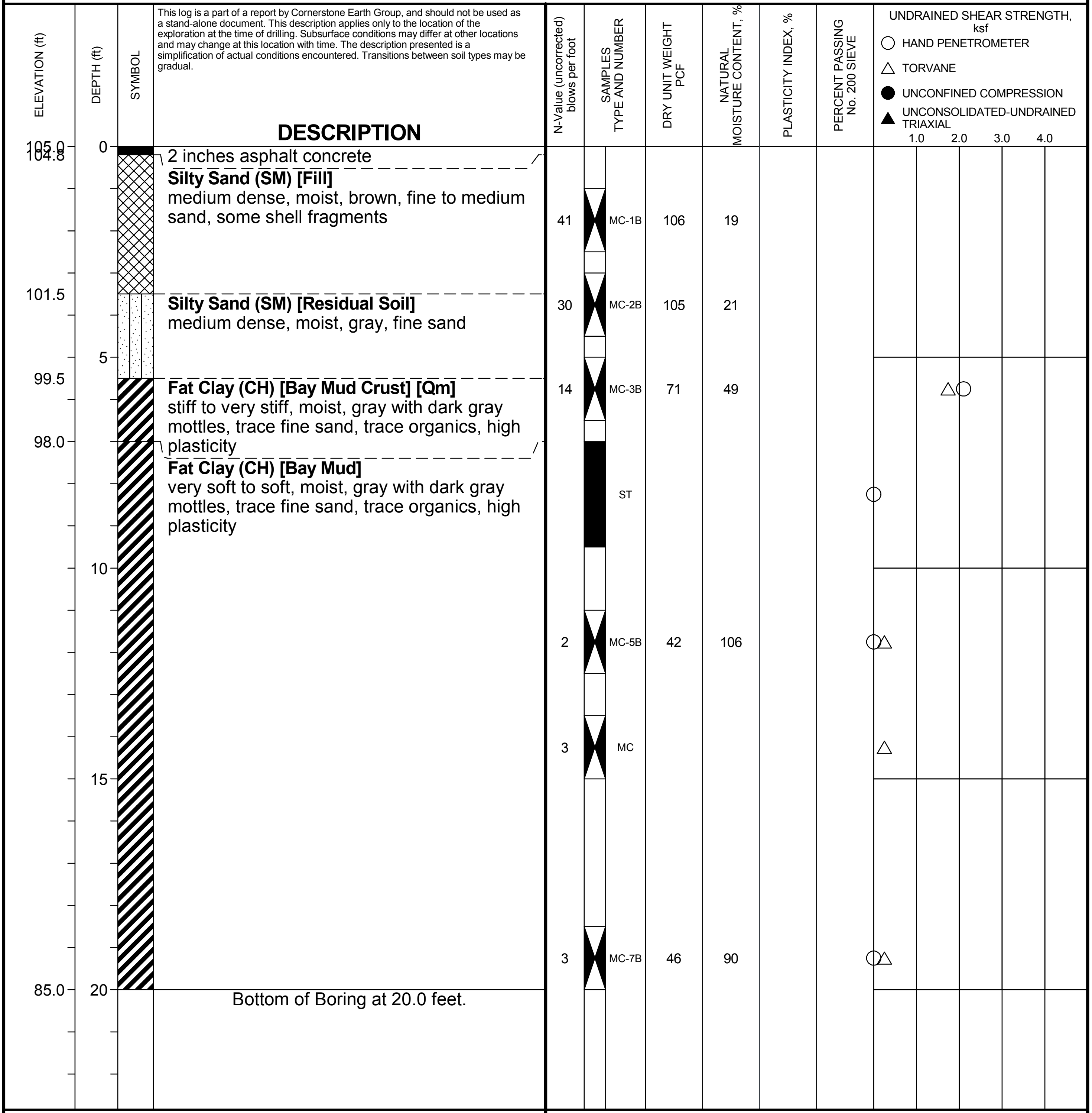

DESCRIPTION

2 inches asphalt concrete

Silty Sand (SM) [Fill]
medium dense, moist, brown, fine to medium sand, some shell fragments

Silty Sand (SM) [Residual Soil]
medium dense, moist, gray, fine sand

Fat Clay (CH) [Bay Mud Crust] [Qm]
stiff to very stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity

Fat Clay (CH) [Bay Mud]
very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity

Bottom of Boring at 20.0 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %
41	MC-1B	106	19
30	MC-2B	105	21
14	MC-3B	71	49
	ST		
2	MC-5B	42	106
3	MC		
3	MC-7B	46	90

BORING NUMBER EB-8

PAGE 1 OF 3

CORNERSTONE
EARTH GROUP

PROJECT NAME Charter Square K-5 School
PROJECT NUMBER 826-2-1
PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA
DATE STARTED 5/27/16 **DATE COMPLETED** 5/27/16
GROUND ELEVATION 105 FT +/- **BORING DEPTH** 55 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.55040° **LONGITUDE** -122.26506°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
∇ **AT TIME OF DRILLING** 11.1 ft.
NOTES
▼ **AT END OF DRILLING** 11.1 ft.

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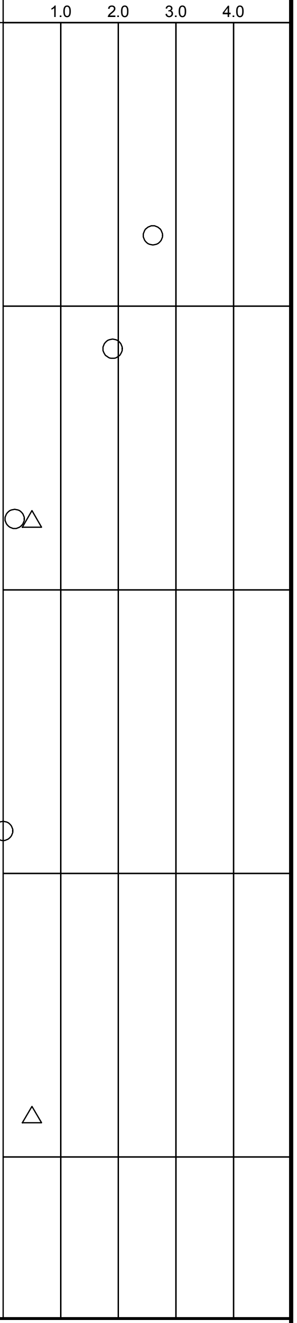
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
105.0	0	2 inches asphalt concrete
104.8		**Silty Sand (SM) [Fill]** medium dense, moist, brown, fine to medium sand, some shell fragments
102.5		**Silty Sand (SM)** medium dense, moist, gray, fine sand
100.8	5	**Fat Clay (CH) [Bay Mud Crust] [Qm]** very stiff to stiff, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity Liquid Limit = 82, Plastic Limit = 34
98.5	10 15 20	**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity
83.0		

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
43	MC-1C	104	20		
12	MC				
11	MC-3A	73	46	47	
2	MC-4C	45	96		
4	MC-5B	44	99		
3	MC				

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



BORING NUMBER EB-8

PROJECT NAME Charter Square K-5 School

PROJECT NUMBER 826-2-1

PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA

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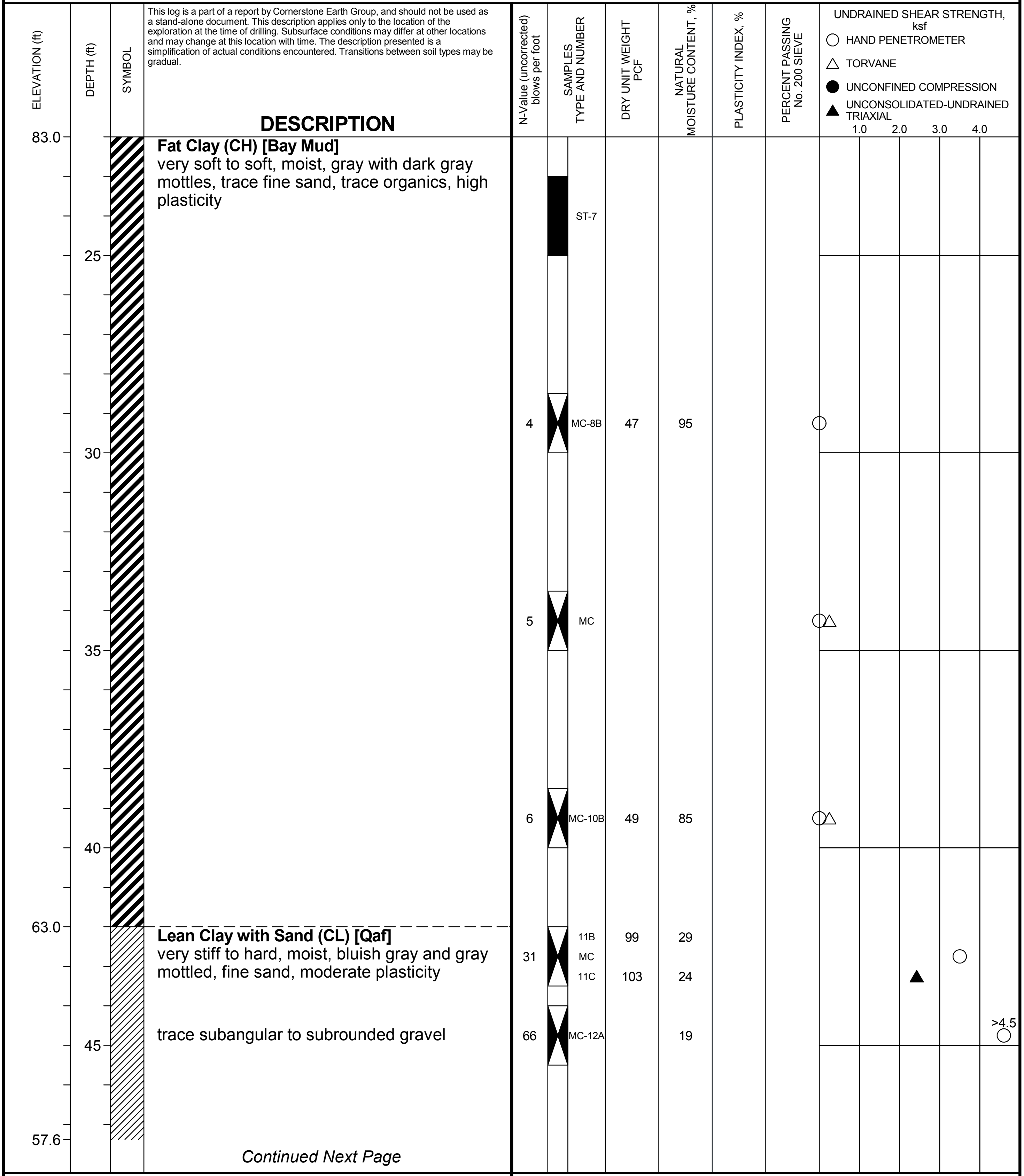


ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
83.0		**Fat Clay (CH) [Bay Mud]** very soft to soft, moist, gray with dark gray mottles, trace fine sand, trace organics, high plasticity							
				ST-7					
	25								
			4	MC-8B	47	95			○
	30								
			5	MC					○△
	35								
			6	MC-10B	49	85			○△
	40								
63.0		**Lean Clay with Sand (CL) [Qaf]** very stiff to hard, moist, bluish gray and gray mottled, fine sand, moderate plasticity		11B	99	29			
			31	MC					○
				11C	103	24			▲
		trace subangular to subrounded gravel	66	MC-12A		19			>4.5 ○
	45								
57.6									

Continued Next Page

BORING NUMBER EB-8

PROJECT NAME Charter Square K-5 School

PROJECT NUMBER 826-2-1

PROJECT LOCATION 1050 to 1088 Shell Boulevard, Foster City, CA

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

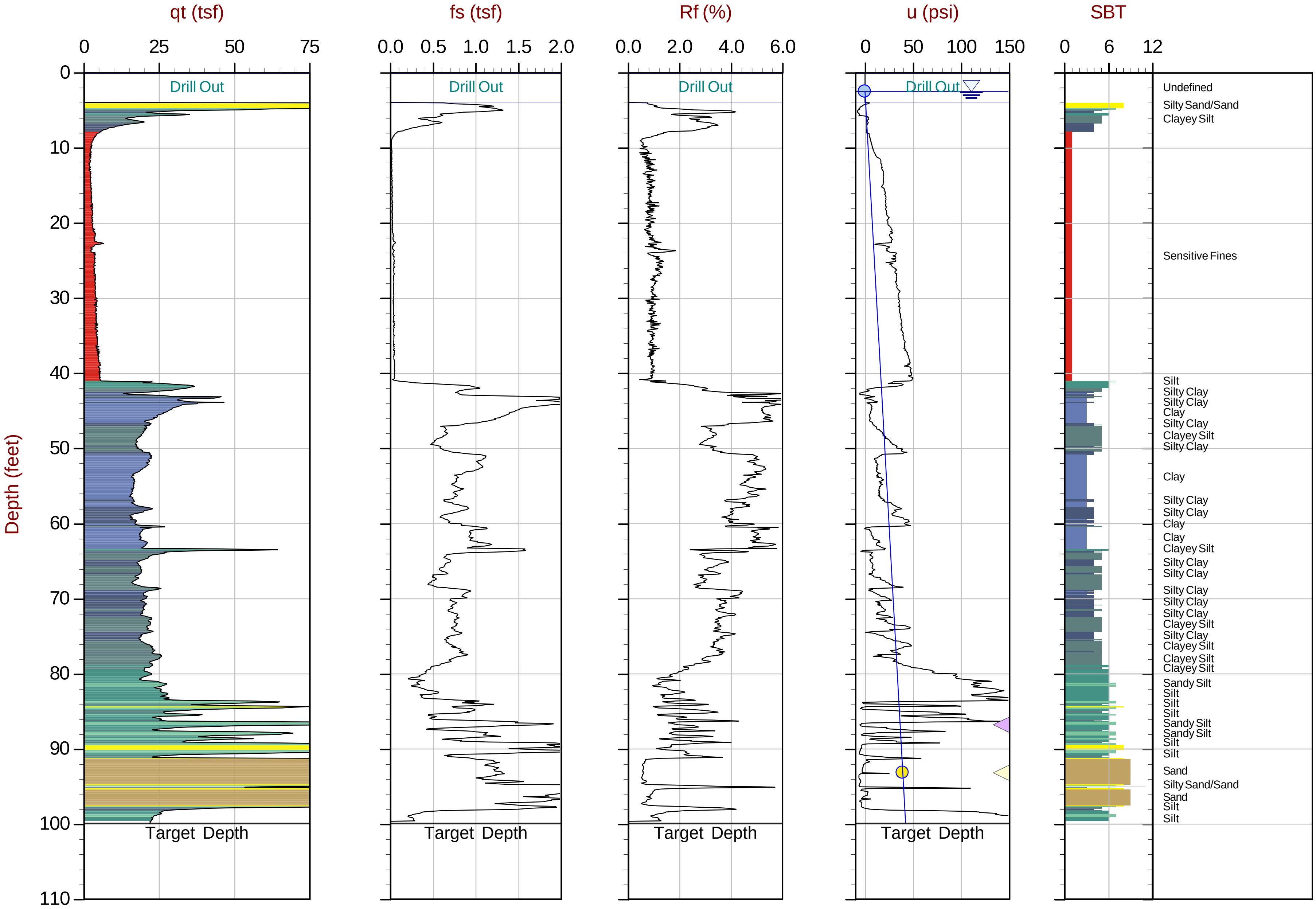
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
57.6 57.5		**Lean Clay (CL) [Qaf]** very stiff to stiff, moist, olive gray and gray mottled, some fine sand, moderate plasticity							
			33	MC-13B	90	32			▲ ○
	50								
			22	MC					○
51.0		**Silty Sand (SM)** dense, wet, brown, fine to medium sand	27	SPT					>4.5 ○
50.3		**Lean Clay (CL)** hard, moist, olive gray and gray mottled, some fine sand, moderate plasticity							
50.0	55	Bottom of Boring at 55.0 feet.							
	60								
	65								
	70								

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cornerstone Earth Group

Job No: 16-56031
Date: 05:23:16 07:54
Site: 1050 Shell Blvd

Sounding: CPT-01
Cone: 447:T1500F15U500



Max Depth: 30.450 m / 99.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 16-56031_CP01.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM 10 N N: 4156122 E: 564970
Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

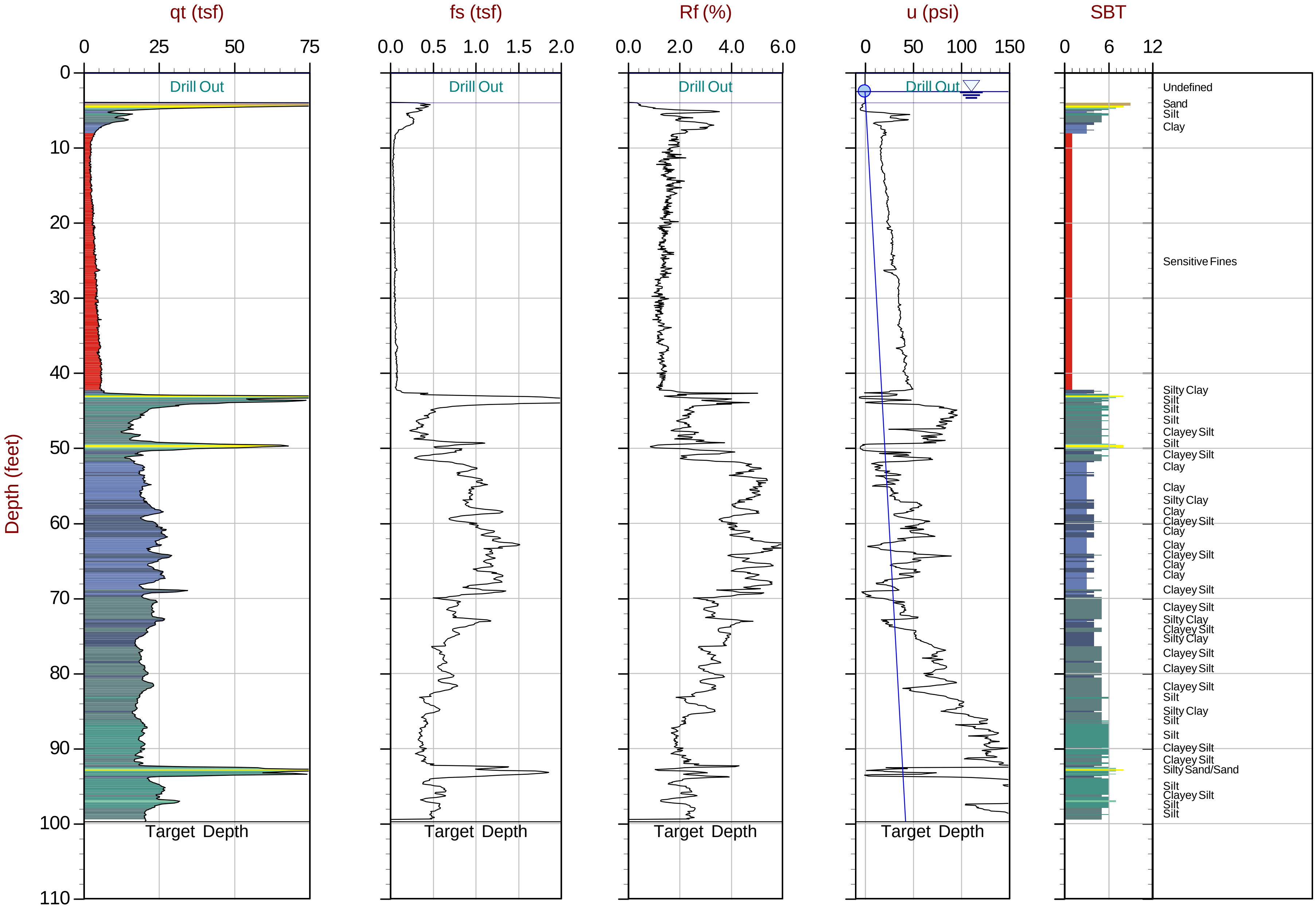
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cornerstone Earth Group

Job No: 16-56031
Date: 05:23:16 09:33
Site: 1050 Shell Blvd

Sounding: CPT-02
Cone: 447:T1500F15U500



Max Depth: 30.400 m / 99.74 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 16-56031_CP02.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM 10 N N: 4156133 E: 564951
Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

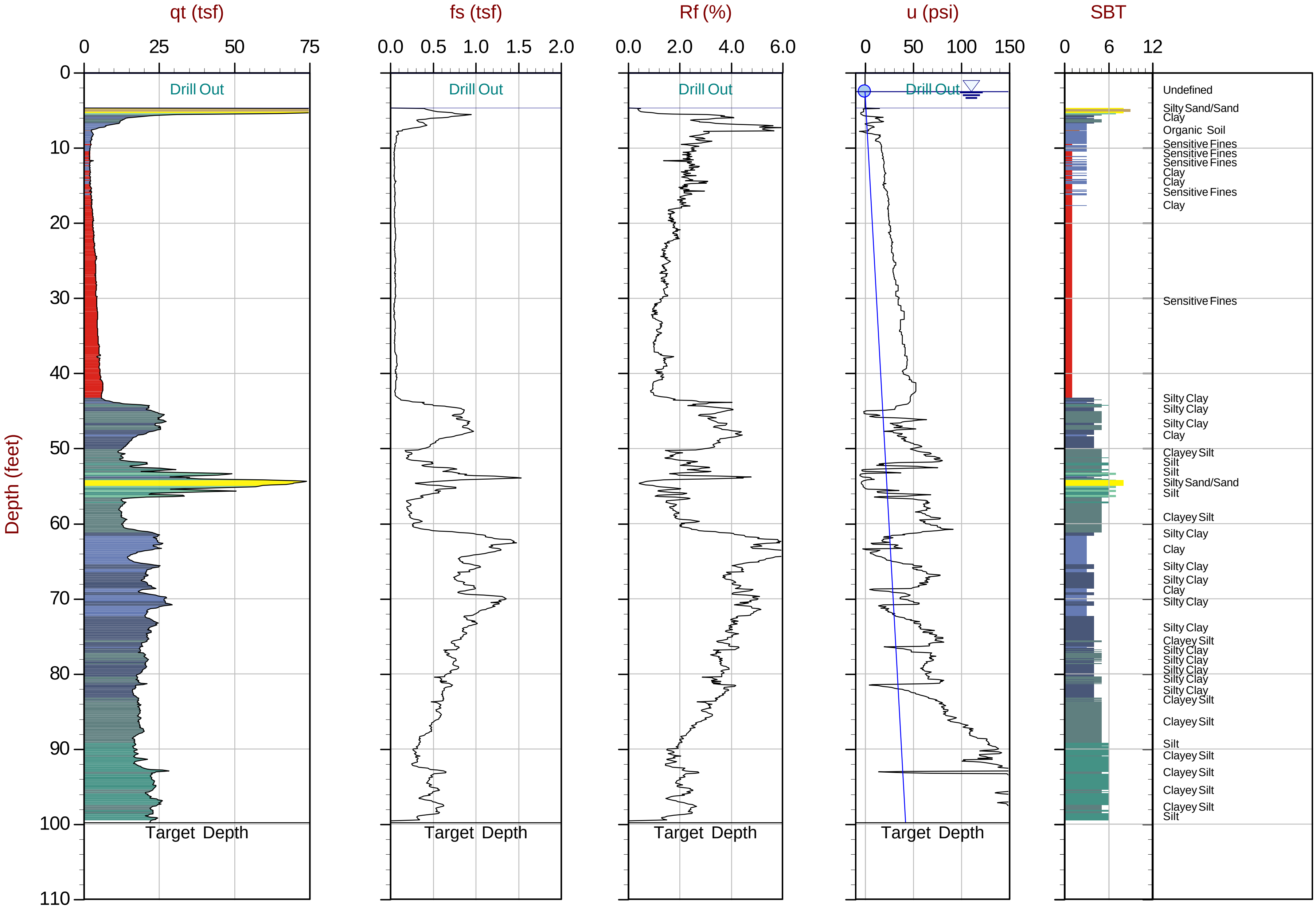
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cornerstone Earth Group

Job No: 16-56031
Date: 05:23:16 12:16
Site: 1050 Shell Blvd

Sounding: CPT-03
Cone: 447:T1500F15U500



Max Depth: 30.425 m / 99.82 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 16-56031_CP03.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM 10 N N: 4156088 E: 564941
Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

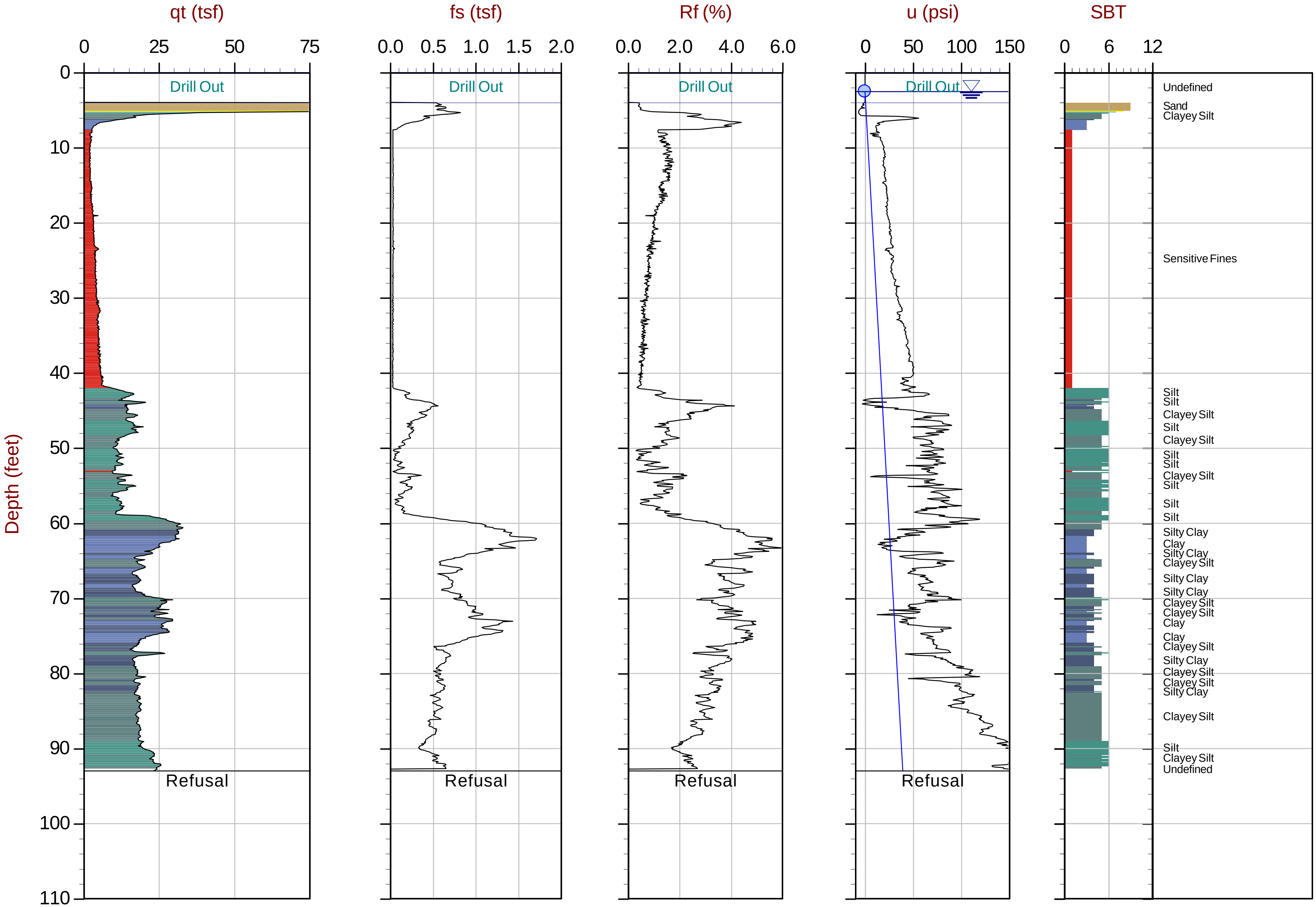
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cornerstone Earth Group

Job No: 16-56031
Date: 05:23:16 10:52
Site: 1050 Shell Blvd

Sounding: CPT-04
Cone: 447:T1500F15U500



Max Depth: 28.350 m / 93.01 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 16-56031_CP04.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM 10 N N: 4156113 E: 564933
Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

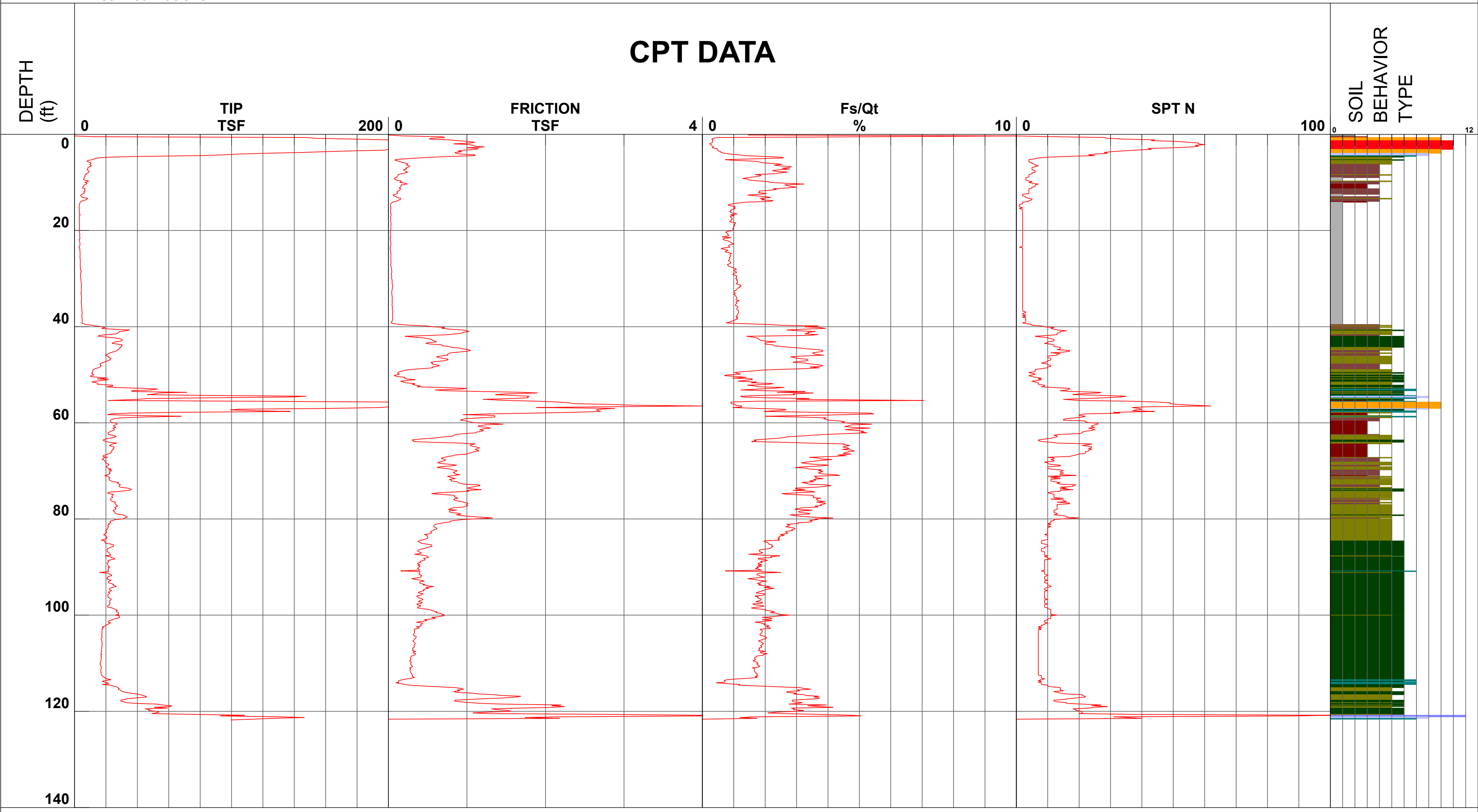
Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project Charter Square k-5 School
Job Number 826-2-1
Hole Number CPT-05
EST GW Depth During Test 11.00 ft
Operator JH SF
Cone Number DDG1281
Date and Time 5/25/2016 8:14:24 AM
Filename SDF(968).cpt
GPS
Maximum Depth 121.78 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 (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

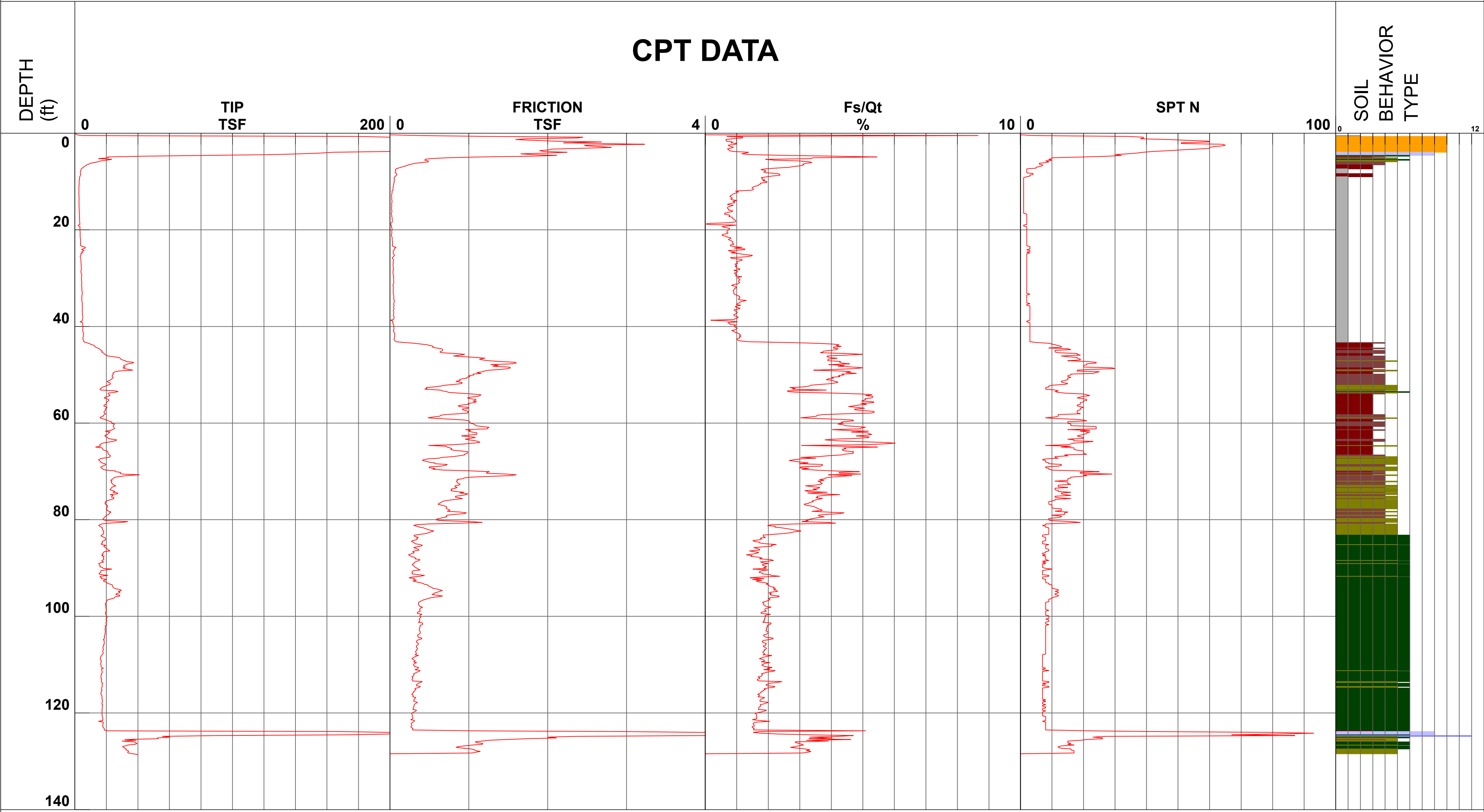
Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project	Charter Square k-5 School	Operator	JH SF	Filename	SDF(971).cpt
Job Number	826-2-1	Cone Number	DDG1281	GPS	
Hole Number	CPT-06	Date and Time	5/25/2016 3:43:47 PM	Maximum Depth	128.61 ft
EST GW Depth During Test		11.00 ft			

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
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■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
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■ 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 (*)

Cone Size 10cm squared

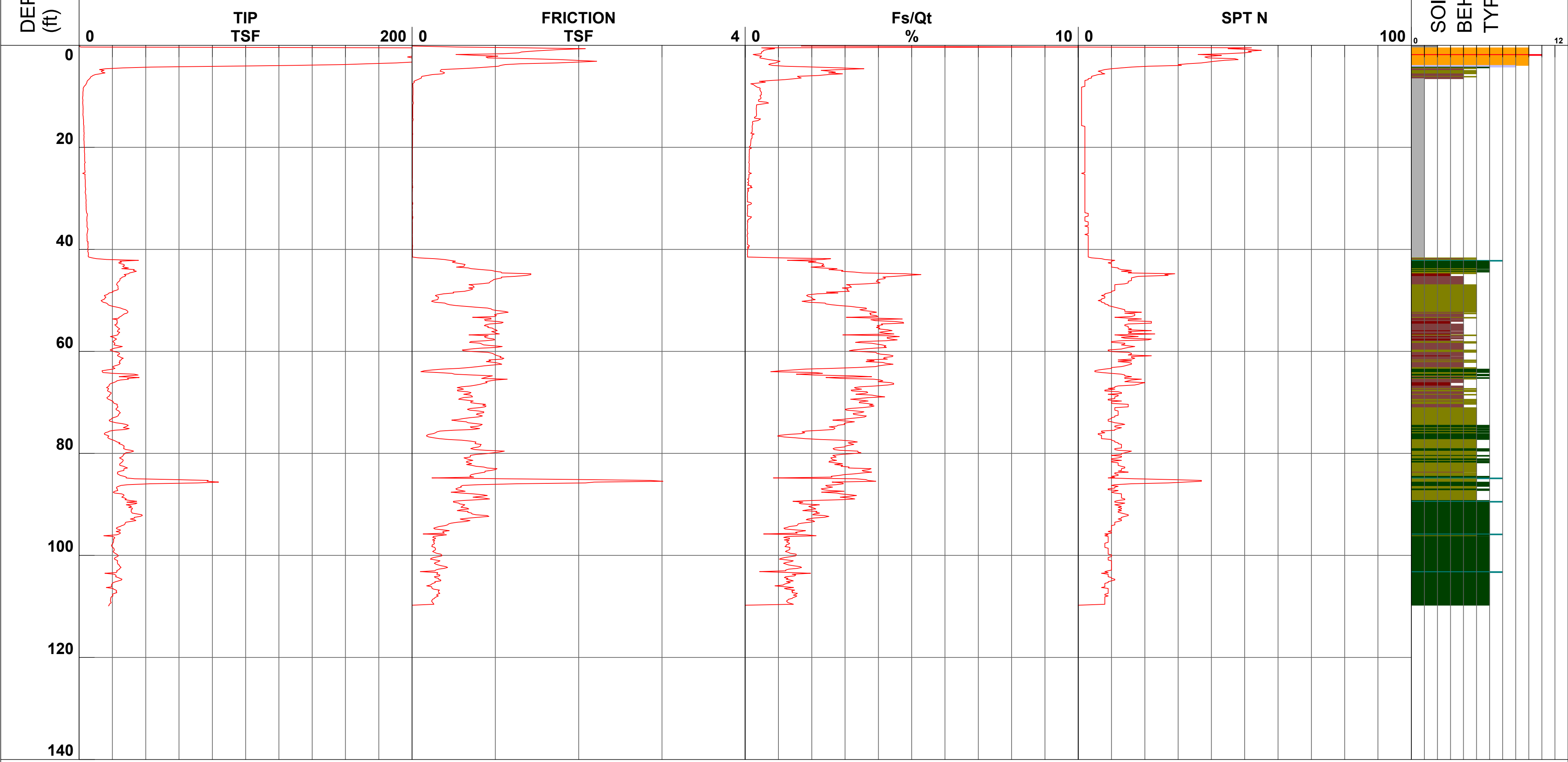
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Charter Square k-5 School	Operator	JH SF	Filename	SDF(969).cpt
Job Number	826-2-1	Cone Number	DDG1281	GPS	
Hole Number	CPT-07	Date and Time	5/25/2016 10:45:57 AM	Maximum Depth	109.91 ft
EST GW Depth During Test		11.00 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 (*)

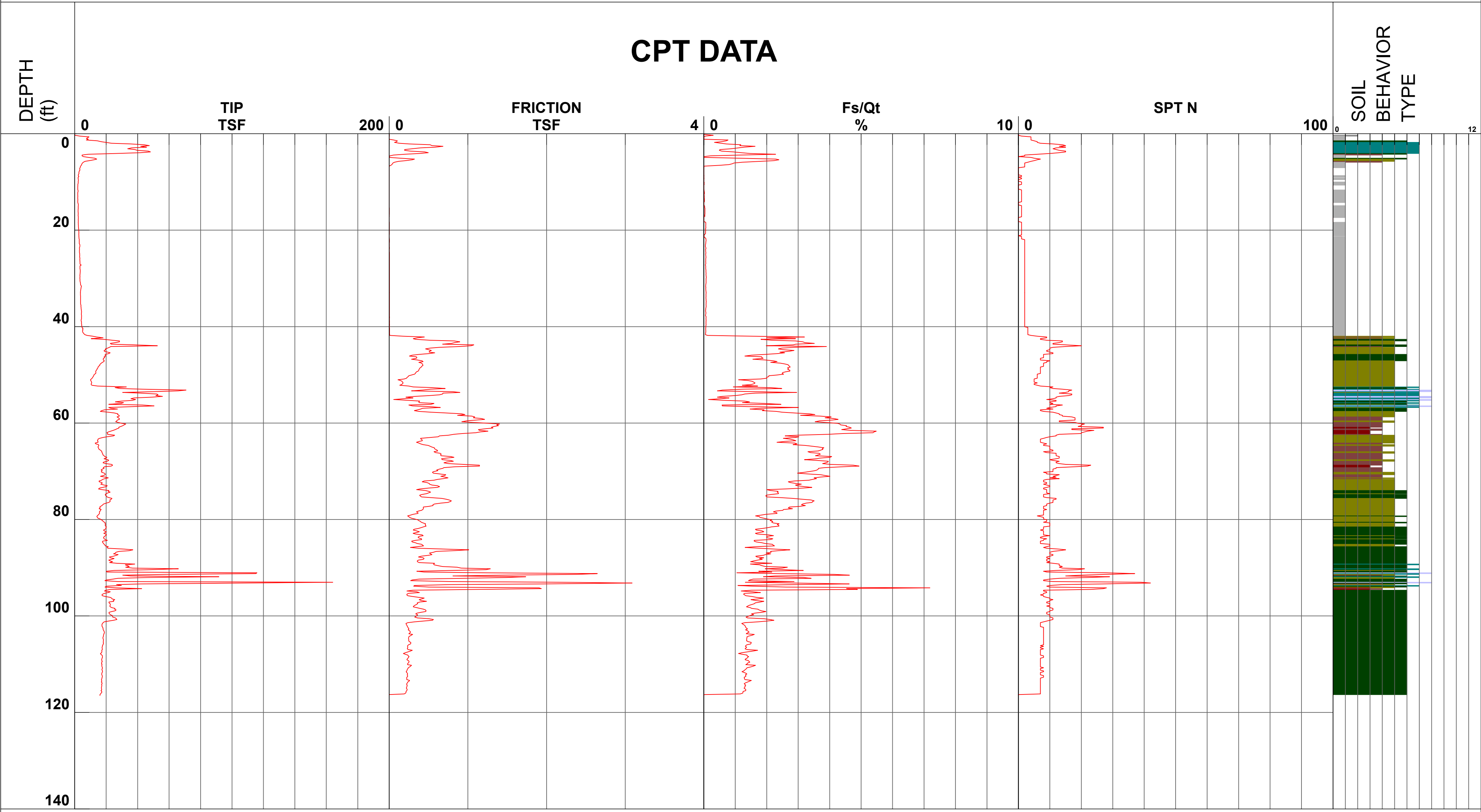
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project Charter Square k-5 School
Job Number 826-2-1
Hole Number CPT-08
EST GW Depth During Test 11.00 ft
Operator JH SF
Cone Number DDG1281
Date and Time 5/25/2016 1:53:55 PM
Filename SDF(970).cpt
GPS
Maximum Depth 116.47 ft

Net Area Ratio .8



■ 1 - sensitive fine grained
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■ 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 (*)

Cone Size 10cm squared

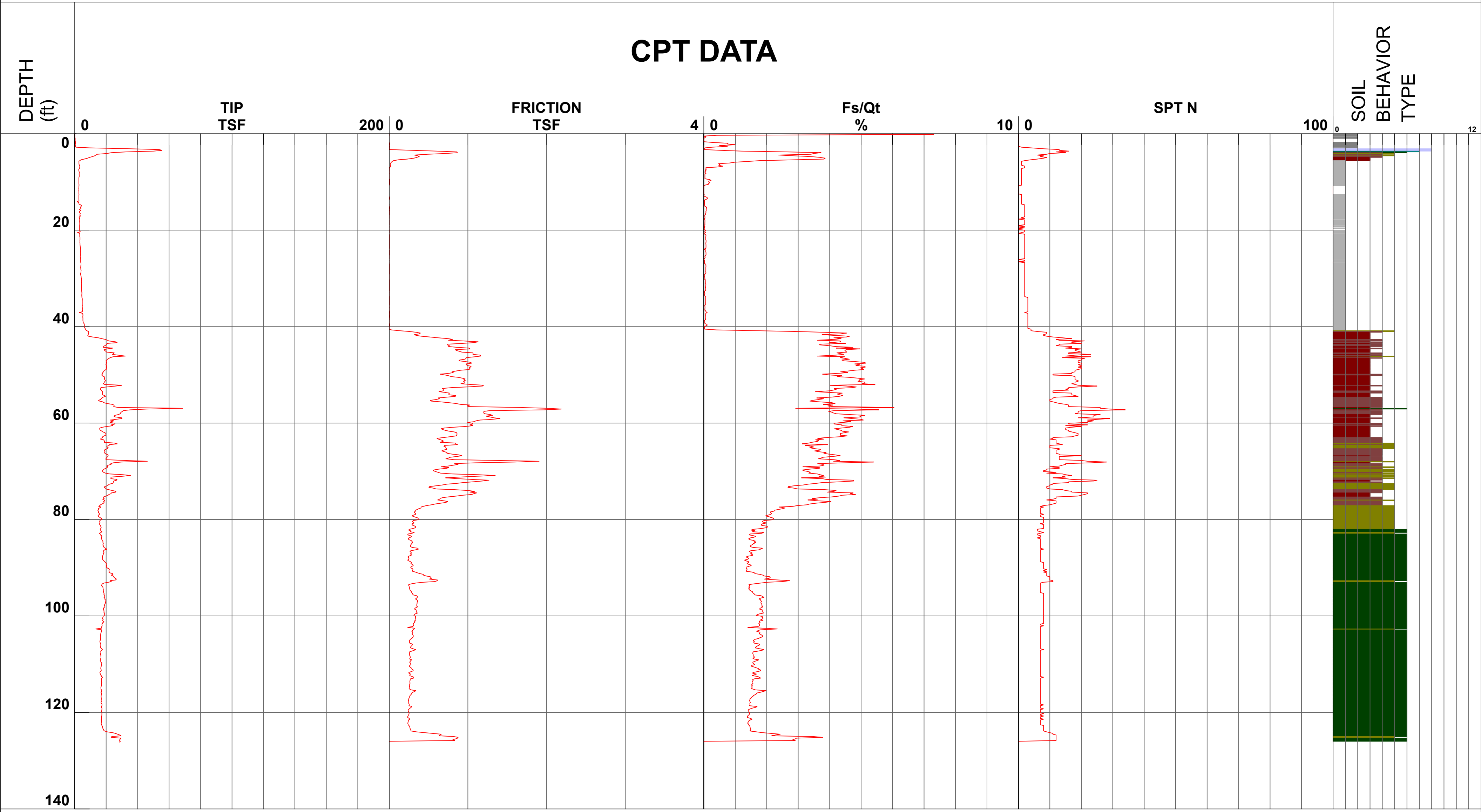
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Middle Earth Geo Testing Inc.

Project Charter Square k-5 School
Job Number 826-2-1
Hole Number CPT-09
EST GW Depth During Test 11.00 ft
Operator JH SF
Cone Number DDG1281
Date and Time 5/25/2016 5:57:06 PM
Filename SDF(972).cpt
GPS
Maximum Depth 126.15 ft

Net Area Ratio .8



- ■ 1 - sensitive fine grained
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- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
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- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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 44 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 42 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.

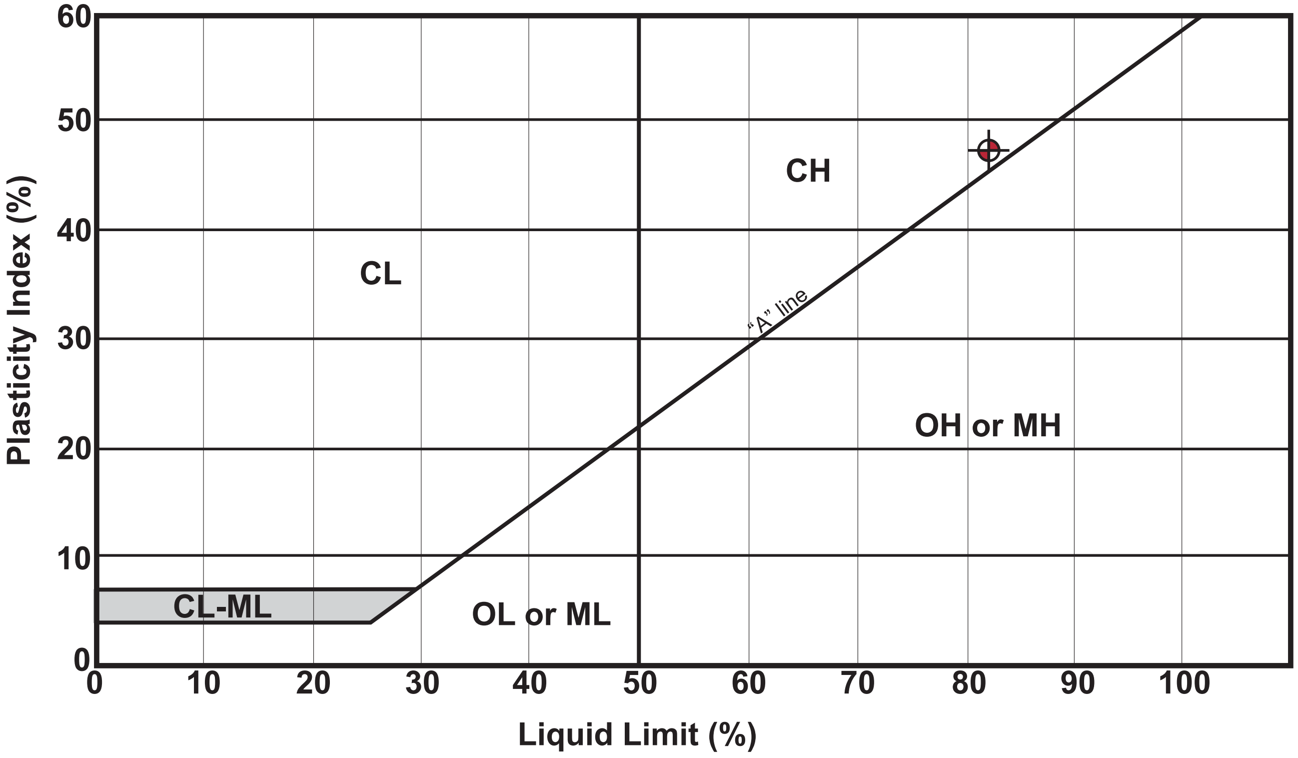
Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test is shown on the boring log at the appropriate sample depth.

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

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

Corrosion: Three soluble sulfate determinations (ASTM D4327), resistivity tests (ASTM G57), chloride determinations (ASTM D4327), and pH determinations (ASTM G51) were performed on three representative samples of the subsurface soils. Results of these tests are attached to 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-8	6.0	46	82	35	47	---	Fat Clay (CH) [Bay Mud Crust]

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Charter Square K-5 School
Foster City, CA

Project Number
826-2-1

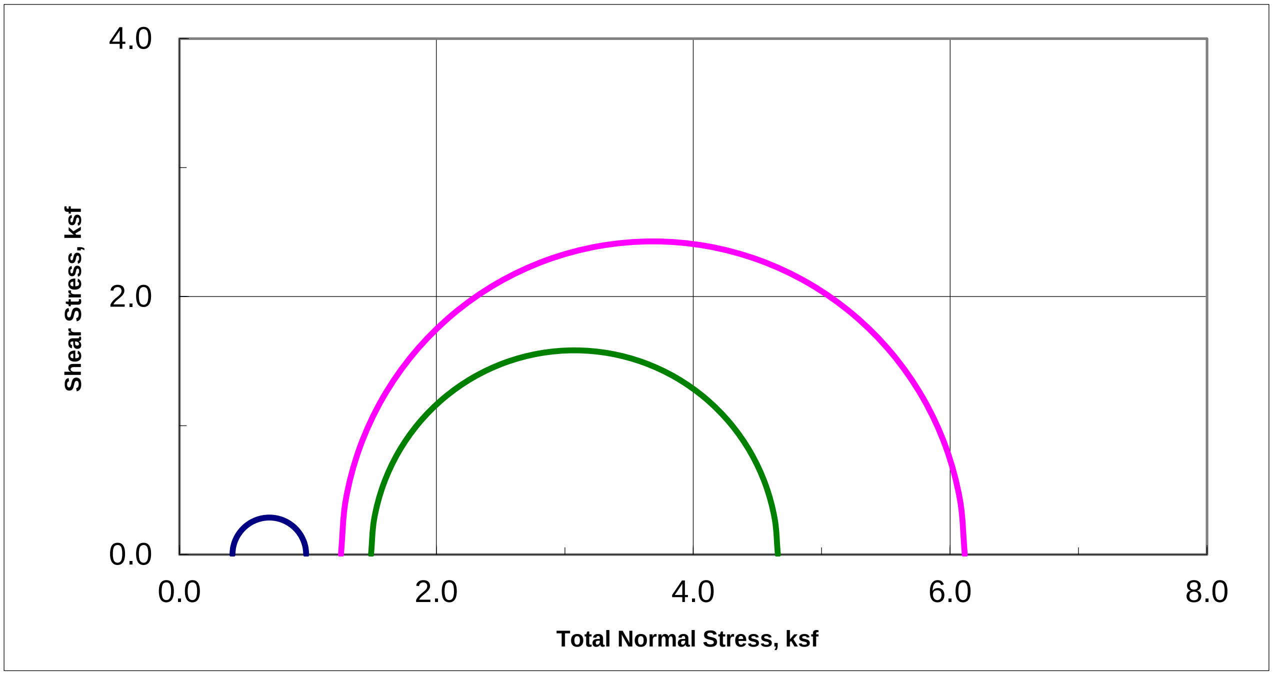
Figure Number
Figure B1

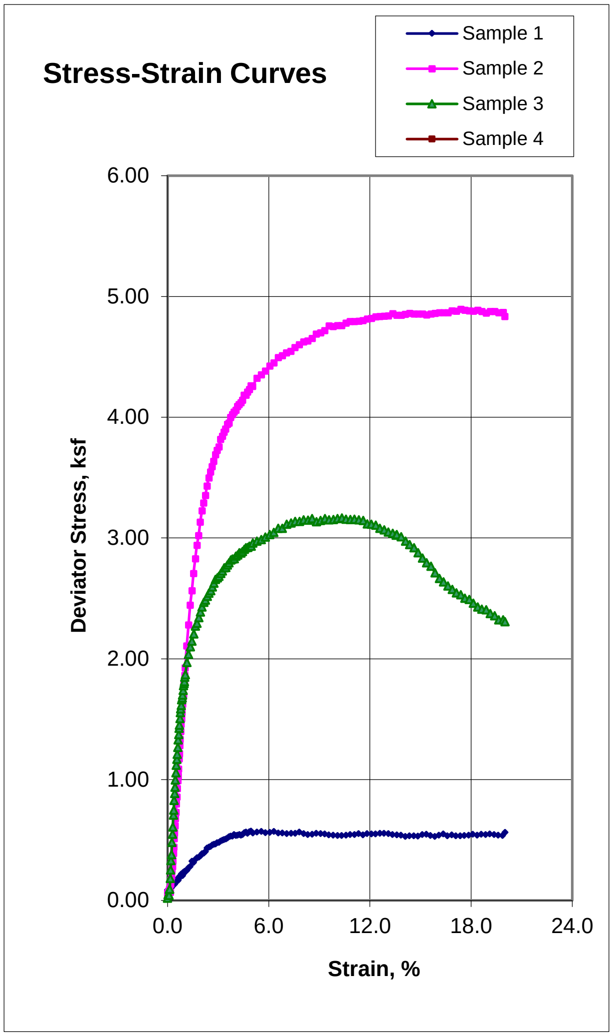
Date
June 2016

Drawn By
RRN

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	99.6	23.5	31.9	
Dry Den,pcf	45.0	102.9	90.4	
Void Ratio	2.749	0.637	0.865	
Saturation %	97.9	99.4	99.5	
Height in	6.04	5.20	5.19	
Diameter in	2.86	2.40	2.42	
Cell psi	2.9	8.7	10.4	
Strain %	4.94	15.00	10.33	
Deviator, ksf	0.574	4.856	3.167	
Rate %/min	1.00	1.00	1.00	
in/min	0.060	0.052	0.052	
Job No.:	**640-995**			
Client:	**Cornerstone Earth Group**			
Project:	**Charter Square K-5 School - 826-2-1**			
Boring:	EB-4	EB-8	EB-8	
Sample:	5	11C	13B	
Depth ft:	10(Tip-5")	43.0	49	

Visual Soil Description	
Sample #	
1	Greenish Gray CLAY (Bay Mud)
2	Gray Sandy CLAY
3	Olive CLAY
4	

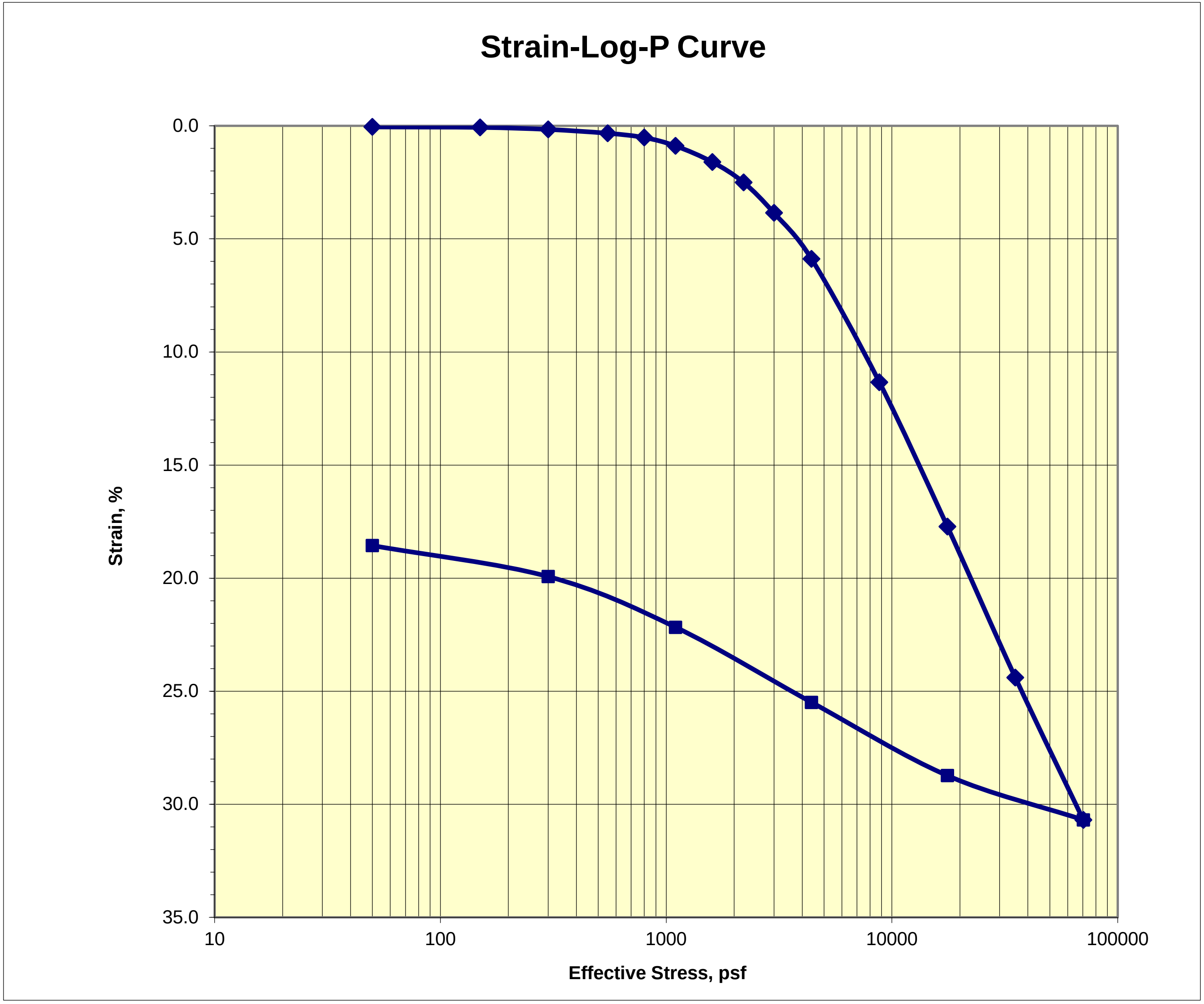
Remarks:

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.:	640-995	**Boring:**	EB-1	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	3	**Reduced:**	PJ
Project:	826-2-1	**Depth, ft.:**	4.5(Top-15")	**Checked:**	PJ/DC
Soil Type:	Dark Gray CLAY (Bay Mud)			**Date:**	6/21/2016



Strain-Log-P Curve

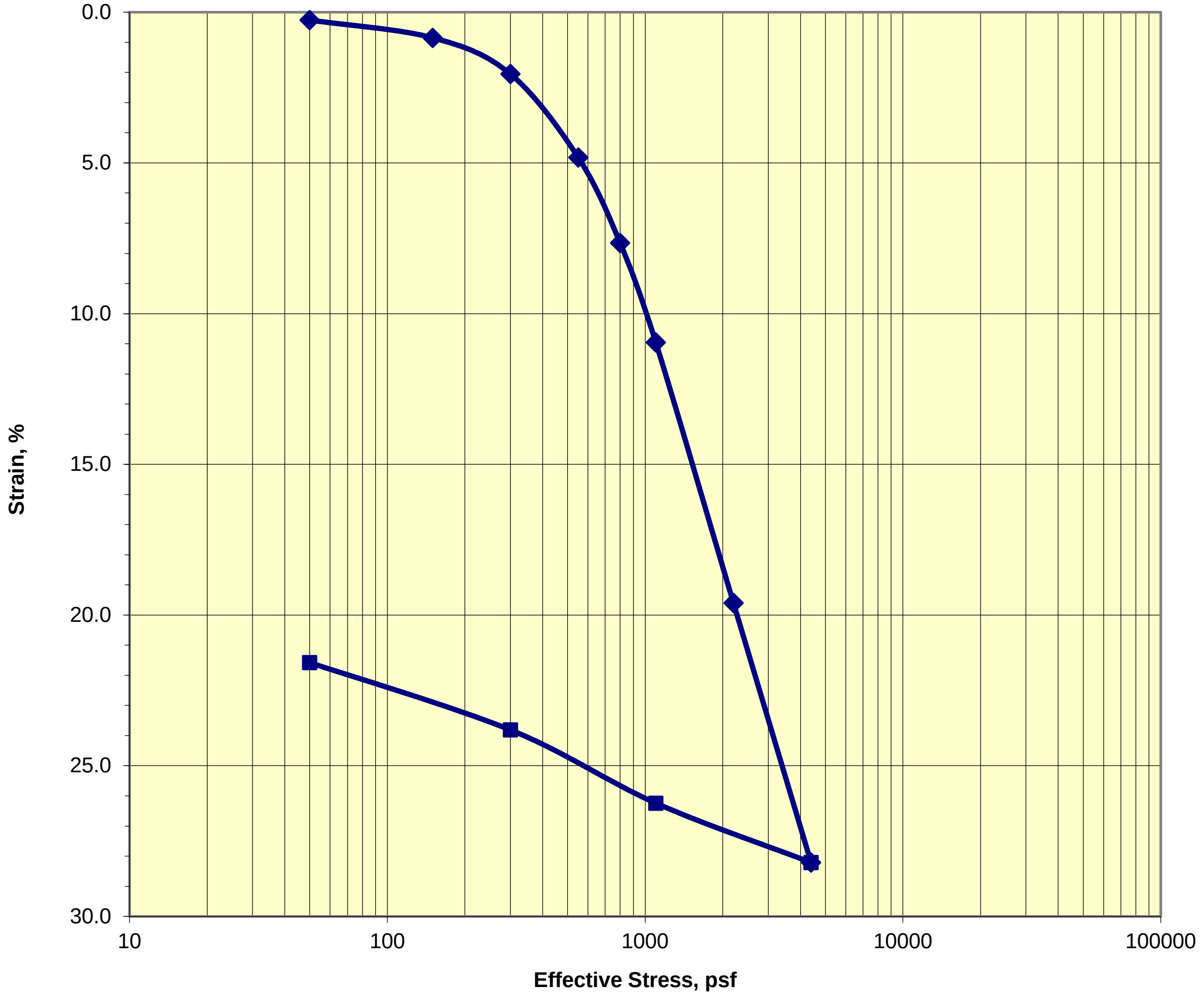
Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	54.0	41.4
Dry Density, pcf:	65.8	80.2
Void Ratio:	1.608	1.140
% Saturation:	92.4	100.0

Remarks:

Consolidation Test
ASTM D2435

Job No.:	640-995	**Boring:**	EB-4	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	5	**Reduced:**	PJ
Project:	826-2-1	**Depth, ft.:**	10(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY (Bay Mud)			**Date:**	6/14/2016

Strain-Log-P Curve



Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	102.6	72.7
Dry Density, pcf:	44.6	56.9
Void Ratio:	2.783	1.962
% Saturation:	99.6	100.0

Remarks:

Corrosivity Tests Summary

CTL # 640-995
Date: 6/10/2016
Tested By: PJ
Checked: PJ
Client: Cornerstone Earth Group
Project: Charter Square K-5 School
Proj. No: 826-2-1
Remarks:

Sample Location or ID			**Resistivity @ 15.5 °C (Ohm-cm)**			**Chloride**	**Sulfate**		**pH**	**ORP**		**Sulfide**	**Moisture**	**Soil Visual Description**
			As Rec.	**Min**	**Sat.**	**mg/kg**	**mg/kg**	**%**		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	**%**	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	4A	9.0	-	-	68	14,115	493	0.0493	8.0	-	-	-	82.6	Gray CLAY
EB-3	1A	1.5	-	-	5,658	32	39	0.0039	8.1	-	-	-	19.0	Gray SAND w/ shells
EB-8	12A	44.0	-	-	207	2,228	86	0.0086	8.1	-	-	-	18.9	Bluish Gray CLAY w/ Sand

APPENDIX C: CONSTRUCTION GUIDELINES ON BAY MUD

Constructing improvements on the Bay Margin presents difficulties throughout the Bay Area. These general guidelines are meant to provide a general understanding of some of the difficulties working in such an environment, where conditions likely include fill material, soft, saturated, weak clays, and shallow ground water. These general guidelines should be used as a supplement to the construction plans and specifications for the project.

GENERAL SOIL AND GROUND WATER CONDITIONS

As discussed in the geotechnical report, the entire site area was once a tidal marsh of the San Francisco Bay with a slough running through the site. Historically, the area was diked and cleared, and fills were placed across the site area. The fill in the area is generally about 2½ to 5½ feet in thickness and generally consists of medium dense silty sand.

The fill is underlain by soft, marine clays, known locally as Bay Mud. In general, the upper 1½ to 3½ feet is somewhat desiccated, and therefore stiffer than the underlying mud, and is often referred to as Bay Mud crust. Below the crust, the clays are saturated, soft, weak, and highly compressible. Moisture contents of the crust material are generally in the 39 to 54 percent range, and moisture contents for the underlying clays are generally in the 85 to 118 percent range.

The soft, compressible Bay Mud is generally underlain by older bay clays that are generally stiff to very stiff and of low compressibility.

Ground water is generally assumed to be near the top of Bay Mud; however, ground water does not typically appear quickly as free water, but does seep out of the mud slowly – often from more highly permeable seams of silts or fine sands. Since Bay Mud was deposited in a marine environment, the ground water will often be brackish. Ground water is seasonally also found to perch in the upper fill materials above the top of Bay Mud.

UNSTABLE SOIL CONDITIONS

During construction on Bay Mud sites, often due to regular construction traffic or compaction during grading, the surface soils or exposed subgrade become unstable. Bay Mud sites are particularly susceptible to instability because of the perched water frequently encountered at these sites, and the soft underlying clays. Instability is typically observed as significant deflection under loading, or rutting due to wheel or track loading. Often unstable soil conditions can be avoided by properly preparing the site for construction activities and/or winter conditions. These preparations often include the following items.

1. The site should be rough graded prior to inclement weather to drain surface water to retention or detention areas, or approved storm water treatment facilities. These approved areas for storm water disposal should be constructed in accordance with the project Storm Water Pollution Prevention Plan (SWPPP), if applicable to the project.

Positive surface grades should direct water to these facilities, and ponding on the site should be avoided.

2. Construction entrances and construction traffic areas should be prepared for the expected traffic with a sufficient construction roadway or aggregate base section to support the traffic without instability. Chemical treatment, stabilization fabric and rock, and temporary paving could potentially be used in these areas.

3. Unstable areas can be difficult to stabilize on Bay Mud sites, and likely required crushed rock and stabilization fabric or geogrid, or other approved methods.

HEAVY EQUIPMENT LIMITATIONS

As discussed, soft clays generally underlie the entire site. For this reason, construction equipment should be limited to medium to lightweight equipment to reduce the potential for instability, damage to shallow utilities, or slope failures. Instability is a significant issue on Bay Mud sites, and often leads to extra efforts to stabilize materials, dry out wet materials, and achieve compaction. The use of heavy equipment will greatly exacerbate this issue. We have the following general guidelines to aid in choosing the appropriate equipment for the site. Where lighter equipment can not be used, then extra care and support efforts will be required to traffic the heavy equipment across the site.

1. Avoid the use of heavy equipment on the site. This includes heavy vehicles – we generally recommend vehicles less than 15 tons – and vehicles with heavy point loads, such as forklifts or other types of lifts.

2. Where fill materials have been partially removed, the exposed soils will be even more susceptible to instability, and we recommend that lighter weight equipment be used in these situations. We do not recommend direct vehicles loads of any kind on exposed Bay Mud.

3. Moderate to heavy equipment should not come close to any excavation in Bay Mud, and should generally stay at least 3 to 4 times the height of the excavation from the edge of the excavation.

4. Traffic routes should be well-prepared and smooth to avoid bouncing of vehicles. Traffic speed should be kept low.

5. Shallow utilities should be located and protected from vehicle loading.

UTILITY CONSIDERATIONS

Bay Mud sites present several significant risks to shallow utilities. The most significant risk to shallow utilities on Bay Mud site is typically damage due to vehicle loading. Traffic loading can cause deflections and rutting, and can also significantly load a utility where shallow cover exists, damaging the utility. In addition, construction means, methods and materials that are not appropriate for Bay Mud sites, and do not take into account the site conditions, can also distress utilities. We suggest the follow guidelines be considered.

1. Shallow utilities should be protected from traffic loading and properly marked during construction. Protection might include slurry or lean concrete cover, trench plates, soil mounding, or other approved methods.
2. Construction equipment should not be allowed to traverse utilities where deflections or rutting is occurring, or subgrade soils are unstable. Properly designed access roads and utility protection should be implemented.
3. Distress to utilities can often not be discovered until much later; therefore, precautionary steps should be taken prior to allowing traffic to cross shallow utilities.
4. The backfill of utilities extending into Bay Mud may need to include lightweight backfill materials to limit backfill weights relative to the weight of material removed. Otherwise, settlement of the underlying Bay Mud may be induced causing sags in the utility.
5. Excavations in Bay Mud for utilities require shoring in most cases. We recommend against the use of trench shields unless special precautions are used during installation and extraction to limit deflections and deformation of the underlying Bay Mud. For example, dragging a trench shield along the trench would not be allowed. In addition, voids between excavation sidewalls and trench shields could allow lateral creep or sloughing of native soils.

OPEN EXCAVATIONS AND TRENCHES

Open excavations and trenches in Bay Mud require special precautions to prevent failures and potentially distress other improvements or cause significant delays and cost to the project. Contractors should carefully review the site conditions and preferably have experience in working on Bay Mud. We suggest the following guidelines be considered.

1. Trench excavations in Bay Mud or in fills overlying Bay Mud may be subject to failure and/or collapse due to the weak strength of Bay Mud. Equipment or stockpiles near excavations can also cause failures. All excavations and trenches should be properly shored or sloped back at an appropriate inclination.
2. Even shored excavations should be checked for potential failure mechanisms such as bottom heave prior to excavation and installation of shoring. The stability of all excavations and shoring should be the contractor’s responsibility.
3. Glory hole excavations and large v-trenching should not be backfilled with heavy import materials as detrimental settlement is likely to occur. Backfill materials should be similar in weight to the weight of materials removed.

4. Excavations extending into Bay Mud should have fill materials and Bay Mud segregated during excavation. Most contractors accomplish this by putting fill material on one side of the trench and Bay Mud on the other. Bay Mud is typically removed from the site because re-use would require a considerable amount of processing and drying to reach optimum moisture content for re-use as engineered fill.

5. Shallow trenches that extend into Bay Mud crust may remain open temporarily during utility installation – at the contractor's risk. However, trenches that extend into Bay Mud should be backfilled as soon as possible to prevent failures or instability of the sidewalls.

6. The contractor should completely review the geotechnical report and these guidelines in addition to the project plans and specifications to understand the difficulties of working on a Bay Mud site.

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 a 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.