

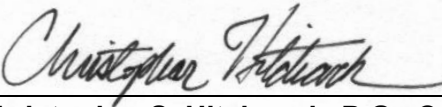
TYPE OF SERVICES	Geotechnical Investigation and Geologic Hazards Evaluation
PROJECT NAME	El Camino High School Modernization
LOCATION	1320 Mission Road South San Francisco, California
CLIENT	South San Francisco Unified School District
PROJECT NUMBER	1517-1-5
DATE	March 16, 2026



GEOTECHNICAL

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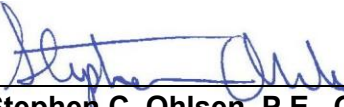

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of South San Francisco Unified School District for the El Camino High School Modernization project in South San Francisco, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A request for proposal (RFP) document titled, “Request of Proposal for Geotechnical Engineering and Geological Hazards Evaluation Services for El Camino High School-Modernization Project”, prepared by Kitchell, dated October 20, 2025.

1.1 PROJECT DESCRIPTION

The project site is located at 1320 Mission Road in South San Francisco, California and is currently occupied by El Camino High School campus. We have discussed the site with you, reviewed relatively recent aerial photographs of the site, and visited the site. We understand that the campus is undergoing a campus-wide modernization to improve site functionality, updated facilities, and address long-term infrastructure needs.

Based on our review of the RFP, dated October 20, 2025, the project includes improvements to the gymnasium and locker rooms to address structural deficiencies and ensure ADA compliance. The project scope also includes the addition of new exterior elevators to enhance accessibility, construction of new retaining walls to support the widening of the existing driveway for fire truck access, and replacement of the existing canopy structures with new canopies requiring new foundations.

Structural loads are not currently known for the proposed structure(s); however, structural loads are expected to be typical of similar-type structures. Grading plans are not available at this time; however, we assume grading will consist of cuts on the order of 1 to 5 feet for the widening of the existing driveways, and fills on the order of 1 to 3 feet for site improvements.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 28, 2025 (revised November 18, 2025) and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of six (6) borings drilled between December 30, 2025 and January 5, 2026 with a combination of truck-mounted hollow-stem auger and limited-access track-mounted hollow stem auger drilling equipment, and four (4) Cone Penetration Tests (CPTs) advanced on December 29, 2025. The borings that were drilled with the truck-mounted hollow-stem auger, Borings EB-4, EB-5, EB-6, and EB-7, were drilled to depths of approximately 22½ to 39½ feet. The borings that were drilled with the track-mounted hollow-stem auger, Borings EB-8 and EB-9, were drilled to depths of approximately 25 feet. CPTs were advanced to depths of 22½ to 59½ feet before practical CPT refusal was encountered within all four CPTs. Seismic shear wave velocity measurements were collected from CPT-1, CPT-2, CPT-3A, CPT-4. During our CPT explorations, CPT-3 was advanced without collecting seismic shear wave velocity measurements. CPT-3 was re-advanced as CPT-3A, and seismic shear velocity measurements were collected until practical refusal at a depth of 59½ feet. This report refers to CPT-3A as CPT-3 in the latter sections and within the figures of this report. The original CPT-3 data was not used in the report. Boring EB-7 was advanced adjacent to CPT-1 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.

As a supplement to our drilling and CPT program, four (4) infiltration tests were performed using traditional hand sampling auger equipment and the Guelph Infiltrometer to depths of 1¾ to 2 feet.

The approximate locations of our exploratory borings, CPTs, and infiltration tests are shown on the Site Plan and Geologic Map, Figure 2. Refer to Appendix A for details regarding our field program.

1.4 PREVIOUS FIELD EXPLORATION

Previous field exploration was performed by Geosphere in 2012 consisting of four (4) borings using truck-mounted hollow stem auger drilling equipment. Previous field exploration was performed by Cornerstone Earth Group in 2024 as part of the approved Central Kitchen and Cafeteria project, which consisted of, three (3) borings using truck-mounted hollow stem auger drilling equipment. The approximate locations of the previous explorations are also shown on the Site Plan and Geologic Map, Figure 2. Refer to Appendix C for copies of the previous exploration logs.

1.5 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, washed sieve analyses, Plasticity Index tests, an R-value test, and corrosion tests. Refer to Appendix B for details regarding our laboratory program.

1.6 CORROSION EVALUATION

Three (3) samples from our borings from depths ranging from 2 to 6 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. In general, the on-site soils can be characterized as mildly to moderately corrosive to buried metal, and non-corrosive to buried concrete. Refer to Section 3 for further information.

1.7 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including a soil quality evaluation, environmental findings and conclusions are provided under separate covers.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGIC SETTING

2.1.1 Regional Geologic Setting

The San Francisco Peninsula is situated at the northern end of the Santa Cruz Mountains separating the Pacific Ocean from San Francisco Bay. The peninsula 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. Younger sedimentary and volcanic units locally cap these basement rocks. Still younger surficial deposits that reflect geologic conditions of the last million years or so cover most of the Coast Ranges.

Movement on the many splays of the San Andreas fault system has produced the dominant northwest-oriented structural and topographic trends 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 extends from the Salton Sea to Cape Mendocino and is about 40 miles wide in the Bay Area, as shown on the Regional Fault Map, Figure 4. The San Andreas fault is the dominant structure in the fault system, capable of producing large earthquakes.

Late Cenozoic uplift of the northern Santa Cruz mountains has occurred, in part, along a series of northwest-striking reverse faults, known as the 'Foothills thrust belt' (Bürgmann et al., 1994), bordering the northeastern margin of the range-front. Bounded by the main trace of the San

Andreas fault to the west, this sequence of southwest-dipping thrusts, associated with a restraining left-bend in the San Andreas fault, has been responsible for the uplift of the Santa Cruz Mountains. While the San Andreas Fault is the dominant structure in this system of faults and is capable of producing the largest earthquakes, many other faults in the San Andreas system are active and capable of producing nearly as large earthquakes. The Serra fault is the northernmost fault in the Foothills thrust belt, extending approximately 12 miles (20 km) north from Hillsborough to Fort Funston, and located approximately 1 to 3 miles northeast of the San Andreas Fault in the vicinity of the school site (Hengesh et al., 2004).

2.1.2 Local Geology

Surficial deposits overlying the Franciscan bedrock in the South San Francisco area include unconsolidated late Pleistocene to Holocene age deposits which are identified as the Merced Formation (map symbol QTm), Colma Formation (Qc), and Alluvium (Qal), as shown on the Vicinity Geologic Map, Figure 3. These units were mapped and described as being deposited on the earlier, irregular topographic surface of Franciscan Complex rocks (Bonilla, 1971) or older Pleistocene deposits. Artificial fill (Qaf) is present locally within the general area.

Underlying these relatively geologically young formations are rocks of the Franciscan Complex of Jurassic or Cretaceous age. The Franciscan Complex is a heterogeneous sequence of rocks deposited in an island arc system along an older subducting plate boundary and consist of graywacke sandstone, altered volcanic rocks, chert, serpentinite and metamorphosed versions of each of these bedrock units. Exposures of Franciscan Complex bedrock are not present in the vicinity of the project nor were they encountered in our borings (See Section 3.4).

Various published regional geologic maps show the campus underlain by the Colma Formation, which is described as "Pleistocene marine sedimentary rocks composed of sand, silt and clay." (Bonilla 1971, 1998; Lajoie et al., 1974; and Brabb and Pampeyan, 1983; Brabb et al., 1998, USGS. The Colma Formation originated in shallow marine, estuarine, and alluvial environments.

The San Francisco Peninsula segment of the San Andreas fault is located approximately 2.1 miles to the southwest of campus as shown on the Regional Fault Map, Figure 4. The late Quaternary Serra fault is located approximately 0.65 miles southwest of the campus. The inactive Hillside fault is located approximately 0.5 miles northeast of the campus. The City College fault zone, although also not considered active, is located 2.5 miles northeast of the project site.

2.2 REGIONAL SEISMICITY

2.2.1 Regional Faults

The San Francisco Bay region is characterized by active, potentially active and older bedrock faults with a historical record of large and damaging earthquakes. Active faults of the San Andreas fault system, including the San Andreas, Hayward, Rodgers Creek, and Calaveras faults, could produce significant earthquakes during the life of the project. In addition,

potentially active compressional structure, including reverse and thrust faults; (Foothill and Serra) found along the foothills of the Santa Cruz Mountains may impact the project.

Figure 4 (Regional Fault Map) shows the location of significant faults in the vicinity of the project. Table 1, *Seismic Source Parameters for Major Faults*, summarizes source parameters for the major faults. These parameters are based on the Working Group for California Earthquake Probabilities (WGCEP, 1999, 2003, 2008 and 2015 [Field et al]), and CGS (2003).

Table 1. Seismic Source Parameters for Major Faults

Fault	Mean Moment Magnitude	Slip Rate (mm/yr)	Mean Recurrence Interval (yrs)
Calaveras	5.8 to 6.9	6+2 to 15+3	54 to 187
Concord – Green Valley	6.0 to 6.7	4+2 to 5+3	201 to 219
Greenville	6.6 to 6.9	2+1	644 to 623
Hayward –Rodgers Creek	6.5 to 7.3	9+2	155 to 205
San Andreas	7.0 to 7.9	17+4	223 to 229
San Gregorio	6.9 to 7.4	3+2 to 7+3	392 to 540

The most significant active and potentially active (Late Quaternary and Holocene) faults within about 50 miles (80 kilometers or km) of the project are described below:

2.2.1.1 San Andreas Fault

The approximately 690 mile-long (1,100 km) San Andreas fault zone, is the principal element of the San Andreas fault system—a network of faults with predominantly dextral (right-lateral) strike-slip displacement that collectively accommodates the majority of relative motion between the North American and Pacific plates. The San Andreas fault is the longest active fault in California. Movement on the San Andreas fault is right-lateral strike-slip, with an estimated total offset of some 560 km (350 miles).

Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault can be divided into a number of fault segments (WGCEP, 1999; CGS, 2003). Each of those segments is capable of rupturing either independently or in conjunction with adjacent segments. In the San Francisco Bay area, the San Andreas fault is defined by the North Coast, Peninsula, and Santa Cruz Mountains segments, all of which ruptured in the 1906 San Francisco earthquake. Significant aspects of these three segments are described below:

North Coast Segment - The North Coast segment extends 111 miles (178 km) from Point Arena southeast to the Golden Gate (WGCEP, 2003). The southern boundary with the Peninsula segment is characterized by a 1.9-mile (3-km)-wide, right releasing bend, splaying off of the San Gregorio fault to the south, a reduced slip rate to the south, and a drop in geodetically modeled

slip associated with the 1906 earthquake.

Correlation of several studies indicates that the most recent paleoevent may have occurred between 1600 and 1650, although wider earthquake windows have been suggested. Recurrence intervals for the past 2,000 years range from 200 to 400 years. Reported Holocene slip rates range from a minimum of 16-18 mm/year to 25-26 mm/year. CGS (2003) combines this segment with the Shelter Cove segment to the north, resulting in a total fault segment length of approximately 200 miles (320 km), and assigns an average slip rate of 24 ± 3 mm/year, and a calculated maximum Moment Magnitude of 7.6.

Peninsula Segment - The Peninsula segment extends approximately 55 miles (88 km) from offshore the Golden Gate southeast to the vicinity of Black Mountain near Los Altos Hills. The Peninsula segment is delineated by well-defined geomorphic features characteristic of right-lateral strike-slip faulting, including deflected drainages, linear drainages, side hill benches, closed depressions (sag ponds), aligned benches, linear scarps, linear troughs and ridges, aligned saddles, and linear vegetation contrasts.

The most recent surface fault event in the Peninsula segment is the 1906 San Francisco earthquake, and the most recent paleoevent may be an earthquake that occurred in 1838, although direct evidence has not yet been determined. The reported dextral offset associated with the 1906 earthquake at the Filoli investigation site (Hall et al., 1999) site was about 8 feet (2.5 meters), with approximately 5.2 feet (1.6 meters) of offset representing a penultimate (possibly 1838) event. The timing of earlier events for the Peninsula segment has not been determined. The most recent paleoseismic studies of the Peninsula segment indicate that the Holocene slip rate is on the order of 17 to 19 mm/year (Hall et al., 1999). CGS (2003) assigns an average slip rate of 17 ± 3 mm/year along this segment, and a calculated maximum Moment Magnitude of 7.1.

Santa Cruz Mountains Segment - The Santa Cruz Mountains segment extends approximately 37 miles (57 km) from the vicinity of Black Mountain in the Santa Cruz Mountains southeast to just south of San Juan Bautista. The northern boundary of this segment is marked by an approximately 0.6-mile(1-km)-wide, left-contractional bend near Black Mountain. The southern boundary and transition to the adjoining Creeping segment to the south is taken as the approximate southern termination of surface fault rupture associated with the 1906 San Francisco earthquake (Lawson, 1908).

While the most recent large earthquake of this segment is the 1989 Mw6.9 Loma Prieta earthquake, the most recent paleoearthquake along the Santa Cruz Mountains segment may have occurred in the mid-1600s, based on interpretation of trench excavations by Schwartz et al. (1998). He reported that 247 to 266 years elapsed between the 1906 earthquake and the most recent paleoevent in the mid-1600s. Fumal et al. (1999) identified as many as seven large earthquakes since 1100 A. D. CGS (2003) separates this segment into two segments (i.e., "Santa Cruz Mountains" and "Pajaro" segments), and assigns an average slip rate of 14 ± 3 mm/year to both, with a calculated maximum Moment Magnitude of 7.0.

2.2.1.2 Hayward Fault

The Hayward fault is a right-lateral strike-slip fault, and one of the major structures of the San Andreas fault system in the San Francisco Bay region. The Hayward fault extends from San Pablo Bay southeastward to at least Warm Springs in Fremont, for a distance of approximately 43 miles (69 km). Recent studies indicate that the fault likely extends for another 11 miles (17 km), with a southern terminus near Milpitas and a total fault length of 54 miles (86 km). Further to the southeast, the fault appears to transfer slip, partly in a series of step-over structures, to the Calaveras fault.

The Hayward fault has been subdivided into two segments by the WGCEP (2003) based on the rupture associated with the most recent large earthquake (in 1868). Various estimates indicate that the 1868 event (M 6.9) ruptured the southern segment for a distance ranging from 19 to 34 miles (30 to 54 km). The boundary between the northern and southern segments is poorly constrained due to uncertainty in the northern extent of the 1868 rupture. The current segment boundary is a point between Montclair and Mills College (Oakland) with an uncertainty of ± 6 miles (WGCEP, 2003).

The average recurrence interval for large earthquakes on the southern segment of the Hayward fault is between 141 to 199 years (mean + 2 sigma deviation). The timing of the penultimate event on the northern segment of the Hayward fault is not well constrained, but there have been at least 4 to 7 surface faulting events in the past 1627 radiocarbon years (WGCEP, 1999), indicating a recurrence interval of less than 270 to a maximum of 710 years over the past 2,000 years. CGS (2003) assigns an average slip rate of 9 ± 1 mm/year to both segments, with a calculated maximum Moment Magnitude of 7.1 for the entire fault.

The Hayward fault is creeping, although the rate of creep varies locally along the fault. The average creep rate is 3.8 to 5.1 mm/year, with a maximum of about 9 mm/year near the southern end in Fremont (Lienkaemper and Galehouse, 1997). The hills to the east of the fault are rising at a rate of 1.5 mm/year, indicating a reverse slip component on the fault.

2.2.1.3 Rodgers Creek Fault

The Rodgers Creek fault is the northern extension of the Hayward fault. The length of the Rodgers Creek fault is estimated to be 39 miles (63 km). The fault is expressed by well-defined geomorphic features characteristic of right-lateral strike-slip faulting, including deflected drainages, linear troughs, side hill benches, closed depressions (sag ponds), aligned benches, and linear ridges. The recurrence interval for large magnitude events is estimated to be between 130 to 370 years, with the most recent paleoevent probably occurring sometime between 1690 and 1745. CGS (2003) assigns an average slip rate of 9 ± 2 mm/year to the fault segment, with a calculated maximum Moment Magnitude of 7.0.

Recent imaging of San Pablo Bay sediments provides evidence that the Hayward and Rodgers Creek faults are connected at the surface, thus indicating that simultaneous rupture of the two faults is possible (Watt et al, 2016). An earthquake resulting from simultaneous rupture is estimated to have a magnitude of $M7.4$.

2.2.1.4 San Gregorio Fault

The San Gregorio fault is an active, structurally complex transpressional fault zone up to 5 km in width. The fault is located mainly offshore of the California coast, although it crosses promontories such as Pillar Point, Point Ano Nuevo, and Moss Beach. Segmentation is poorly constrained due to the presence of multiple fault traces, wide zone of associated deformation, and generally poor exposures. However, WGNCEP (1996) model the fault zone with two sections. The northern fault segment extends for about 80 miles (130 km) from an intersection with the San Andreas fault near Bolinas Lagoon south to the Point Sur region. A southern segment, identified as the Sur and Palo Colorado fault zones, extends approximately 50 miles (80 km) from Point Sur south to a connection with the Hosgri fault in the San Simeon area. The most recent rupturing event on the northern segment, based on studies of the Seal Cove fault, occurred between about 1300 and 1775 (Simpson, et al., 1997). The penultimate event probably occurred between 620 and 1400, thus indicating a recurrence interval between about 400 and 1,000 years. CGS (2003) assigns an average slip rate of 5 ± 2 mm/year to the northern segment of the San Gregorio fault, with a calculated maximum Moment Magnitude of 7.3.

2.2.1.5 City College Fault

The City College fault zone, although not included in Table 1 and not considered an active fault, is situated approximately 2½ miles northeast of the project site. Bonilla (1971, 1998) and Schlocker (1974) identified northwest-striking shear zones within bedrock as forming the City College fault, or City College shear zone, and the Fort Point–Potrero Hill–Hunters Point shear zone. Schlocker (1974) describes sheared and shattered Franciscan Complex rocks along the trend of the City College shear zone, with abundant slickensides and gouge zones exposed in the shear zone. However, available geologic data suggests that neither shear zone has been active in the late Quaternary and there is no historic seismicity associated with these zones.

Other older faults shown on regional maps include the Hillside and San Bruno faults. The San Bruno fault shown on Bonilla (1965) has since been removed from government maps based on work by the USGS (1997). The Hillside fault has been mapped as striking in a northwesterly direction approximately ½ mile northeast of the site. Bonilla (1965) shows the Hillside fault as a discontinuous bedrock fault that offsets Franciscan rocks but is largely covered by unfaulted Quaternary age deposits. The Hillside fault is not considered seismogenic (capable of generating an earthquake) but may be subject to triggered (co-seismic) secondary displacement during large earthquake events on the nearby San Andreas Fault (Marlow, 1994).

2.2.2 Seismicity

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (UCERF2, 2008) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such

events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7% for UCERF2 to about 7.0% for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036. During such an earthquake the danger of fault ground rupture at the site is slight, but strong ground shaking would occur.

The San Francisco Bay area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. Significant earthquakes occurring in the Bay Area are generally associated with crustal movement along well defined, active, fault zones of the San Andreas fault system (see Figure 4). The San Andreas fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989 and passes approximately 2 miles southwest of the campus, according to the California Department of Conservation Division of Mines and Geology (CDMG, 1998). Another major active fault in the Bay area is the Hayward fault, located about 15½ miles northeast of the site. Table 2 lists all known active faults in order of increasing distance within 100 kilometers (62 miles) of the project site.

We reviewed the USGS Quaternary Fault and Fold Database to generate the fault distances. The seismic characteristics of some faults vary along their lengths so different fault segments of the same fault could be listed separately in the table. It should be noted that the fault distances shown in Table 2 were generated using USGS Quaternary Fault and Fold Database and fault models, which differ from the mapped active fault near-source zones (CDMG, 1998). We have also included the faults considered significant in the USGS Hazard Toolbox Disaggregation (Table 3), using the National Seismic Hazard Model (NSHM) 2018 model and the project coordinates. Following Section 21.5.2 of ASCE 7-22, scenario earthquakes contributing less than 10% of the largest contributor were ignored.

Table 2 Approximate Fault Distances

Fault Name	Distance (miles)	Distance (kilometers)
Serra Fault	2.3	1.4
Northern San Andreas	5.0	3.1
San Gregorio	19.0	11.8
Hayward – Rodgers Creek	42.5	26.4
Monte-Vista Shannon	60.5	37.6
Calaveras	65.8	40.9
Concord	78.7	48.9
Rodgers Creek Fault	86.9	54
Green Valley	87.2	54.2
Greenville	90.7	56.4
West Napa	91.6	56.9
Las Positas fault	96.4	59.9
Butano	115.4	71.7
Sargent	118.4	73.6
Midway	122.3	76
Zayante-Vergeles	139.5	86.7
Monterey Bay- Tularcitos	140.8	87.5
San Joaquin	143.7	89.3

Table 3: Approximate Fault Distances (USGS Hazard Toolbox Disaggregation)

Fault Name	Distance (miles)	Distance (kilometers)	Magnitude
San Andreas (Peninsula)	2.0	3.2	7.8

2.3 HISTORICAL EARTHQUAKES

We reviewed and performed a data search of known historical earthquakes of magnitude 5 or greater within a 100-kilometer radius of the site using available published data from the California Division of Mines and Geology (CDMG) computerized earthquake catalog of events through January 2024. Figure 5, Historical Earthquake Map, shows the epicenters of magnitude 5 or greater events. We also included data from Townley and Allen (1939) and the U.S. Geological Survey Earthquake Data Base System, giving 200 years of data in the search area. The results of our computer search indicated that about 57 known earthquakes of Richter Magnitude 5 or greater have occurred within 100 kilometers of the site between 1800 and January 2024.

Four relatively large magnitude earthquakes have occurred in the region during the above noted time period, including the Loma Prieta Earthquake of 1989 that was centered approximately 55 miles southeast of the site.

Table 4: Magnitude 5 or Larger Earthquakes within 100 km

Date	Magnitude (Mw)	Date	Magnitude (Mw)
10/25/2022	5.1	6/2/1899	5.6
8/24/2014	6.0	3/31/1898	6.4
10/31/2007	5.5	8/9/1893	5.6
10/18/1989	5.1	4/30/1892	5.6
10/18/1989	6.9	4/19/1892	6.6
8/8/1989	5.4	10/12/1891	5.8
6/27/1988	5.3	1/2/1891	5.8
6/13/1988	5.3	7/31/1889	5.6
3/31/1986	5.7	5/19/1889	6.0
4/24/1984	6.2	3/26/1882	5.9
1/27/1980	5.4	6/27/1882	5.8
1/24/1980	5.1	4/2/1870	5.8
1/24/1980	5.8	2/17/1870	5.9
10/2/1969	5.3	10/21/1868	6.8
10/2/1969	5.3	7/15/1866	6.0
12/18/1967	5.3	3/26/1866	5.8
11/16/1964	5.2	10/8/1865	6.5
3/22/1957	5.3	5/24/1865	5.9
9/5/1955	5.8	5/21/1864	5.8
12/13/1933	5.0	3/5/1864	6.0
2/15/1927	5.0	2/26/1864	6.1
10/24/1926	5.5	7/4/1861	5.8
10/22/1926	6.3	11/26/1858	6.2
10/22/1926	6.4	2/15/1856	5.9
8/8/1916	5.0	1/2/1856	5.7
7/1/1911	6.5	8/27/1855	5.5
4/18/1906	7.9	5/15/1851	5.0
8/3/1903	6.2	6/25/1838	7.4
6/11/1903	6.1	6/21/1808	5.5
5/19/1902	5.4		

2.4 MAXIMUM PAST GROUND SHAKING

The 1906 earthquake on the San Andreas Fault was the highest magnitude earthquake recorded in California. Lawson (1908, p. 259) reported considerable damage from the 1906 earthquake in the area, although it should be noted that the southern portions of the City of San Francisco and Daily City areas were sparsely to very sparsely developed at that time. “The intensity of the earthquake in general area was characterized as “strong” with many buildings partially wrecked and/or were moved upon their foundations, and a majority of the houses had the plaster badly cracked. Ninety-four percent of the chimneys fell, and dishes and similar objects were universally thrown down. Ground cracks and localized subsidence was noted near the edge of the bay (Lawson, 1908). Nason (1980) considers the damage here to be Rossi-Forel intensity VIII. The Loma Prieta earthquake of 1989 was smaller in magnitude (M 6.9) than the 1906 earthquake (M 8.25), but its epicenter was considerably further away from the site than the 1906 quake.

SECTION 3: SITE CONDITIONS

3.1 GEOMORPHOLOGY AND RECENT HISTORY

El Camino High School is located on a gently sloping to nearly level, southwesterly facing slope on the southern flank of San Bruno Mountain. The site is underlain by locally incised surficial deposits of Colma Formation which were graded in order to achieve the design grades for the various site buildings and athletic fields and ball courts. The extent of the grading would have been minor in the southern portion of the campus where the natural ground surface is gently inclined.

Aerial photographs listed in the References show the site vicinity spanning the period from 1943 to the present. As of 1943 the northern and western portions of the school was an undeveloped agricultural field, and a series of long narrow greenhouses were already constructed on the eastern portion of the site. At some point prior to the construction of the campus, a prominent southwest flowing drainage course partially in the alignment of the current Evergreen Drive was diverted to allow for urban development. By 1965 the existing campus had been constructed and the drainage channel appears to have been filled in within the vicinity of the campus. Between 1965 and 1993 the site was relatively unchanged. By 2000 landscaping in the central portion of southwest end of the campus was removed and replaced with parking areas. By 2012 solar panels were installed at the southernmost corner of the site. Since 2012 the site has remained relatively unchanged.

3.2 SITE RECONNAISSANCE AND SURFACE DESCRIPTION

The site is bounded by Lawndale Boulevard to the northwest, residential development to the northeast, residential development and Evergreen Drive to the southeast, and Mission Road to the southwest. As shown on Figure 2, Site Plan and Geologic Map, the proposed improvements are concentrated within the central portion of the school.

Our Certified Engineering Geologist performed a reconnaissance of the site on December 22, 2025. We noted that, due to the extensively developed nature of the site and adjacent areas, the subsurface earth materials (fill and Colma Formation) generally are not exposed. We inferred from observations of relative elevation changes and the adjacent street grades that minor cuts and fill placement were likely part of the original site grading in order to achieve the design grades. The upper portion and the middle portion of the campus building pads and fields are on cut and fill terraces, the lower portion of the campus is fill over alluvium. No significant cracking of concrete sidewalks or asphalt driveways and parking areas was observed along the inferred cut/fill contacts or at the top of steeper slopes with no signs of slope distress noted.

Based on review of GoogleEarth (WSG84 datum), existing site grades gently slope up from the intersection of Mission Road and Evergreen Drive at about Elevation 69 feet to the top of the slope adjacent to Lawndale Boulevard about Elevation 170 feet. The main portion of the campus is divided into approximately three terraces, with the lower terrace at around Elevation 90 feet. Building Unit A1, Unit C, and Unit D generally occupy the lower terrace. The middle terrace is generally at Elevation 103 feet with Building Unit B, Unit H, and Unit J. An approximately 10- to 15-foot-tall slope generally divides the middle and lower terrace with the slope trending downward in a southern direction, with slopes generally ranging from 6:1 to 4:1 (horizontal:vertical). The upper terrace is generally at Elevation 120 feet and greater with Building Unit G, Unit 2 through 5, and Unit E2. An approximately 15- to 20-foot-tall slope generally divides the upper and middle terrace with the slope trending downward in a southern direction, with slopes generally ranging from 6:1 to 3:1 (horizontal:vertical).

Surface pavements generally consisted of 2 inches of asphalt concrete (AC) supported directly on subgrade to overlying 10 inches of aggregate base. Based on our observations, the surface pavements are in fair condition; with localized alligator cracking in the drive aisle planned to be widened as part of this project.

3.3 AS-BUILT REVIEW

Based on our review of the RFP, we understand that the existing Building Unit G – Gym is typically supported on a continuous wall footing supporting precast tilt-up walls and interior slab-on-grade with a thickened edge. Approximately 20½ feet south of the Unit G – Gym’s southern tilt-up wall, a continuous wall footing for the exterior storefront is expected to be approximately 7 feet deep and approximal 3¼ feet wide. An approximately 12-foot-long cantilevered slab extends south of exterior storefront continuous footing. The cantilevered slab tapers in thickness from 5-inches at its most southern edge to 12-inches at the connection to the exterior storefront continuous footing.

3.4 SITE GEOLOGY AND SUBSURFACE CONDITIONS

We advanced six borings in the vicinity of the planned improvements and they are described below. Four geologic cross-sections, shown on Figures 7, 8, 9, and 10 were prepared to illustrate our interpretation of the subsurface conditions.

Beneath the surface grades, our explorations generally encountered undocumented fill overlying Colma Formation [Qc] deposits. A more detailed description of the subsurface conditions is presented in the following sections.

3.4.1 Undocumented Fills

"Undocumented fill" is generally considered a soil material that has been placed at a site without proper documentation regarding its source, quality, or compaction levels, meaning there is no record of how it was placed or if it meets currently accepted compaction standards.

Beneath the pavement section or ground surface, Borings EB-4, EB-6, EB-7, and EB-8, generally encountered undocumented fill to depths ranging between 1 to 8 feet. Borings EB-4, and EB-8 encountered undocumented fill to a depth of 2½ to 3½ feet consisting of loose to medium dense sand with varying amounts of silt and clay. Boring EB-6 encountered approximately 1 foot of undocumented fill consisting of dense clayey gravel with sand. Boring EB-7 encountered undocumented fill to a depth of 8 feet consisting of medium dense clayey sand.

Borings EB-5, EB-7, EB-9, and EB-10 did not encounter undocumented fills.

3.4.2 Colma Formation [Qc]

Beneath the undocumented fills or where undocumented fill was not present, our explorations encountered Colma Formation [Qc] consisting of medium dense to very dense silty sands, medium dense to very dense clayey sands, and very stiff to hard sandy lean clay to the maximum depth explored of 39½ feet. In general, medium dense Colma Formation sands were encountered to depths ranging between 3½ to 17½ feet. Dense to very dense Colma Formation sands and very stiff to hard Colma Formation clays were generally encountered below a depth of 17½ feet. Colma Formation was encountered as shallow as ½-foot beneath the surface pavements in Boring EB-5 and EB-9 and as deep as 8 feet in Boring EB-7.

Beneath the terminal boring depths, our CPT explorations (CPT-1, CPT-2, CPT-3, and CPT-4) generally encountered dense to very dense Colma Formation deposits to the maximum depth of 59½ feet, where practical CPT refusal was encountered. All four CPTs encountered practical CPT refusal within the Colma Formation.

3.4.3 Previous Field Explorations

We have included previous explorations B-3 and B-4, performed by Geosphere in 2012, in the central portion of campus, and explorations EB-3 performed by Cornerstone Earth Group in 2024 in the southeastern portion of the campus on our Figure 2, Site Plan and Geologic Map.

Beneath the pavement section Borings B-3 generally encountered dense clayey sand to a depth of 2½ feet, medium dense to dense silty sand to 18 feet and dense sand to 20 feet. Beneath the pavement section Boring B-4 generally encountered medium dense to dense sand to a depth of 20 feet. We note that no identification of "Colma Formation" was made in either boring

by the previous consultant. The Colma Formation contact point was inferred based on our surrounding borings and engineering judgement.

Previous explorations EB-1 through EB-3, performed for the previous Central Kitchen and Cafeteria project, generally encountered undocumented fill over alluvium [Qal] and Colma Formation [Qc] deposits. The alluvium encountered was anticipated to be part of the historic drainage channel associated with the CGZ Liquefaction Hazard Zone. These channel deposits were not encountered within any of the recent explorations performed across the campus for the modernization improvements and therefore are not anticipated to cross beneath any of the proposed improvements.

3.4.4 Plasticity/Expansion Potential

We performed three Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests resulted in PIs ranging between non-plastic and 15, indicating low expansion potential to wetting and drying cycles.

3.4.5 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 9 to 28 percent, indicating approximately 1 percent below to 18 percent above the estimated laboratory optimum moisture.

In addition, the soil encountered in Boring EB-9's upper 5 feet has an in-situ moisture content ranging from 17 to 28 percent, indicating 7 to 18 percent over the estimated laboratory optimum moisture.

3.5 GROUNDWATER

Groundwater was encountered in some of our explorations, Borings EB-5, EB-8, EB-9, and EB-10 and CPT-1, CPT-2, CPT-3, and CPT-4, at depths ranging between 7 and 22 feet below current grades corresponding to Elevations ranging between 71 and 115½ feet (Google Earth Datum, WGS84). All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered. The table below provides depth to groundwater that was encountered during our site investigation.

Table 5: Depth to Groundwater

Boring/CPT Number	Surface Elevation (Feet) [WGS84]	Depth to Groundwater (feet)	Groundwater Elevation* (feet) [WGS84]	Terminated Depth of Boring/CPT (feet)
EB-4	134	N.E. ¹	---	22.3
EB-5	125	19	106	24
EB-6	124	N.E. ¹	---	24.5
EB-7	123	N.E. ¹	---	39.3
EB-8	103	21.5	81.5	25
EB-9	91	13	78	25
EB-10	91	20	71	25
CPT-1	123	7.4 ²	115.6	40.0
CPT-2	103	22.4 ²	80.6	22.6
CPT-3	91	7.0 ²	83.0	59.5
CPT-4	91	11.0 ²	79.0	44.1

1 – N.E. is an abbreviation for Not Encountered during our explorations.

2 – Depth to encountered groundwater is an estimation from the pore pressure dissipation test performed in the CPT explorations

Historic high groundwater within the general project vicinity has been mapped at a depth of approximately 2 to 10 feet below existing grades (CGS, San Francisco South 7.5-Minute Quadrangle, 2021). A mapped high groundwater depth contour of 2 feet generally follows the southeastern property boundary and Evergreen Drive, which trends in a northeast to southwest direction. The mapped high groundwater depth contour of 10 feet generally cuts through the terrace that is located between the Unit G and the Unit B and Unit H buildings. We also reviewed groundwater data available online from the website GeoTracker (geotracker.waterboards.ca.gov). Groundwater measurements collected in May 2005 ranged from approximately 13 to 28 feet below ground surface at a site approximately ½-mile south of the site, and groundwater measurements collected between July 2016 and January 2021 ranged from 10 to 18 feet below ground surface at another site approximately ½-mile south-southwest of the site.

As the site is generally terraced, we assume depth to groundwater is variable, but have grouped generally based on the surface elevation of the respective terraces for our analyses. Within the lower terraces (surface elevation 108 feet and lower in elevations), we generally evaluated a groundwater depth of two feet below current site grades, with the exception of CPT-4. CPT-4 was evaluated with a groundwater depth of five feet below current site grades as CPT-4 was performed along the northwestern property boundary and therefore closer to the 10 foot contour (CGS). As site grades rise with the adjacent terrace (surface elevation 109 feet and greater in elevations), we generally evaluated a groundwater depth of 10 feet below current site grades for our analyses. The table below presents our general design groundwater depth, except for CPT-4. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

Table 6: Depth to Design Groundwater Depth

Surface Elevation (feet) [WGS84]	Design Groundwater Depth (feet)
108 and lower in elevation	2
109 and greater in elevation	10

3.6 CORROSION SCREENING

We tested three samples collected at depths of 2, 4, and 6 feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in the table below.

Table 7: Summary of Corrosion Test Results

Sample Location	Depth (feet)	Soil pH ¹	Resistivity ² (ohm-cm)	Chloride ³ (mg/kg)	Sulfate ^{4,5} (mg/kg)
EB-6	6.0	7.8	6,727	4	9
EB-7	2.0	7.2	4,661	3	82
EB-10	4.0	6.9	9,314	2	9

Notes: ¹ASTM G51
²ASTM G57 - 100% saturation
³ASTM D3427/Cal 422 Modified
⁴ASTM D3427/Cal 417 Modified
⁵1 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 to affecting corrosion potential.

3.6.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 7 and published correlations between resistivity and corrosion potential, the soils may be considered moderately to mildly corrosive to buried metallic improvements, and non-corrosive to concrete improvements (Chaker and Palmer, 1989).

In accordance with the 2025 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-25 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, a cement type restriction is not 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. We have summarized applicable exposure categories and classes from ACI 318-25 Table 19.3.1.1 below.

Table 8: ACI 318-25 Table 19.3.1.1 Exposure Categories and Classes

Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
F0 ¹	S0 ²	W1 ³	C1 ⁴

1 (F0) "Concrete not exposed to freezing-and-thawing cycles" (ACI 318-25)

2 (S0) "Water soluble sulfate in soil, percent by mass" (ACI 318-25)

3 (W1) "Concrete exposed to moisture but not to an external source of chlorides" (ACI 318-25)

4 (C1) "Concrete is exposed to moisture, but not to an external source of chlorides" (ACI 318-25)

We recommend the structural engineer and a corrosion engineer be retained to confirm the above information and provide additional recommendations, as needed.

3.7 IN-SITU WATER INFILTRATION TESTING PROGRAM

To estimate the infiltration rate of the soils at locations and depths provided by the RFP for the project, we performed four in-situ field infiltration tests on January 5, 2026, using a Guelph permeameter by SoilMoisture Equipment Corp., Model #2800, in general accordance with ASTM D5126. Generally, the Guelph permeameter is a constant head device which uses two water-filled chambers to measure infiltration rate in a shallow borehole. A constant head level is established in the borehole and the rate of water outflow into the surrounding soil is documented. The rate of flow when it reaches a steady state, or constant rate, is used to determine an approximate infiltration rate for that location and depth.

The approximate location of the field infiltration tests (I-1 through I-4) are shown on the Site Plan and Geologic Map, Figure 2. The infiltration tests were performed at approximate depths of 1¾ and 2 feet below existing site grades. The test results are summarized below.

Table 9: In-Situ Field Guelph Permeameter Test Results

Boring / Infiltration Test	Soil Type	Infiltration Depth (feet)	Infiltration Rate (in/hour)
I-1	Clayey Sand (SC)	1¾	0.0
I-2	Clayey Sand (SC)	1¾	0.0
I-3	Clayey Sand (SC)	1¾	0.7
I-4	Clayey Sand (SC)	2.0	0.1

3.7.1 Reliability of Field Test Data

Test results may not be truly indicative of the long-term, infiltration rates. Other factors including stratifications, heterogeneous deposits, overburden stress, disturbance, organic content, and other factors can influence test results. In addition, for stratified soils such as those encountered at the site, the average horizontal permeability is typically greater than the average vertical permeability.

3.7.2 Findings and Recommendations

The infiltration tests performed resulted in infiltration rates of approximately 0.0 to 0.7 inch per hour (in/hr). Infiltration tests I-1, I-2, and I-4 indicate that the near surface soil generally has a very low potential for water infiltration. Infiltration test I-3 indicates that the near surface soil generally has a moderate potential for water infiltration.

We recommend the above estimate be confirmed in the field at the time of construction, as required. In addition, the project civil engineer should review the above infiltration and provide additional recommendations as deemed necessary.

3.7.3 Comments and Design Considerations

As discussed, the tests were performed at discrete locations and depths. In addition, some disturbance in preparing the test also can occur. Therefore, the above results can vary significantly and may not be representative over the entire site. Additionally, the infiltration tests were performed during inclement weather. The surface water was directed away from our infiltration tests, however, subsurface water flow was observed to occur in infiltration tests I-1 and I-2.

Localized areas and depths with higher or lower permeable materials can increase or decrease the actual infiltration rates. Therefore, we recommend the potential for variations be considered when evaluating the soil infiltration capacity or performance.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 100 kilometers of the site. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone, known formerly as a Special Studies Zone (CDMG, 1982) and no surface expression of active faulting was seen on aerial photographs or during the geologic field reconnaissance by our Engineering Geologist. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 HISTORICAL GROUND FAILURES

Youd, et al. (1978) and Knudsen, et al. (2000) report that during the 1906 earthquake, pavements were buckled, arched, and fissured in the business district of San Francisco, which was built on ground made by filling in the cove of Yerba Buena. They show two occurrences of lateral spreading and also areas of ground cracking (of an uncertain origin) in the Colma area, approximately two miles to the west-northwest of the school, in the area of El Camino Real.

There were no reported occurrences of ground failure in the site vicinity during the 1989 Loma Prieta Earthquake (Seed et al, 1990, Tinsley et al, 1998).

4.3 LIQUEFACTION POTENTIAL

As shown on Figure 6, CGS EZRIM Map, a portion of the school lies within a State-designated Liquefaction Hazard Zone (CGS, South San Francisco Quadrangle, 2021). However, the proposed improvements within the gymnasium and locker rooms and the proposed new canopies are not located within the mapped Liquefaction Hazard Zone. The lower terrace driveway expansion with the proposed retaining wall appears to be proposed within the mapped Liquefaction Hazard Zone. Refer to Figure 2 site plan for the location of the proposed improvements relative to the mapped Liquefaction Hazard Zone. Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet or where practical CPT refusal was met, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

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

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design groundwater depth ranging between 2 and 10 feet. 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 groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_c) to estimate the plasticity of the layers.

The results of our analyses (CPT-1, CPT-2, CPT-3, and CPT-4) are presented in Figures 11A through 11D of this report. Calculations for these CPTs are attached as Appendix D.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface of less than ½-inch based on the Yoshimine et al. (2006) method. As noted above, the soils encountered at the site below the surficial fills, including the medium dense silty and clayey sands are classified as Colma formation and of Pleistocene in age. As such, we anticipate that seismically induced settlements observed at the site after a design level event will be less than the estimates presented above. Additionally, dense to very dense Colma Formation sands and very stiff to hard Colma Formation clays were generally encountered below a depth of 17½ feet. Therefore and in our opinion, we did not encounter potentially liquifiable material below the depth of 17½ feet.

As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of less than ⅓-inch over a horizontal distance of 30 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation 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 Garriss (1995) indicates that the non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

As discussed above, the site is terraced with existing site grades gently slope up from the intersection of Mission Road and Evergreen Drive at Elevation 69 feet to the top of the slope adjacent to Lawndale Boulevard Elevation 170 feet (Datum unknown). The slopes between the terraces generally range between 10 to 20 feet in height.

As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements in the vicinity of CPT-1 and CPT-2 based on the LDI calculations are considered negligible.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff Colma Formation clays and medium dense to dense Colma Formation sands above the design groundwater depth, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

Regional seismic hazard mapping by CGS (CGS, 2021) does not show the project site located within an earthquake-induced landslide zone. There are no landslides mapped in at or immediately adjacent to the site. According to mapping by Bonilla, 1998, three small landslides are present on the southern flank of San Bruno Mountain approximately 0.85 to 1.25 miles east-northeast of the site.

Minor cut and fill slopes occur around the campus. These slopes are generally moderately inclined, ranging up to 3:1 (H:V) and no greater than approximately 20 feet in height. These slopes generally appeared to be cut slopes. Our review of historic aerial photos confirmed the absence of any landslide-related features at the site. Our field reconnaissance documented the absence of cracking or damage of the built infrastructure including pavements and sidewalks that might be associated with slope instability on cut and fill slopes at the site. The potential for landsliding occurring at the site is judged to be low.

4.7 FLOODING AND RESERVOIR INUNDATION

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

Based on interactive maps maintained by the California Division of Safety of Dams (DWR, 2015) the closest reservoirs to the site are the Stanford Heights reservoir and twin University Mound sites over 4 miles to the northeast and northwest of the site respectively. Inundation

maps show the watersheds flow north and away from the project site. Additionally, the San Andreas Lake site is over 3½ miles south-southeast of the site. Inundation maps show the watershed flow is east to the bay and away from the project site.

We therefore conclude that hazards from flooding or dam failure resulting in reservoir inundation are low.

4.8 TSUNAMIS/SEICHES

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 2½ miles inland from the San Francisco Bay shoreline and is approximately 75 to 135 feet above mean sea level (Google Earth, WGS84). In addition, the site is not located within a Tsunami Design Zone (ASCE Tsunami Hazard Tool, 2022). Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.9 VOLCANIC ERUPTION

Hazards associated with active volcanoes include inundation by ash, pyroclastic flows, and mudflows. The severity of volcanic hazards is associated with distance to the volcanic source, magnitude and type of volcanic activity, and direction of prevailing winds. No young volcanic deposits or other indications of recent volcanic activity are present in the immediate vicinity of the school campus.

In collaboration with the California Governor's Office of Emergency Services and the California Geological Survey, the US Geological Survey published a Volcanic Threat Assessment report which provides a broad perspective on California's exposure to volcanic hazards (Mangan et al., 2019). The school campus is located several hundred miles from the nearest potentially or historically active areas of volcanic activity (the Clear Lake volcanic field and Long Valley volcanic area). San Mateo County is not one of the counties identified with elevated volcanic hazards (Mangan et al., 2019). Volcanic eruption hazard for the school site therefore is very low.

4.10 NATURALLY OCCURRING ASBESTOS

Naturally occurring asbestos (NOA) is the geologic occurrence of six types of naturally occurring asbestos minerals including chrysotile, crocidolite, amosite, asbestiform tremolite, asbestiform actinolite, and asbestiform anthophyllite (CGS, 2002). Ultramafic rock such as serpentine and greenstone can contain NOA. Medical studies have shown a potential connection between environmental exposure to these naturally occurring types of asbestos and cancer.

Naturally occurring asbestos (NOA) in California typically is associated with ultramafic bedrock, which has not been identified at the project site based on our review of boring logs and site reconnaissance observations. The Colma Formation present beneath the project site is not

known to contain NOA. No site-specific hazard for NOA is identified in the immediate vicinity of the school in published regional mapping of asbestos hazard for the State of California developed jointly by the USGS and CGS (Gosen and Clinkenbeard, 2011).

The nearest outcrops of serpentinite that could potentially contain NOA have been mapped about 3 miles east-southeast of the site in the Oyster Point neighborhood of South San Francisco. Similar bedrock that might contain ultramafic rocks potentially containing NOA are not mapped under the site, and not observed during our site reconnaissance, or encountered in the geotechnical borings. There is a possibility that imported fill used in the construction of the campus may contain earth materials containing asbestos, however those have not been observed. Therefore, it is our opinion that the potential for NOA to be present at the site does exist but is low.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

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.

- Potential for seismic settlements
- Presence of undocumented fill
- Potential for underpinning/proximity to existing foundations
- Presence of cohesionless sandy soils
- Potential for shallow groundwater
- Soil corrosion potential

5.1.1 Potential for Seismic Settlements

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. As noted above, based on our analysis and review of explorations, the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils and impact the proposed improvements is low. Our analysis indicates that liquefaction-induced settlement on the order of less than 1/2-inch could occur, resulting in differential settlement less than 1/3-inch. Foundations should be designed to tolerate the anticipated total and differential settlements. Based on estimated foundation loads, it should be feasible to support the proposed building improvements on shallow foundations; however, the foundations will need to be designed to tolerate total and differential settlement due to static loads and liquefaction-induced settlement. Detailed foundation recommendations are presented in the "Foundations" section.

5.1.2 Presence of Undocumented Fill

As discussed, the site is blanketed by 1 to 8 feet of undocumented (man-made) fill that appears to be derived from on-site soils. The existing fills are locally loose and may be highly variable following site demolition and may not uniformly support the proposed structures and adjacent improvements. Therefore, we recommend that all undocumented fill material present beneath proposed building areas be over-excavated and re-compacted prior to foundation construction.

In addition, we understand that a historic drainage channel ran south of the proposed improvements was infilled sometime during initial site development. Any undocumented fill encountered during site grading should be completely reworked from within improvement areas. The final depth and lateral extent of undocumented fills over-excavation within the building footprint should be further evaluated during construction. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

If observation and testing records are available for the existing fill materials, they should be provided for our review. Provided undocumented fills are adequately over-excavated and re-compacted as engineered fill, the potential impact due to undocumented fill settlement should be low. A more detailed discussion of undocumented fill mitigation is presented in the “Earthwork” section.

5.1.3 Potential for Underpinning and Proximity to Existing Foundations

We understand the proposed foundations within Unit E2 or the Unit G – Gym improvements may be directly adjacent to existing building slab-on-grade or foundations. Underpinning may be required in areas where excavations and new foundation elements extend below the bottom of adjacent existing footings. Cuts adjacent to the existing slab should be properly supported to prevent soil from sloughing underneath the existing slab. Existing footings should also be properly supported and excavations extending into the influence zone of existing footings may need to be performed in sequence and may need to be underpinned depending on the specific situation. In addition, any new loads placed adjacent to existing building foundations from the proposed improvements may cause additional settlement beneath the existing building foundations. Further discussion and recommendations addressing this concern are presented in the “Earthwork” and “Foundation” section of this report.

5.1.4 Cohesionless Sandy Soils

The site is underlain by generally cohesionless, sandy soils with silty and clayey fines ranging from 16 to 36 percent. Due to the presence of cohesionless sandy soils, excavation sidewalls for demo, foundations, retaining walls, etc. may cave in or accumulate a significant amount of slough and will likely not stand vertical when excavated. Trenches for utilities and other excavations will likely have to be sloped to accommodate the potential caving and sloughing conditions. Grading and excavation contractors should be made aware of this condition and plan on forming footings, sloping trench sidewalls, preparing slab-on-grade subgrade just prior to concrete placement, and other similar construction issues as relates to temporary slopes, utility excavations, etc.

In addition, these types of soils are highly subject to erosion from wind and water. We recommend that any new final slopes in sandy soils be 3:1 (H:V) or flatter to limit erosion. All exposed surfaces should be vegetated or otherwise protected from erosion.

5.1.5 Potential for Shallow Groundwater Conditions

Shallow groundwater was measured at depths ranging from approximately 7 to 22 feet below the existing ground surface. As discussed above, we recommend a high design groundwater depth of two to ten feet, refer to “Groundwater” section above. Our experience with similar sites in the vicinity indicates that shallow groundwater 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.

5.1.6 Soil Corrosion Potential

As noted above, a preliminary soil corrosion screening was performed based on the results of analytical tests on samples of the near-surface soil. In general, based on the preliminary screening, the corrosion potential for buried metallic structures, such as metal pipes, is considered moderately to mildly corrosive and non-corrosive to buried concrete. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 5 feet for sulfates and pH to confirm the classification of corrosive to mortar coated steel and concrete.

5.2 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.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the explorations 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 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

The areas designated for improvements should be cleared of all surface and subsurface deleterious materials including any pre-existing foundations, slabs, buried utility lines, and pavements, if encountered. Excavations extending below the planned finished improvement grades should be cleaned and backfilled with suitable material compacted to the recommendations presented in the “Compaction” section of this letter. We recommend that the backfilling be carried out under our observation.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, and utilities within the areas of planned improvements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

6.1.2 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. Sand-cement slurry should have a minimum compressive strength of 75 psi.

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 risk for owners 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.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 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.

6.3 MITIGATION OF UNDOCUMENTED FILLS

As noted above, 1 to 8 feet of undocumented fills were encountered within our exploratory Borings EB-4, EB-6, EB-7, and EB-8. Borings EB-4, EB-5, and EB-6 were performed in the vicinity of Unit E2, where the proposed new elevator and catwalk foundations are proposed. Borings EB-4, EB-5, and EB-6 were performed in the vicinity of Unit G – Gym, where new foundations and ADA compliance are planned.

Undocumented fill is anticipated to be encountered within the existing building Unit E2 as a result of prior development grading. In addition to the fills encountered within our borings, we understand that a historic drainage channel was infilled around the time of initial site development. Based on review of the samples collected in our exploratory borings, some of the fills appear to have been previously placed with some compaction effort. However, due to the variable nature of undocumented fill, we recommend that, where encountered, all undocumented fills be completely reworked within improvement areas. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

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 that the upper 12 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below. In our opinion, the fills encountered at this site are acceptable to be left in place beneath planned pavement and flatwork areas.

6.4 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. Based on our site exploration, the loose sands encountered in the upper three feet shall be classified as OSHA Soil Type C materials. Recommended soil parameters for temporary shoring are provided in the “Temporary Shoring” section of this report.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. All temporary excavations should be sloped in accordance with OSHA soil classification requirements.

6.5 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.

6.6 WET SOIL STABILIZATION GUIDELINES

Native soil 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.

As discussed in the “Subsurface” section in this report, the in-situ moisture contents are about one percent below to 18 percent over the estimated laboratory optimum in the upper 10 of the soil profile. In addition, the soil encountered in Boring EB-9’s upper 5 feet has an in-situ moisture content ranging from 17 to 28 percent, indicating 7 to 18 percent over the estimated laboratory optimum. Groundwater is anticipated to occur as shallow as 2 feet below surface grades as well. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

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

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of six 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.

6.6.2 Removal and Replacement

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

6.6.3 Chemical Treatment

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

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

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

6.7.2 Potential Import Sources

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

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

6.8 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches 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 "Wet Soil Stabilization Guidelines" section of this report.

Table 10: Compaction Requirements

Description	Material Description	Minimum Relative ¹ Compaction (percent)	Moisture ² Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Below Shallow Foundations	On-Site Soils	90	>1
Retaining Wall Backfill	Without Surface Improvements	90	>1
	With Surface Improvements	95 ⁴	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base ³	90	Optimum

Table 10 continues

Table 10: Compaction Requirements (continued)

Description	Material Description	Minimum Relative ¹ Compaction (percent)	Moisture ² Content (percent)
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base ³	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

¹Relative compaction based on maximum density determined by ASTM D1557 (latest version)

²Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)

³Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

⁴Using light-weight compaction or walls should be braced

6.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (¾-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.

6.10 UNDERPINNING

Cuts for the proposed new foundations and sections of slab-on-grade should be properly supported to prevent soil from sloughing out from underneath the existing slabs and

foundations. If deeper cuts do occur and extend beneath existing footings, the footings should also be properly supported and excavations extending into the influence zone of existing footings may need to be performed in sequence and may need to be underpinned depending on the specific situation. There are several methods that could be used to underpin existing footings, if needed. This may include building haunches to below the bottom of excavation depth beneath the footing. The haunches would act as small footings and should not exceed the bearing capacity for the existing building foundations.

On a preliminary basis, to address this issue, the footings may be underpinned in a segmented manner. Segments for underpinning from a geotechnical perspective generally range from be about 3 to 4 feet wide, at 8 to 12 foot spacing from center, and excavated at alternate locations along the existing footing width. The final underpinning design should be determined by the structural engineer. Underpinning should be constructed in an alternating segmented manner to avoid undermining the entirety of the existing footing all at once. Underpinning excavations should be continuously lagged from the top down to avoid caving of the native soil. Alternatively, the footing could be supported with periodic helical anchors. The helical anchor should be designed and installed by a helical anchor design-build contractor. Cornerstone should review any proposed underpinning design prior to the start of construction and should be onsite while operations for underpinning or excavating in sequence beneath foundations are being performed. Additionally, we recommend that, where possible, underpinning excavations be excavated and backfilled same day.

6.11 SITE DRAINAGE

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

6.12 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

6.12.1 Storm Water Infiltration

To aid in determining if storm water infiltration may be feasible at the site, we provide the following site information that may aid in determining the feasibility of infiltration facilities at the site.

- Based on our exploration and review of regional geologic and USDA soil survey mapping, the near-surface soils at the site consist of clay and sandy clay that are likely categorized as Hydrologic Soil Group D. These soils are generally expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, the Colma Formation

soils will significantly limit the infiltration of stormwater. If site specific infiltration data is needed for storm water design, we recommend that field infiltration testing be performed near planned storm water improvements.

- It should be noted that seasonal high groundwater is mapped at a depth of 2 to 10 feet, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

6.12.2 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, we recommend the following items be considered for design and construction.

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements.
- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.
- If required, the bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements.

6.12.3 LID Improvements Adjacent to Pavements

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

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or
- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in

accordance with the recommendations in the “Retaining Walls” section of this report, including vehicular or fire truck surcharge loading.

- 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.

6.13 LANDSCAPE CONSIDERATIONS

We recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- 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 7: CBC SEISMIC DESIGN CRITERIA

7.1 CBC SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2025 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Shear wave velocity measurements were collected while advancing CPT-1, CPT-2, CPT-3, and CPT-4. All four CPTs encountered practical CPT refusal within the underlying Colma Formation. Following Section 20.3 of ASCE 7-22, we extended the last measured shear wave to a depth of 100 feet, to calculate V_s . This resulted in a time-averaged shear wave velocity for the top 30 meters (V_s) ranging between 1044 to 1425 feet per second (318 to 434 meters per second). Per Table 20.2-1 of ASCE 7-22, we have classified the project site as Soil Classification CD. The mapped spectral acceleration parameters S_s and S_1 were calculated using the ASCE Hazard tool, located at <https://ascehazardtool.org/>, based on the site

coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters for Site Class CD.

Table 11: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	CD
Site Latitude	37.665976°
Site Longitude	-122.442188°
MCE _R Spectral Response Acceleration at 0.2 seconds for Site Class CD, S _s	2.51g
MCE _R Spectral Response Acceleration at 1 second for Site Class CD, S ₁	0.99g
MCE _R Spectral Response Acceleration at Short Periods Adjusted for Site Class Effects - S _{MS}	2.7g
MCE _R Spectral Response Acceleration at 1-second Period Adjusted for Site Class Effects – S _{M1}	1.97g
Design Spectral Response Acceleration at Short Periods – S _{DS}	1.8g
Design Spectral Response Acceleration at 1-second Period – S _{D1}	1.31g
MCE _G Peak Ground Acceleration – PGA _M	0.95g

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed improvements may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed. Optional foundation recommendations for drilled, cast-in-place piers are also provided for the proposed canopies or other onsite improvements.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings

Conventional shallow footings should bear on engineered fill and follow the minimum footing dimensions in the table below. 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 noted above, where undocumented fills are encountered within the bottom of footing excavations, the fill should be completely removed, moisture conditioned, and re-compacted in place as engineered fill to a minimum of 90 percent relative compaction. In areas where the fills are removed as part of the foundation excavation or where no fills are encountered, the bottom of footing excavation should be re-compacted in place to a minimum of 90 percent relative compaction.

Table 12: Minimum Footing Dimensions

Number of Building Stories	Minimum Footing Width (inches)	Minimum Depth to Bottom of Footing (inches)
1	12	15
2	15	18
3	18	24

Footings constructed to the above dimensions are capable of supporting a maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads. These pressures are based on factors of safety of 3.0 and 2.0 applied to the ultimate bearing pressure for dead, dead plus live, respectively. The structural engineer should apply design coefficients and factors for seismic force-resisting systems (i.e. overstrength factor of the structure) to the ultimate bearing capacity of 6,000 psf to determine the seismic (all loads) allowable bearing capacity.

These 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.

8.2.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 13: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
New Isolated Column Footing	60 to 120 kips
New Perimeter Strip Footing	2 to 6 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½-inch, with about ⅓-inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be on the order of ⅓-inch over a horizontal distance of 30 feet, resulting in a total estimated differential footing movement of up to about ⅔-inch between foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

8.2.3 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 0.4 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. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity. The structural engineer should apply appropriate factors to the ultimate values provided, as appropriate.

8.2.4 Conventional Shallow 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.

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.

Due to the presence of clean sand and gravel, footing excavation walls will likely not stand vertical and will need to be sloped to a minimum 1:1 inclination or Stay-Form or similar may need to be placed within the footing excavations as they are excavated during construction of the foundation elements. Granular material encountered in the footing bottoms will likely be disturbed to a depth of 6 to 8 inches following excavation and will need to be compacted to 90 percent relative compaction prior to steel placement. Care should be taken to not disturb the compacted granular material during steel placement. We should re-observe the footing excavations in granular materials after reinforcing steel has been placed and just prior to concrete placement. Footing excavations should also be kept moist by regular sprinkling with water to prevent desiccation and potential raveling of the granular materials. As an alternative, a rat slab can be placed over the granular material after we have observed the footing excavation to protect the granular material prior to steel placement.

8.3 DRILLED PIERS – CANOPIES

If shallow footings are determined not feasible or shallow footings are not desired, the proposed new canopies may be supported on drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of 12 inches and extend to a depth of at least 6 feet below

adjacent grade. Adjacent piers centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required.

8.3.1 Vertical Capacity and Settlement

The vertical capacity of the piers may be designed based on an ultimate skin friction of 250 pounds per square foot (psf) to a depth of 5 feet, 500 psf from 5 to 10 feet, and 800 psf below a depth of 10 feet. Frictional resistance to uplift loads may be developed along the pier shafts based on an ultimate frictional resistance of 80 percent of the downward capacities; the structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate uplift capacity. The upper 12 inches of soil from the top of the pier should be neglected for design purposes due to the present of undocumented fill and adjacent landscaping soils.

Total static settlement of individual piers should not exceed ½-inch to mobilize static capacities and post-construction differential settlement between adjacent piers should not exceed ¼-inch due to static loads.

8.3.2 Lateral Capacity

Lateral loads exerted on the structure may be resisted by a passive resistance based on an ultimate equivalent fluid pressure of 400 pcf acting against twice the projected area of piers, up to a maximum uniform pressure of 2,000 psf at depth. The upper 12 inches of soil should be neglected when determining lateral capacity due to the presence of undocumented fills and landscaping soils. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

8.3.3 Drilled Pier Construction Considerations

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the soil profile, verify that the piers extend the minimum depth into suitable materials, and that the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry, and relatively free of loose material before reinforcing steel is installed and concrete is placed. If groundwater cannot be removed from the excavations prior to concrete placement, drilling slurry or casing may be required to stabilize the shaft and the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water or drilling slurry in the concrete.

Based on our explorations in the location of the new canopies, Colma Formation sandy soils with silty and clayey fines ranging from 16 to 36 percent were encountered. We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult drilling conditions. The contractor should plan on encountering potentially caving soils and other materials that may require casing or other stability measures to prevent caving and sloughing into the pier foundations if piers extend more than about 6 feet beneath site grades.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

For the areas of new interior slabs-on-grade, as the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed slabs-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade 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. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

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 15-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
3/4”	90 – 100
No. 4	0 – 10
No. 200	0 – 5

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.3 EXTERIOR FLATWORK

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

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 10. The design R-value was chosen based on the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 14: Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base ¹ (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.0	9.5
4.5	2.5	8.5	11.0
5.0	3.0	9.0	12.0
5.5	3.0	11.0	14.0
6.0	3.5	11.5	16.0
6.5	4.0	12.5	16.5

¹Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 10

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 use the pavements.

10.1.1 Pervious Asphalt Cement Concrete

We understand that pervious pavements may be desired. For pervious asphalt pavements, we assume that pervious pavements would be used in auto parking or drive aisles, and that pervious pavement areas would not be constructed in heavy truck traffic or truck parking areas. Pervious asphalt concrete pavements generally consist of two surface layers; an Open Graded Friction Course (OGFC) asphalt layer over an asphalt-treated permeable base layer (ATPB). These layers are supported on a uniformly sized crushed rock reservoir layer consisting of ¾- to 1½-inch diameter clean, crushed rock. In some instances, subgrade of the section is left unprepared to encourage infiltration; however, the permeability of the subgrade at the subject site is very low to moderate, therefore subgrade can be prepared normally. If drainage of the reservoir layer is desired, subdrains would be required.

For a Traffic Index (TI) of 4.0, we recommend the pervious asphalt concrete areas consist of 1-inch of OGFC over 2½ inches of ATPB, all over at least 7- inches of a uniform size, open-graded crushed rock (¾- to 1½-inch diameter). For a TI of 5, we recommend the pervious asphalt concrete areas consist of 1-inch of OGFC over 3-inches of ATPB, all over at least 9-inches of uniform size, open-graded crushed rock. The crushed rock and underlying subgrade should be prepared in accordance with the “Earthwork” section of our report.

In order for pervious pavements to maintain their effectiveness, they require occasional maintenance consisting of brooming or vacuuming to remove the fines from the surface. Pervious pavements should not have surface treatments, such as slurry or fogs seals applied, as this will seal off the pavement and prevent surface water infiltration.

10.2 PORTLAND CEMENT CONCRETE

The Portland Cement Concrete (PCC) pavement recommendations outlined below are based on methods presented in American Concrete Pavement Association (ACPA, 2006). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. The following table presents minimum PCC pavements thicknesses for various traffic loading categories and the anticipated maximum Average Daily Truck Traffic (ADTT).

Table 15: PCC Pavement Recommendations

Traffic Category	Minimum PCC Thickness ¹ (inches)	Class 2 Aggregate Base (inches)
Maximum ADTT = 3	5.0	6.0
Maximum ADTT = 20	5.5	6.0

¹Subgrade design R-Value = 10

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, 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.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. 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 be designed for the following pressures:

Table 16: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	40 pcf	40 pcf + 8H
3:1	45 pcf	45 pcf + 8H
2½:1	50 pcf	50 pcf + 8H
2:1	60 pcf	60 pcf + 8H
Additional Surcharge Loads	⅓ of vertical loads at top of wall	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure

** 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 2025 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls greater than 6 feet in retained soil height. At this time, we are not aware of any retaining walls greater than 6 feet in retained soil height for the project. The design of walls with less than 6 feet in retained soil height, for seismic lateral earth pressures in addition to static earth pressures is not required by the 2025 CBC, and in our opinion is not warranted. If retaining walls are planned with retained soil heights of greater than 6 feet, we should be notified to evaluate the potential for seismic lateral earth pressures.

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.

11.5 FOUNDATIONS

In general, conventional at-grade site retaining walls may be supported on continuous conventional footings. Strip footings should bear on 12-inches of engineered fill and extend at least 12 inches below the lowest adjacent grade.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting a maximum allowable bearing pressure of 1,675 psf for dead loads, 2,500 psf for combined dead plus live loads, and 3,325 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the 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.

Based on our explorations CPT-3 and CPT-4, potentially liquifiable material was encountered at or near the anticipated conventional shallow foundation zone for the proposed retaining wall located along the lower terrace (surface elevation around 90 feet). We recommend that the bottom of retaining wall footings proposed along the lower terrace be compacted in place to a minimum of 90 percent relative compaction.

Based on our cursory review it appears that liquefiable layers could potentially impact the performance of shallow foundations; therefore, we screened the foundations for reduced bearing capacity based on the above criteria. Based on our analysis, the above recommended bearing capacity and other criteria, as necessary, have been adjusted to account for a reduction in shear strength in those layers considering the effects of liquefaction caused by MCE_G earthquake motions.

11.5.1 Lateral Loading for Retaining Walls

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 0.4 applied to the un-factored footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pcf may be used in design. The available soil resistance for sliding can be determined using the provided ultimate frictional resistance values and the ultimate passive pressures provided in this paragraph to calculate the net available soil resistance. Based on the 2025 CBC Section 1807.2.3, the factor of safety against sliding is then determined as this available soil resistance divided by the net lateral force applied to the wall, with a minimum required Factor of Safety of 1.5 (or 1.1 when earthquake loads are included).

Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of South San Francisco Unified School District specifically to support the design of the El Camino High School Modernization project in South San Francisco, 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 groundwater 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.

South San Francisco Unified School District may have provided Cornerstone with plans, reports and other documents prepared by others. South San Francisco Unified School District 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 their representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

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

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

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work

has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

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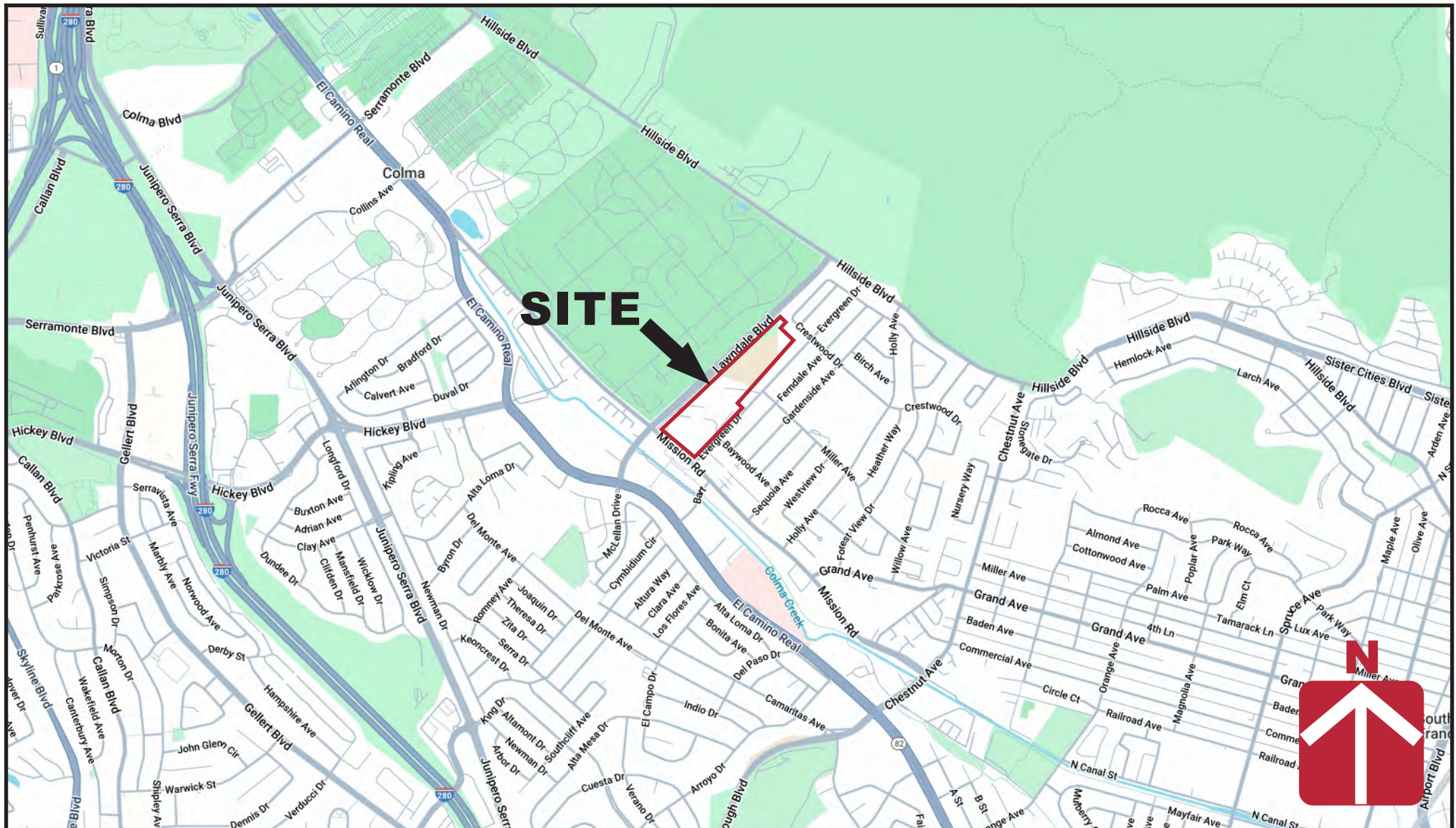
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Vicinity Map

El Camino High School Modernization
1320 Mission Road
South San Francisco, CA

Project Number

1517-1-5

Figure Number

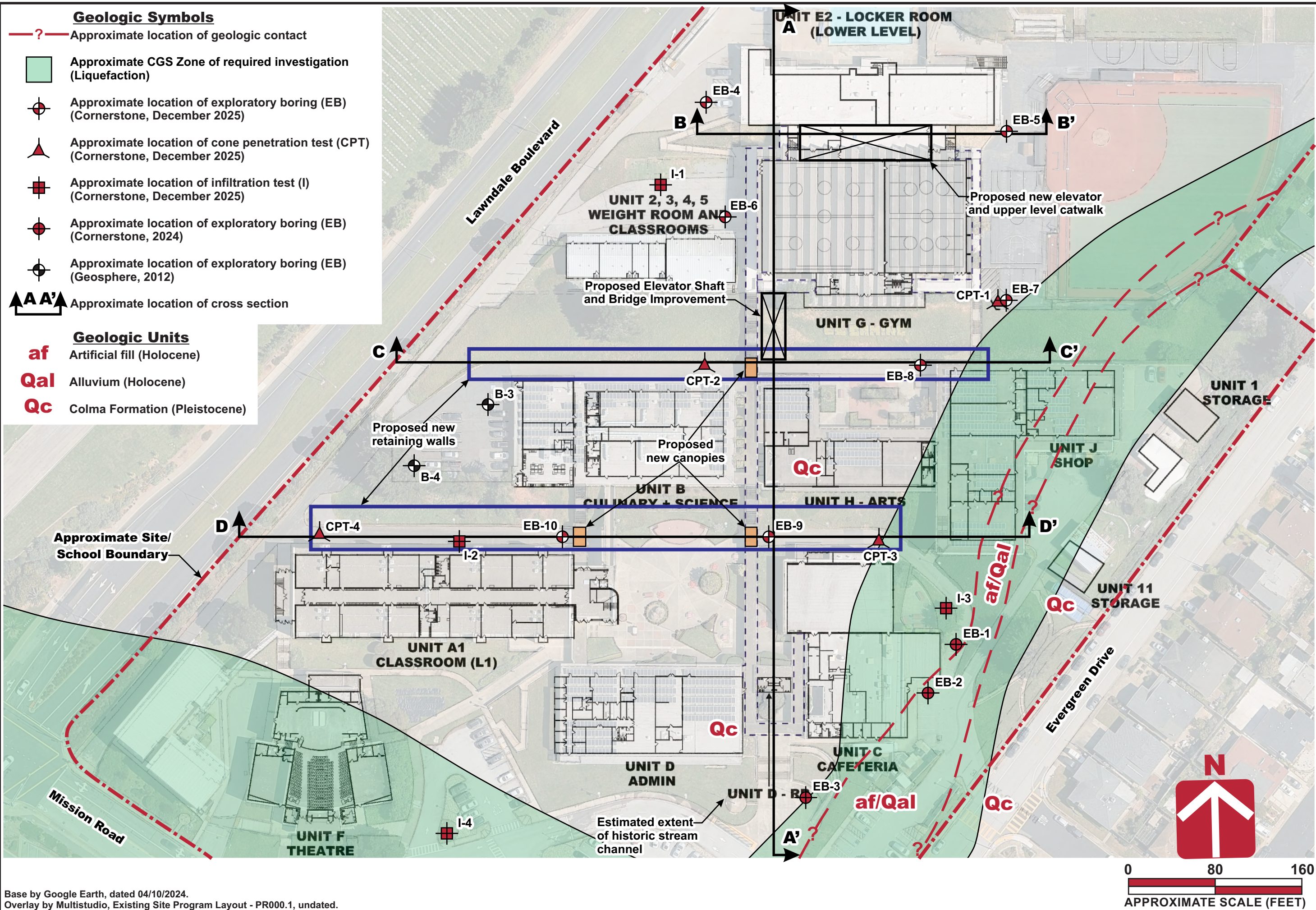
Figure 1

Date

January 2026

Drawn By

RRN



1517-1-5

Figure Number

Figure 2

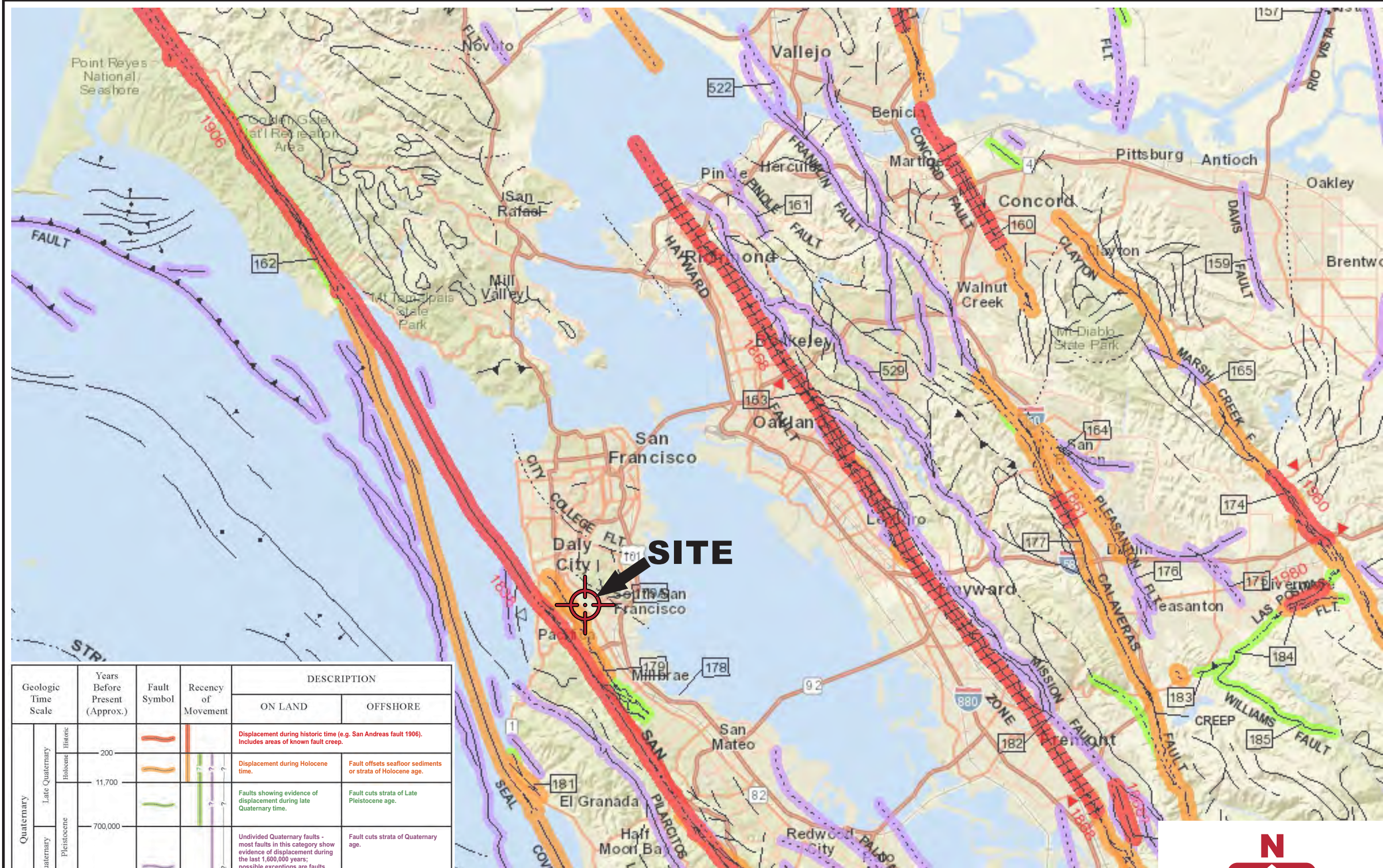
Date February 2026

Drawn By RRN

Site Plan and Geologic Map

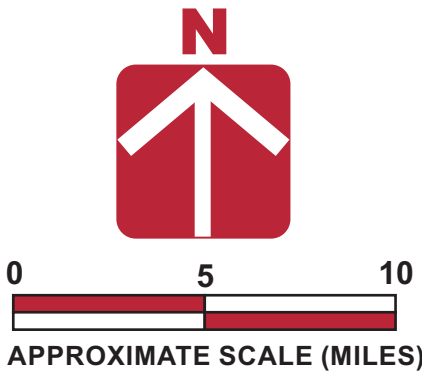
El Camino High School Modernization
1320 Mission Road
South San Francisco, CA

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Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary	Holocene			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Early Quaternary	Pleistocene			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
		11,700			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Quaternary age.
Pre-Quaternary		700,000				
		1,600,000				
		4.5 billion (Age of Earth)				

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



Project Number

1517-1-5

Figure Number

Figure 4

Date

January 2026

Drawn By

RRN

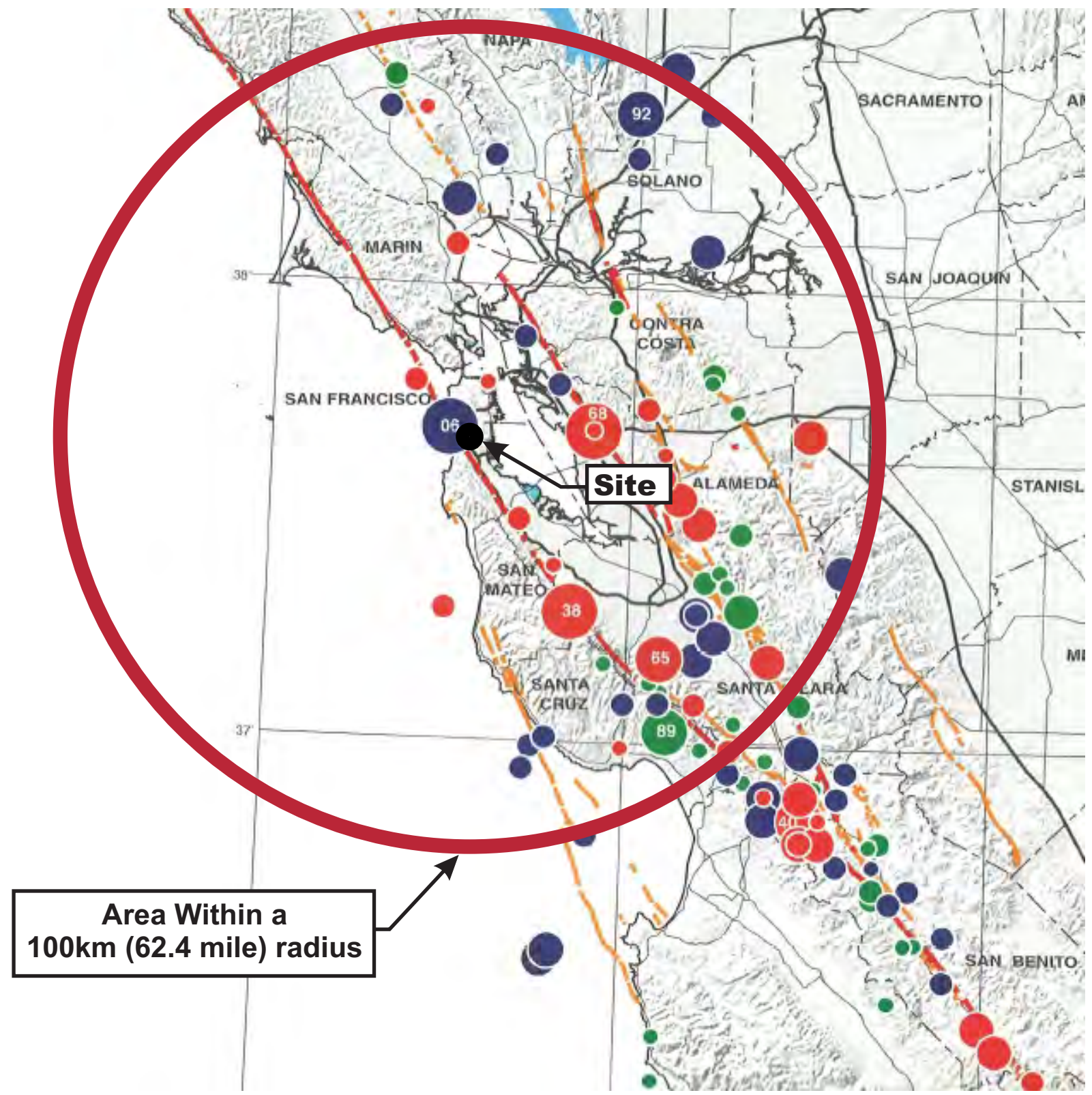
Regional Fault Map

El Camino High School Modernization
1320 Mission Road
South San Francisco, CA



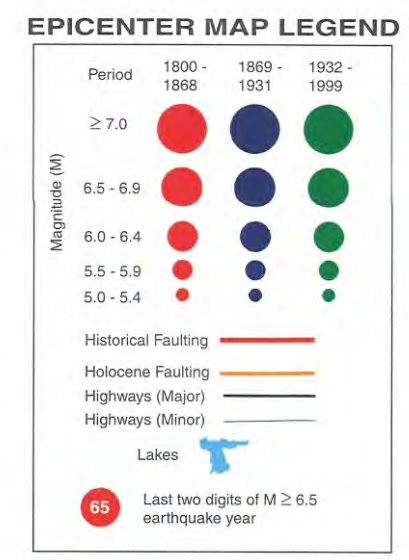
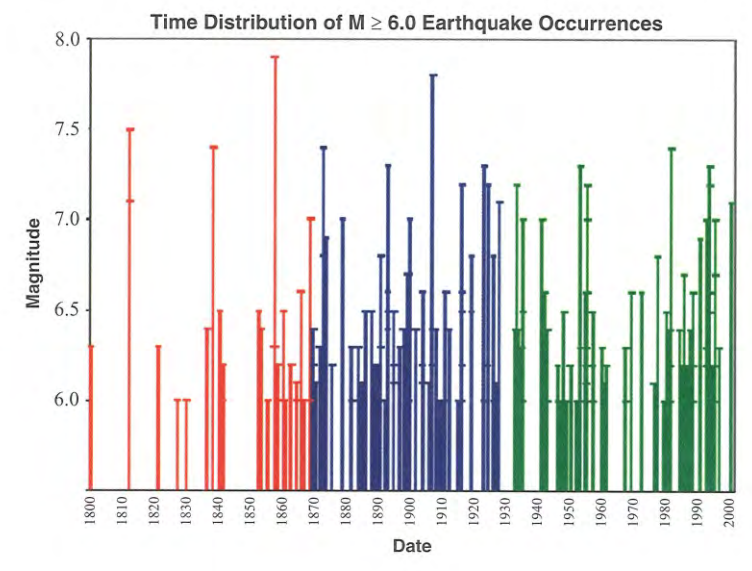
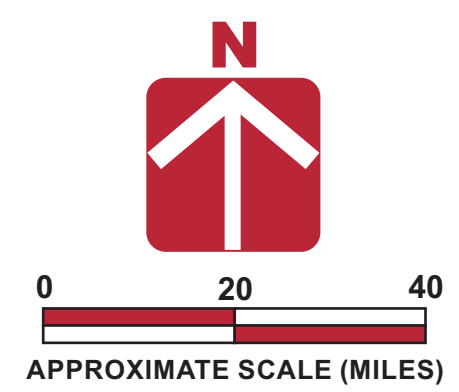
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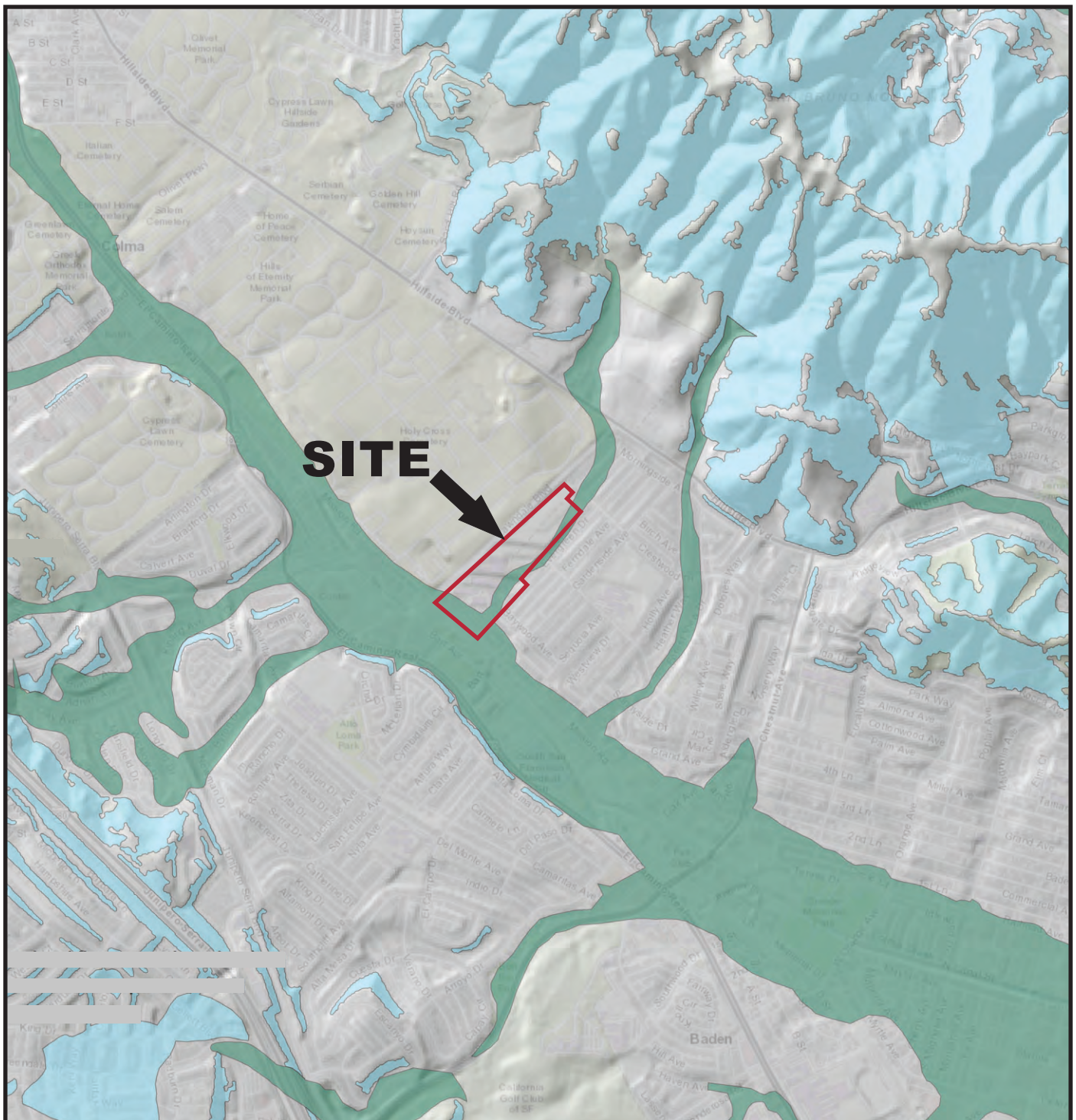
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Area Within a
100km (62.4 mile) radius

From: T. Toppozada & Others (2000)





Explanation

Liquefaction



Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides



Areas where previous occurrence on landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.



0 2,000 4,000

APPROXIMATE SCALE (FEET)

Base: USGS, State of California Seismic Hazard Zones, South San Francisco 7.5-Minute Quadrangle, San Mateo County, California, Released: September 23, 2021



CGS EZRIM Map

El Camino High School Modernization
1320 Mission Road
South San Francisco, CA

Project Number

1517-1-5

Figure Number

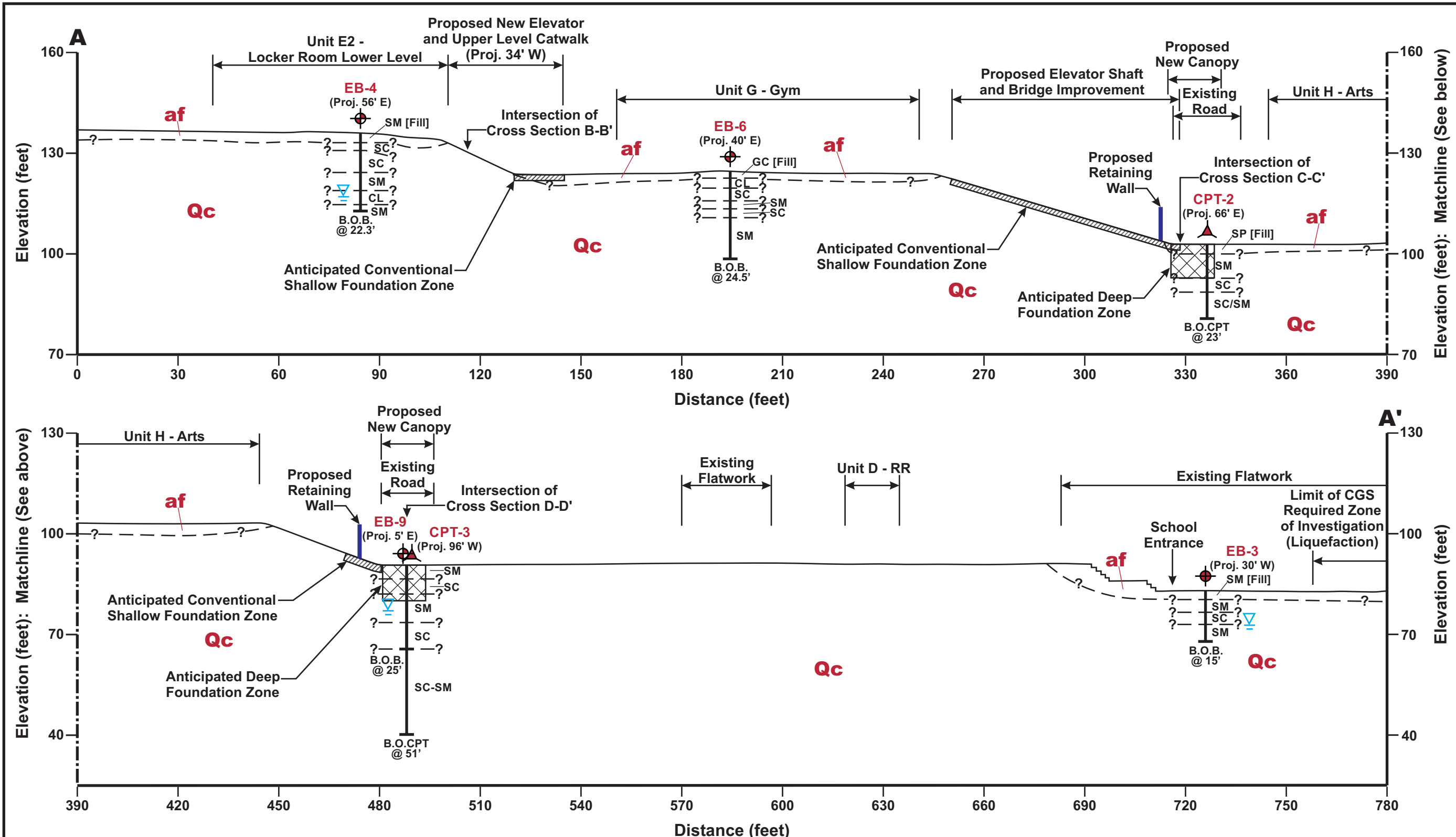
Figure 6

Date

January 2026

Drawn By

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Geologic Units		Symbols	
af	Artificial fill (Holocene)	[Fill]	Artificial fill
Qc	Colma Formation (Pleistocene)	CL	Lean Clay
		SC	Clayey Sand
		SM	Silty Sand
		GC	Clayey Gravel
			Approximate ground water depth at time of drilling; actual depth may vary
			Approximate location of exploratory boring (EB) (Cornerstone, December 2025)
			Approximate location of cone penetration test (CPT) (Cornerstone, December 2025)
			Approximate location of exploratory boring (EB) (Cornerstone, 2024)

Notes:

- 1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 2) 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.
- 3) See Figure 2 for location of cross section.

Project Number

1517-1-5

Figure Number

Figure 7

Date

March 2026

Drawn By

RRN

Geologic Cross Section A-A'

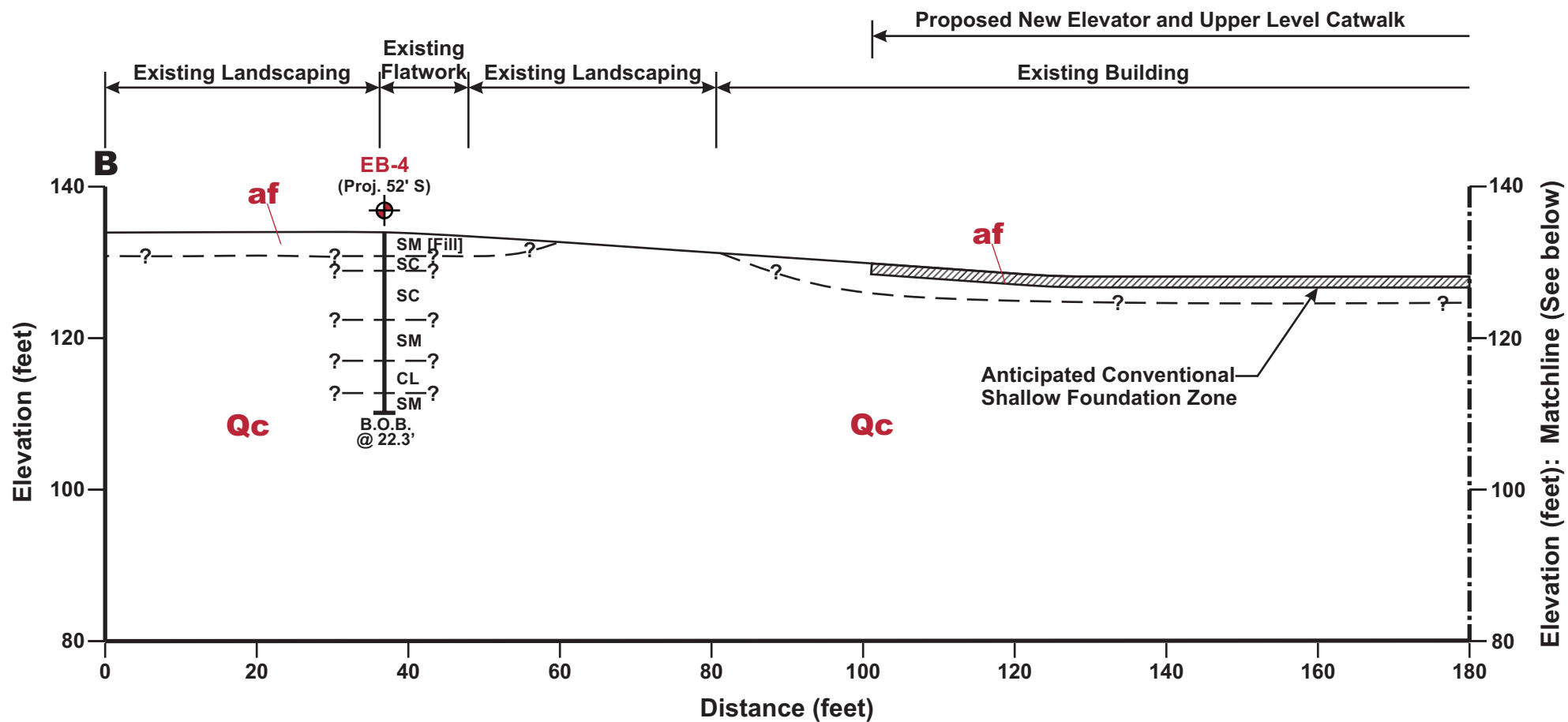
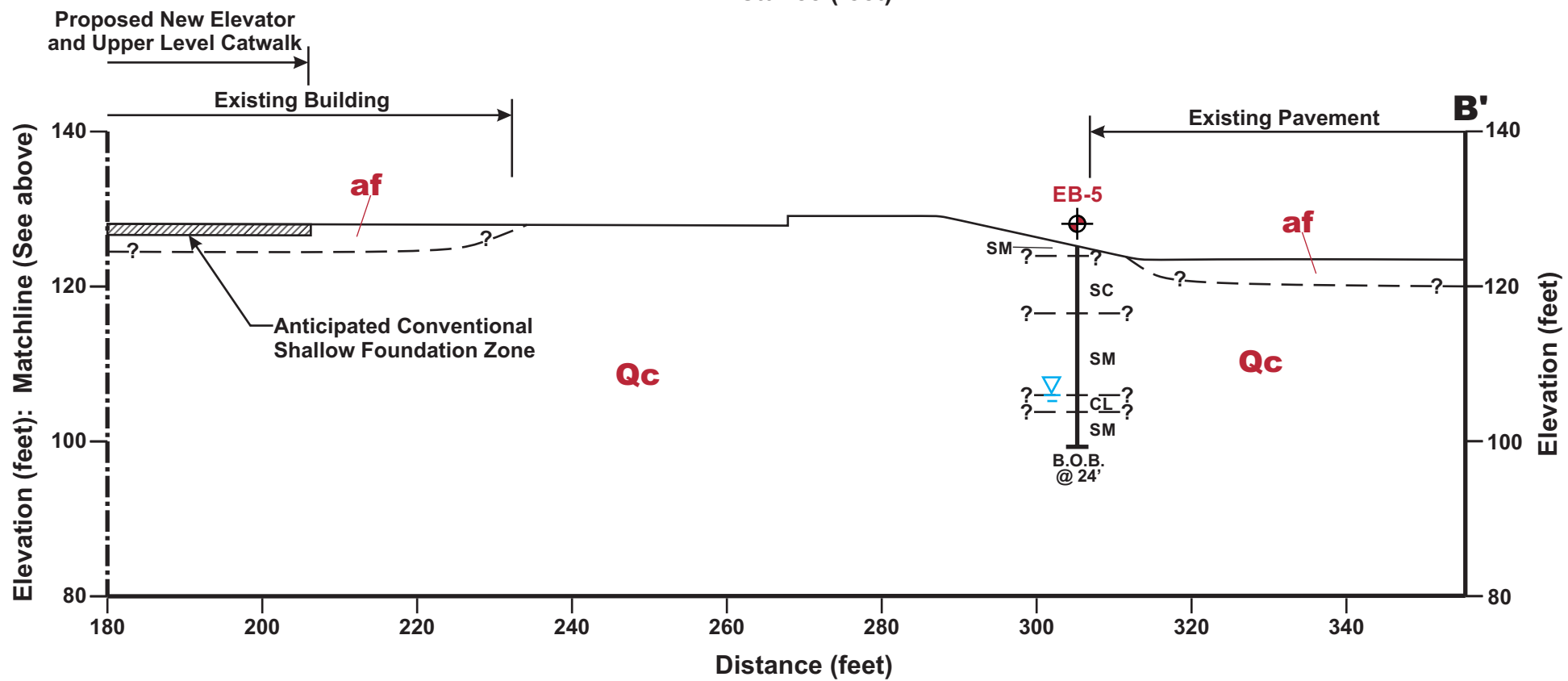
El Camino High School Modernization

1320 Mission Road

South San Francisco, CA

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Section B-B'
(View Looking North)
1"=20' H:V

Explanation

af Artificial fill (Holocene)

Qc Colma Formation (Pleistocene)

Symbols

[Fill] Artificial fill

CL Lean Clay

SC Clayey Sand

SM Silty Sand

Approximate ground water depth at time of drilling; actual depth may vary

Approximate location of exploratory boring (EB) (Cornerstone, December 2025)

Notes:

- 1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 2) 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.
- 3) See Figure 2 for location of cross section.

Project Number
1517-1-5

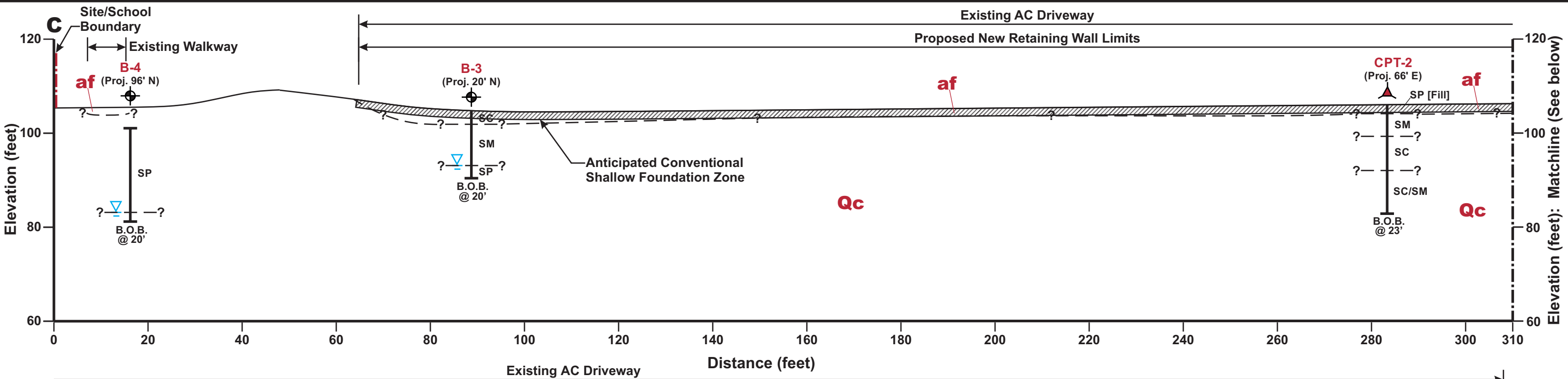
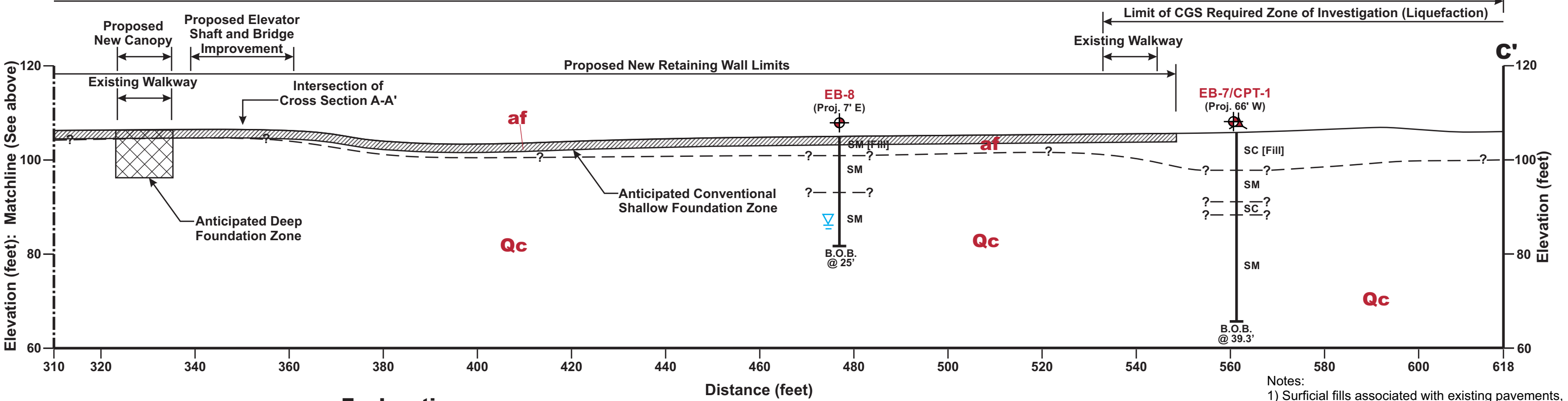
Geologic Cross Section B-B'

El Camino High School Modernization
1320 Mission Road
South San Francisco, CA

Figure Number
Figure 8

Date
March 2026

Drawn By
RRN



Explanation

Symbols

[Fill]	Artificial fill		Approximate ground water depth at time of drilling; actual depth may vary
SC	Clayey Sand		Approximate location of exploratory boring (EB) (Cornerstone, December 2025)
SM	Silty Sand		Approximate location of cone penetration test (CPT) (Cornerstone, December 2025)
SP	Poorly Graded Sand		Approximate location of exploratory boring (EB) (Geosphere, 2012)

Geologic Units

af Artificial fill (Holocene)

Qc Colma Formation (Pleistocene)

Section C-C'
(View Looking North)
1"=20' H:V

Notes:

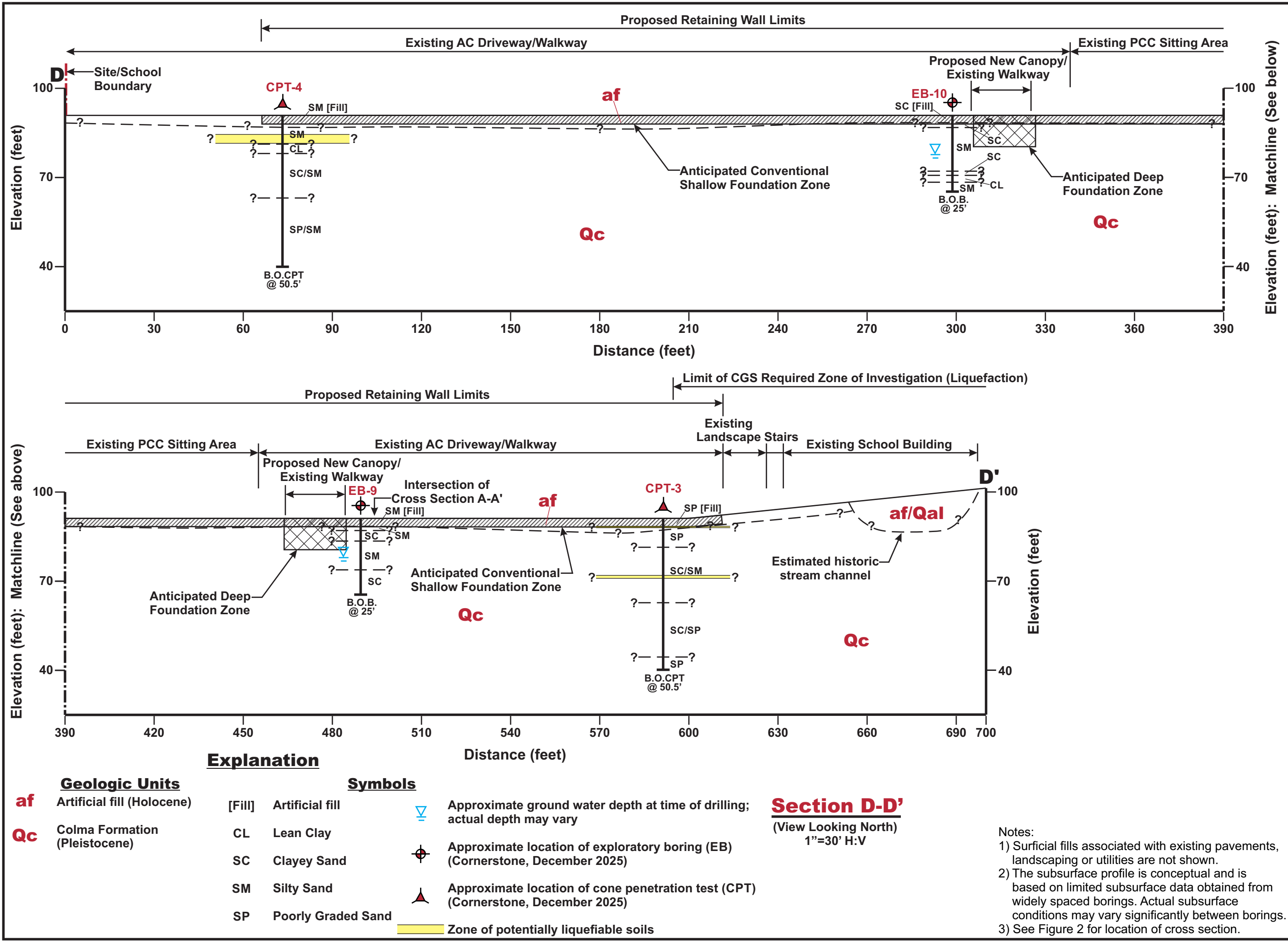
- 1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings
- 3) See Figure 2 for location of cross section.

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Geologic Cross Section C-C'

El Camino High School Modernization
1320 Mission Road
South San Francisco, CA

Project Number 1517-1-5	
Figure Number Figure 9	
Date March 2026	Drawn By RRN



Explanation

Geologic Units		Symbols	
af	Artificial fill (Holocene)	[Fill]	Artificial fill
Qc	Colma Formation (Pleistocene)	CL	Lean Clay
		SC	Clayey Sand
		SM	Silty Sand
		SP	Poorly Graded Sand
			Zone of potentially liquefiable soils

Section D-D'
(View Looking North)
1"=30' H:V

Approximate ground water depth at time of drilling; actual depth may vary

Approximate location of exploratory boring (EB) (Cornerstone, December 2025)

Approximate location of cone penetration test (CPT) (Cornerstone, December 2025)

- Notes:
- 1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
 - 2) 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.
 - 3) See Figure 2 for location of cross section.



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PROJECT/CPT DATA

Project Title **El Camino High School Modernization**

Project No. **1517-1-5**

Project Manager **EA**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.81**

PGA (Amax) **0.95** (g)

SITE SPECIFIC PARAMETERS

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

Design Water Depth (feet) **10**

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

Ave. Unit Weight Below GW (pcf) **125**

FIGURE **11A**

CPT NO. **1**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **10** FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.00 (Inches)

TOTAL SEISMIC SETTLEMENT **0.0** INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **0.0**

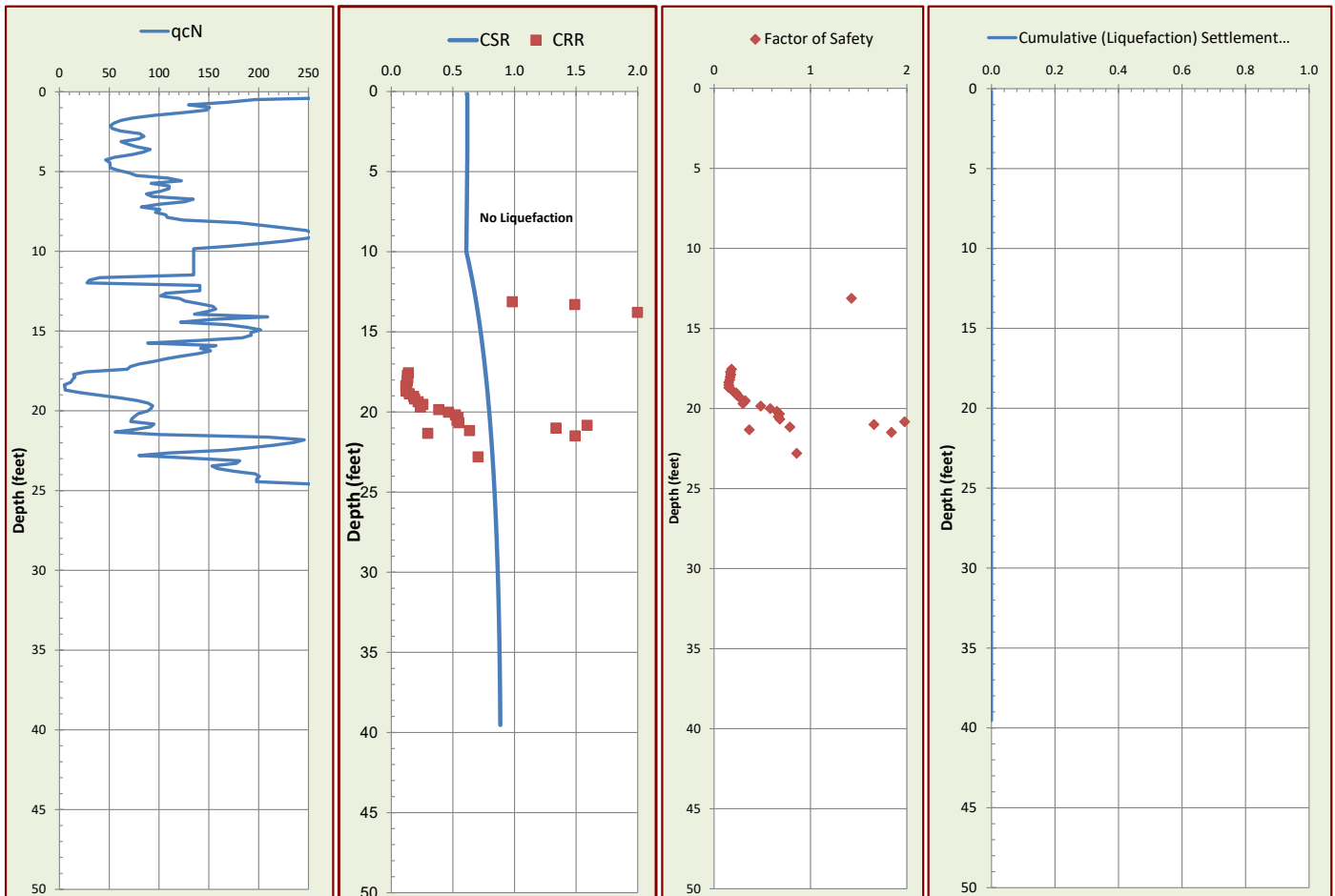
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

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

²LDI Values Only Summed to 2H Below Grade.





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PROJECT/CPT DATA

Project Title **El Camino High School Modernization**

Project No. **1517-1-5**

Project Manager **EA**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.81**

PGA (Amax) **0.95** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **22.4**

Design Water Depth (feet) **2**

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

Ave. Unit Weight Below GW (pcf) **125**

FIGURE **11B**

CPT NO. **2**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **2** FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.00 (Inches)

TOTAL SEISMIC SETTLEMENT **0.0** INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **10.0**

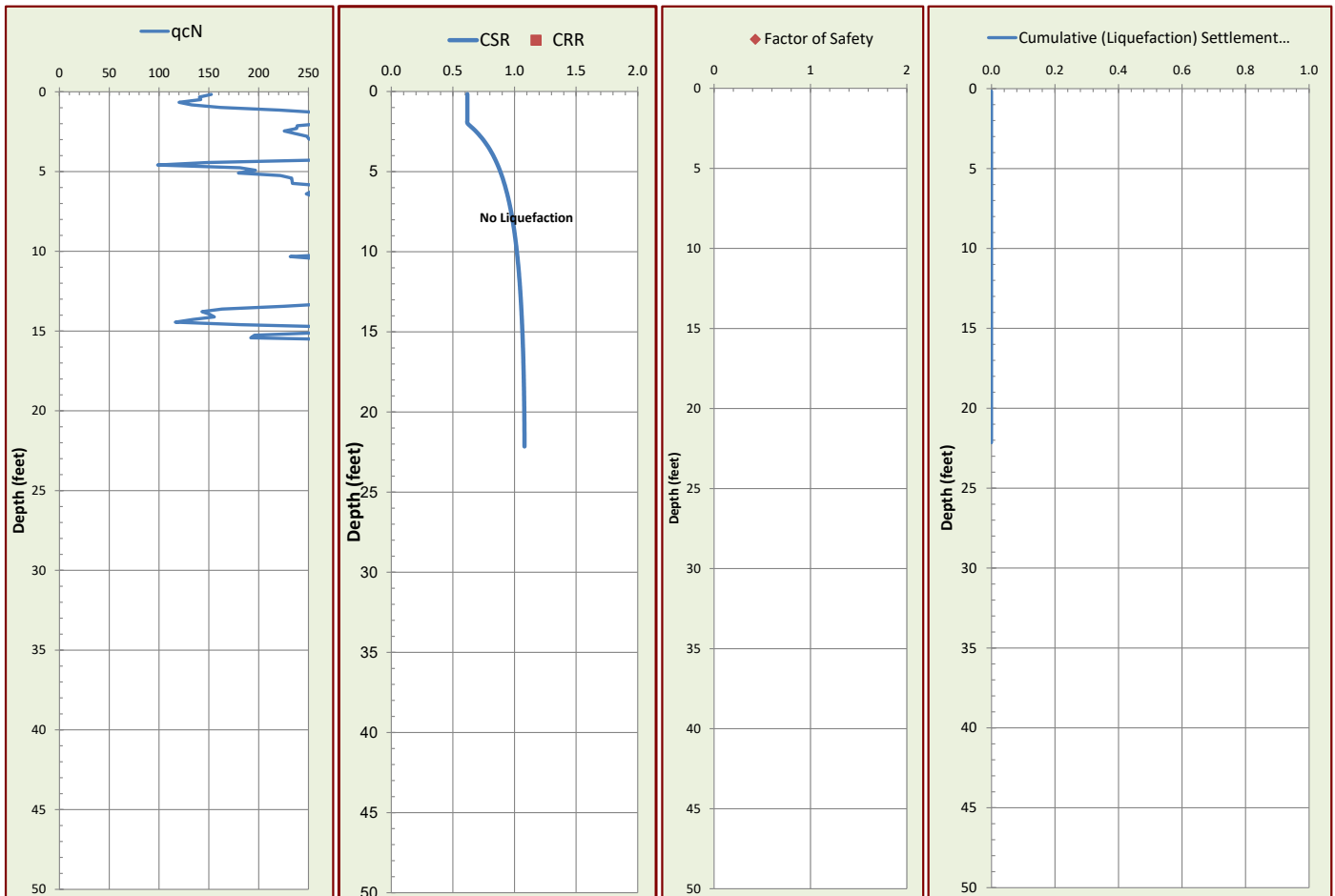
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

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

²LDI Values Only Summed to 2H Below Grade.





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PROJECT/CPT DATA

Project Title **El Camino High School Modernization**

Project No. **1517-1-5**

Project Manager **EA**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.81**

PGA (Amax) **0.95** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **7**

Design Water Depth (feet) **2**

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

Ave. Unit Weight Below GW (pcf) **125**

FIGURE **11C**

CPT NO. **3**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **2** FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.38 (Inches)

TOTAL SEISMIC SETTLEMENT **0.4** INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **0.0**

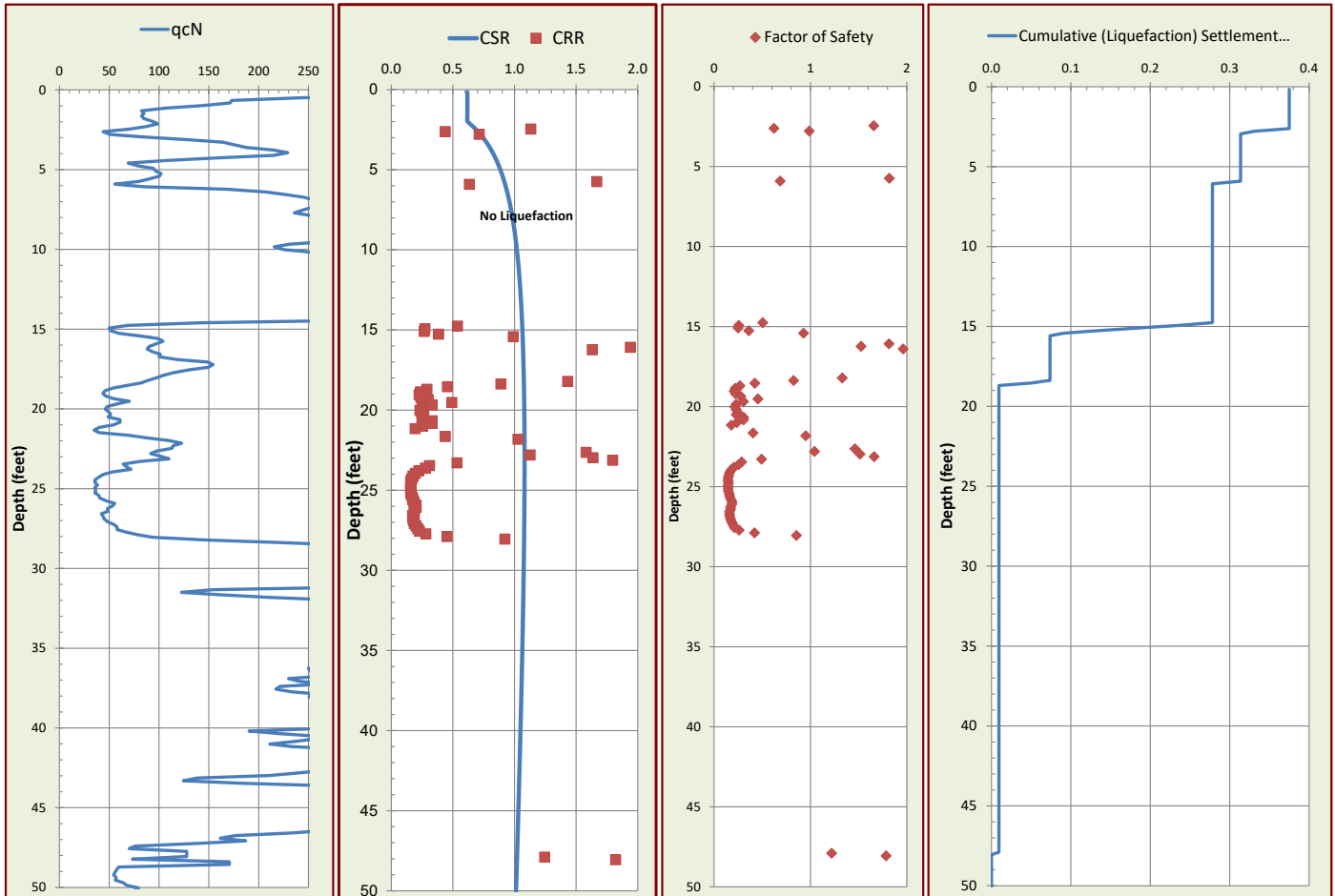
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

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

²LDI Values Only Summed to 2H Below Grade.





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PROJECT/CPT DATA

Project Title **El Camino High School Modernization**

Project No. **1517-1-5**

Project Manager **EA**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.81**

PGA (Amax) **0.95** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **11**

Design Water Depth (feet) **5**

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

Ave. Unit Weight Below GW (pcf) **125**

FIGURE **11D**

CPT NO. **4**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **5** FEET

0.08 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.31 (Inches)

TOTAL SEISMIC SETTLEMENT **0.4** INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **0.0**

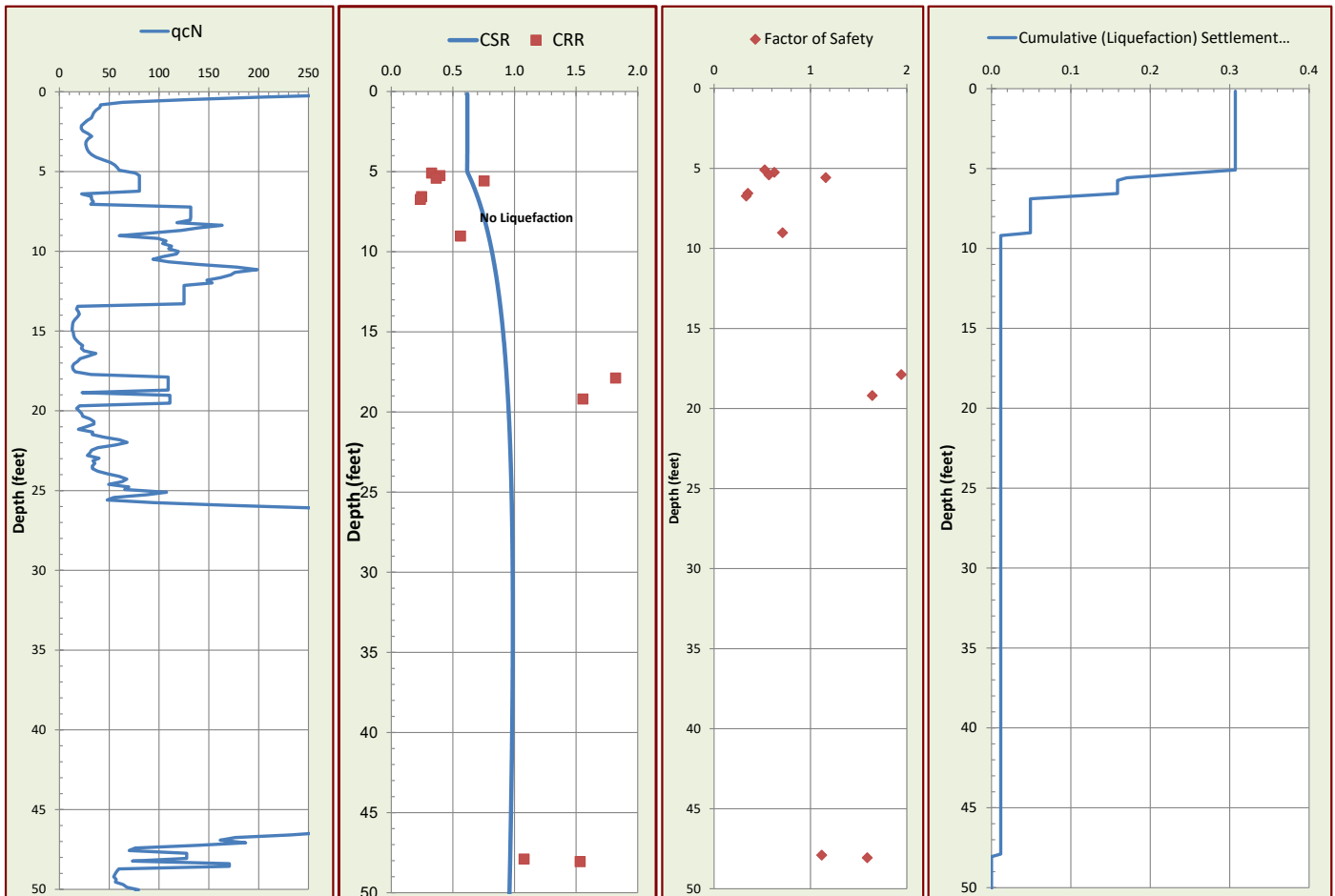
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

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

²LDI 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 and limited access track mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Four 8-inch-diameter exploratory borings using the truck-mounted, hollow-stem auger drilling equipment were drilled on December 30, 2025 to depths of 22 $\frac{1}{3}$ to 39 $\frac{1}{3}$ feet. Two 6 $\frac{1}{2}$ -inch-diameter exploratory borings using the track-mounted, hollow-stem auger drilling equipment were drilled on January 5, 2026 to depths of approximately 25 feet. Four CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on December 29, 2025, to depths ranging from 22 $\frac{2}{3}$ to 59 $\frac{1}{2}$ feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring and CPT elevations were based on google earth (WGS84 Datum). 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. 2-inch O.D. split-spoon samplers were 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.

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

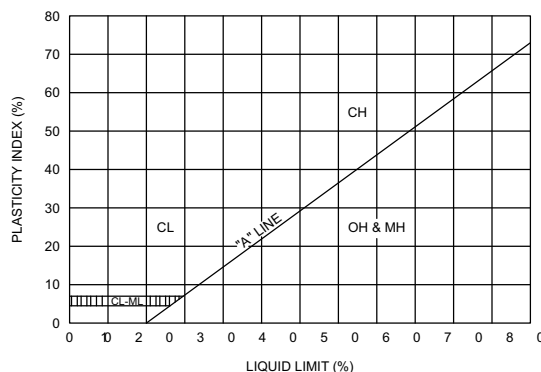
SAMPLER TYPES

	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

BORING NUMBER EB-4

PAGE 1 OF 1

PROJECT NAME El Camino High School ModernizationPROJECT NUMBER 1517-1-5PROJECT LOCATION South San Francisco, CADATE STARTED 12/30/25 DATE COMPLETED 12/30/25DRILLING CONTRACTOR Exploration Geoservices, Inc.DRILLING METHOD Mobile B-53R, 8 inch Hollow-Stem AugerLOGGED BY BTM

NOTES _____

GROUND ELEVATION 134 FT +/- BORING DEPTH 22.3 ft.LATITUDE 37.666986° LONGITUDE -122.442263°**GROUND WATER LEVELS:**▼ AT TIME OF DRILLING Not Encountered▼ 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	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				
134.0	0		Silty Sand (SM) [Fill] loose, moist, brown with dark brown mottles, fine to medium sand	15	MC-1B	107	9							
131.5	5		Clayey Sand (SC) [Qc] medium dense, moist, light gray and light brown mottled, fine to medium sand	19	MC-2B	108	17		20					
				44	MC-3	106	18		21					
			color changes to light reddish brown	50	MC-4B	110	15							
122.5	15		Silty Sand (SM) [Qc] very dense, moist, light gray brown fine to medium sand	56	SPT-5		14							
117.0	20		Sandy Lean Clay (CL) [Qc] very stiff, moist, brown with gray mottles, fine to medium sand, low plasticity	43	MC-6B	111	19							
113.0			Silty Sand (SM) [Qc] very dense, moist, light reddish brown, fine to medium sand	50	MC-7B	105	11							
111.7			Bottom of Boring at 22.3 feet.	4"										
	25													



CORNERSTONE EARTH GROUP

BORING NUMBER EB-5

PAGE 1 OF 1

PROJECT NAME El Camino High School Modernization

PROJECT NUMBER 1517-1-5

PROJECT LOCATION South San Francisco, CA

DATE STARTED 12/30/25 DATE COMPLETED 12/30/25

DRILLING CONTRACTOR Exploration Geoservices, Inc.

DRILLING METHOD Mobile B-53R, 8 inch Hollow-Stem Auger

LOGGED BY BTM

NOTES

GROUND ELEVATION 125 FT +/- BORING DEPTH 24 ft.

LATITUDE 37.666827° LONGITUDE -122.441341°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 19 ft.

▼ AT END OF DRILLING 19 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				
125.0	0		2 inches asphalt concrete over 5 inches aggregate base											
124.4			Silty Sand (SM) [Qc] dense, moist, light reddish brown, fine to medium sand	44	MC-1	105	12							
122.5			Clayey Sand (SC) [Qc] medium dense, moist, light brown with gray mottles, fine to medium sand	46	MC-2	114	16		28					
119.5	5		Sandy Lean Clay (CL) [Qc] very stiff, moist, brown with gray mottles, fine to medium sand, low plasticity	44	MC-3B	116	16							
117.0			Silty Sand (SM) [Qc] medium dense, moist, light reddish brown, fine to medium sand	46	MC-4B	102	9							
	10		becomes very dense	50 5"	SPT-5		6							
	15			50 6"	MC-6B	111	16							
106.3			Sandy Lean Clay (CL) [Qc] very stiff, moist, gray with brown mottles, fine to medium sand, low plasticity	44	7A SPT 7B		28 21							
105.3	20		Silty Sand (SM) [Qc] very dense, moist, light gray brown, fine to medium sand	50 5"	MC-8B	110	15							
				50 6"	SPT-9		12							
101.0	25		Bottom of Boring at 24.0 feet.											



CORNERSTONE EARTH GROUP

BORING NUMBER EB-6

PAGE 1 OF 1

PROJECT NAME El Camino High School Modernization

PROJECT NUMBER 1517-1-5

PROJECT LOCATION South San Francisco, CA

DATE STARTED 12/30/25 DATE COMPLETED 12/30/25

DRILLING CONTRACTOR Exploration Geoservices, Inc.

DRILLING METHOD Mobile B-53R, 8 inch Hollow-Stem Auger

LOGGED BY BTM

NOTES _____

GROUND ELEVATION 124 FT +/- BORING DEPTH 24.5 ft.

LATITUDE 37.666693° LONGITUDE -122.442243°

GROUND WATER LEVELS:

▼ AT TIME OF DRILLING Not Encountered

▼ AT END OF DRILLING Not Encountered

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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				
124.0	0		2 inches asphalt concrete							○ HAND PENETROMETER				
123.0			Clayey Gravel with Sand (GC) [Fill] dense, moist, dark gray, fine subangular gravel	50	MC-1	113	16	15						
121.0			Sandy Lean Clay (CL) [Qc] hard, moist, gray with reddish brown mottles, fine sand, low plasticity Liquid Limit = 27, Plastic Limit = 12	61	MC-2B	105	15							
118.0	5		Clayey Sand (SC) [Qc] dense, moist, reddish brown with gray mottles, fine to medium sand	62	MC-3	107	10							
116.0			Silty Sand (SM) [Qc] dense, moist, light brown, fine to medium sand											
113.5	10		Clayey Sand (SC) [Qc] medium dense, moist, gray with brown mottles, fine to medium sand	20	SPT-4		13		36					
			Silty Sand (SM) [Qc] very dense, moist, light gray brown, fine to medium sand	66	SPT-5		11							
	15			50 5"	MC-6B	103	13							
				50 6"	SPT-7		7							
	20			50 6"	SPT-8		9							
99.5	25		Bottom of Boring at 24.5 feet.											



CORNERSTONE EARTH GROUP

BORING NUMBER EB-7

PAGE 1 OF 2

DATE STARTED 12/30/25 DATE COMPLETED 12/30/25

DRILLING CONTRACTOR Exploration Geoservices, Inc.

DRILLING METHOD Mobile B-53R, 8 inch Hollow-Stem Auger

LOGGED BY BTM

NOTES

PROJECT NAME El Camino High School Modernization

PROJECT NUMBER 1517-1-5

PROJECT LOCATION South San Francisco, CA

GROUND ELEVATION 123 FT +/- BORING DEPTH 39.3 ft.

LATITUDE 37.666443° LONGITUDE -122.441404°

GROUND WATER LEVELS:

AT TIME OF DRILLING Not Encountered

AT END OF DRILLING Not Encountered

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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
123.0	0		2 inches asphalt concrete over 5 inches aggregate base							
122.4			Clayey Sand (SC) [Fill] medium dense, moist, light reddish brown and gray mottled, fine to medium sand	21	MC-1B	114	13			
				38	MC-2B	118	12			
	5			55	MC-3B	121	12			
115.0			Silty Sand (SM) [Qc] medium dense, moist, brown and dark brown mottled, fine to medium sand	33	MC-4B	120	10			
	10									
				52	MC-5B	113	15			
108.0	15		Clayey Sand (SC) [Qc] medium dense, moist, reddish brown with gray mottles, fine sand	28	SPT-6		12		22	
105.5			Clayey Sand (SC) [Qc] dense, moist, reddish brown with gray mottles, fine sand	42	SPT-7		9			
	20			40	SPT-8		18		25	
101.0			Silty Sand (SM) [Qc] very dense, moist, gray with reddish brown mottles, fine to medium sand	57	SPT-9		11			
	25									
97.0										

Continued Next Page



CORNERSTONE EARTH GROUP

BORING NUMBER EB-8

PAGE 1 OF 1

PROJECT NAME El Camino High School Modernization

PROJECT NUMBER 1517-1-5

PROJECT LOCATION South San Francisco, CA

DATE STARTED 1/5/26 DATE COMPLETED 1/5/26

DRILLING CONTRACTOR Cuesta Geo

DRILLING METHOD MPP Track Rig, 6½ inch Hollow-Stem Auger

LOGGED BY BTM

NOTES _____

GROUND ELEVATION 103 FT +/- BORING DEPTH 25 ft.

LATITUDE 37.666267° LONGITUDE -122.441707°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 21.5 ft.

▼ AT END OF DRILLING 21.5 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				
103.8	0		2 inches asphalt concrete over 10 inches aggregate base											
102.0			Silty Sand (SM) [Fill] loose to medium dense, moist, reddish brown, fine to medium sand	17	MC-1B	112	11							
99.5			Silty Sand (SM) [Qc] medium dense to dense, moist, light reddish brown, fine to medium sand	42	MC-2B	114	12							
	5			61	MC-3B	113	13							
				30	SPT-4		14							
				43	MC-5B	106	18							
91.0			Silty Sand (SM) [Qc] very dense, moist, gray brown, fine to medium sand	59	SPT-6		12							
	15													
			becomes dense	67	MC-7B	107	20							
				46	SPT-8		23							
78.0	25		Bottom of Boring at 25.0 feet.											



CORNERSTONE EARTH GROUP

BORING NUMBER EB-9

PAGE 1 OF 1

DATE STARTED 12/30/25 DATE COMPLETED 12/30/25

DRILLING CONTRACTOR Exploration Geoservices, Inc.

DRILLING METHOD Mobile B-53R, 8 inch Hollow-Stem Auger

LOGGED BY BTM

NOTES

PROJECT NAME El Camino High School Modernization

PROJECT NUMBER 1517-1-5

PROJECT LOCATION South San Francisco, CA

GROUND ELEVATION 91 FT +/- BORING DEPTH 25 ft.

LATITUDE 37.665895° LONGITUDE -122.442208°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 13.5 ft.

▼ AT END OF DRILLING 13 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				
91.0	0		2 inches asphalt concrete over 4 inches aggregate base											
90.5			Silty Sand (SM) [Qc] medium dense, moist, reddish brown with gray mottles, fine to medium sand	40	MC-1B	106	17		16					
			increasing silt fines NP = Nonplastic	26	MC-2B	96	28	NP						
86.5	5		Clayey Sand (SC) [Qc] medium dense, moist, reddish brown with gray mottles, fine to medium sand	34	MC-3B	106	21							
83.5			Silty Sand (SM) [Qc] dense, moist, brown, fine to medium sand	38	SPT-4		21							
	10													
			becomes very dense	50 5"	SPT-5		21							
	15													
73.5			Clayey Sand (SC) [Qc] very dense, moist, reddish brown to brown, fine sand	50 6"	MC-6B	108	20							
	20													
	25		Bottom of Boring at 25.0 feet.	74	SPT-7		15							



CORNERSTONE EARTH GROUP

BORING NUMBER EB-10

PAGE 1 OF 1

PROJECT NAME El Camino High School ModernizationPROJECT NUMBER 1517-1-5PROJECT LOCATION South San Francisco, CADATE STARTED 1/5/26 DATE COMPLETED 1/5/26DRILLING CONTRACTOR Cuesta GeoDRILLING METHOD MPP Track Rig, 6½ inch Hollow-Stem AugerLOGGED BY BTM

NOTES _____

GROUND ELEVATION 91 FT +/- BORING DEPTH 25 ft.LATITUDE 37.665744° LONGITUDE -122.442865°**GROUND WATER LEVELS:**▽ AT TIME OF DRILLING 13.5 ft.▼ AT END OF DRILLING 20 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				
91.0	0		2 inches asphalt concrete over 8 inches aggregate base							○ HAND PENETROMETER				
90.1			Clayey Sand (SC) [Qc]							△ TORVANE				
			medium dense, moist, reddish brown with gray mottles, fine to medium sand	30	MC-1B	105	16			● UNCONFINED COMPRESSION				
				45	MC-2B	107	16			▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL				
87.0			Silty Sand (SM) [Qc]							1.0 2.0 3.0 4.0				
			medium dense, moist, gray with reddish brown mottles, fine to medium sand	51	MC-3B	107	19							
				27	SPT-4		21							
				52	MC-5B	109	20							
			becomes dense	33	SPT-6		22							
72.0			Clayey Sand (SC) [Qc]											
			dense, moist, brown, fine to medium sand	48	MC-7B	114	17							
70.5			Sandy Lean Clay (CL) [Qc]											
			hard, moist, grayish brown, fine to medium sand, low plasticity	81	SPT-B		20							
68.5			Silty Sand (SM) [Qc]											
			dense, moist, brown, fine to medium sand	45	SPT-9		21							
66.0	25		Bottom of Boring at 25.0 feet.											

>4.5

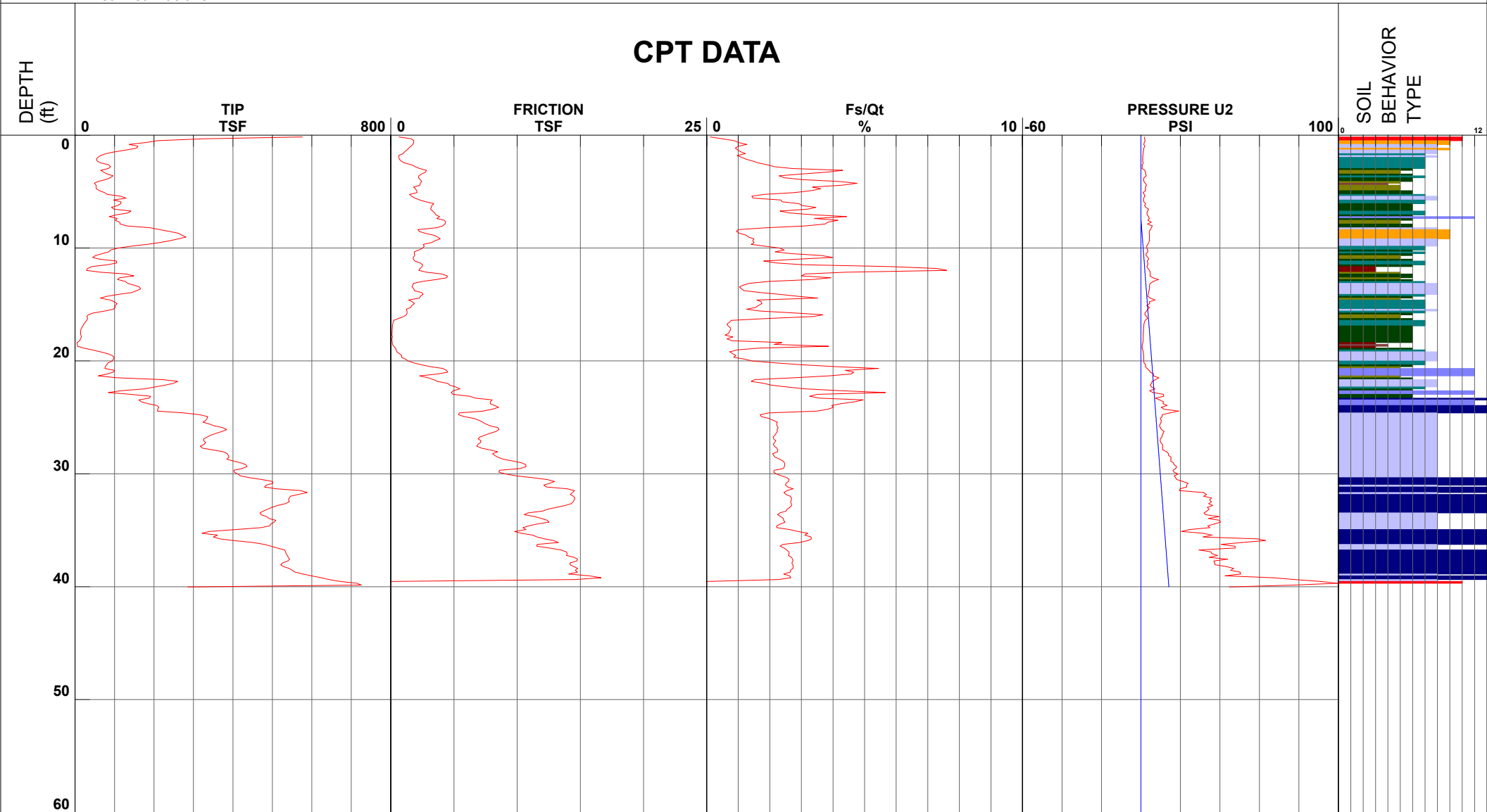


Cornerstone Earth Group

Project El Camino High School Modernization Operator JM - SS
Job Number 1517-1-5 Cone Number DDG1587
Hole Number CPT-01 Date and Time 12/29/2025 9:02:59 AM
EST GW Depth During Test 7.47 ft

Filename SDF(079).cpt
GPS
Maximum Depth 40.03 ft

Net Area Ratio .8



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

Cone Size 15cm²

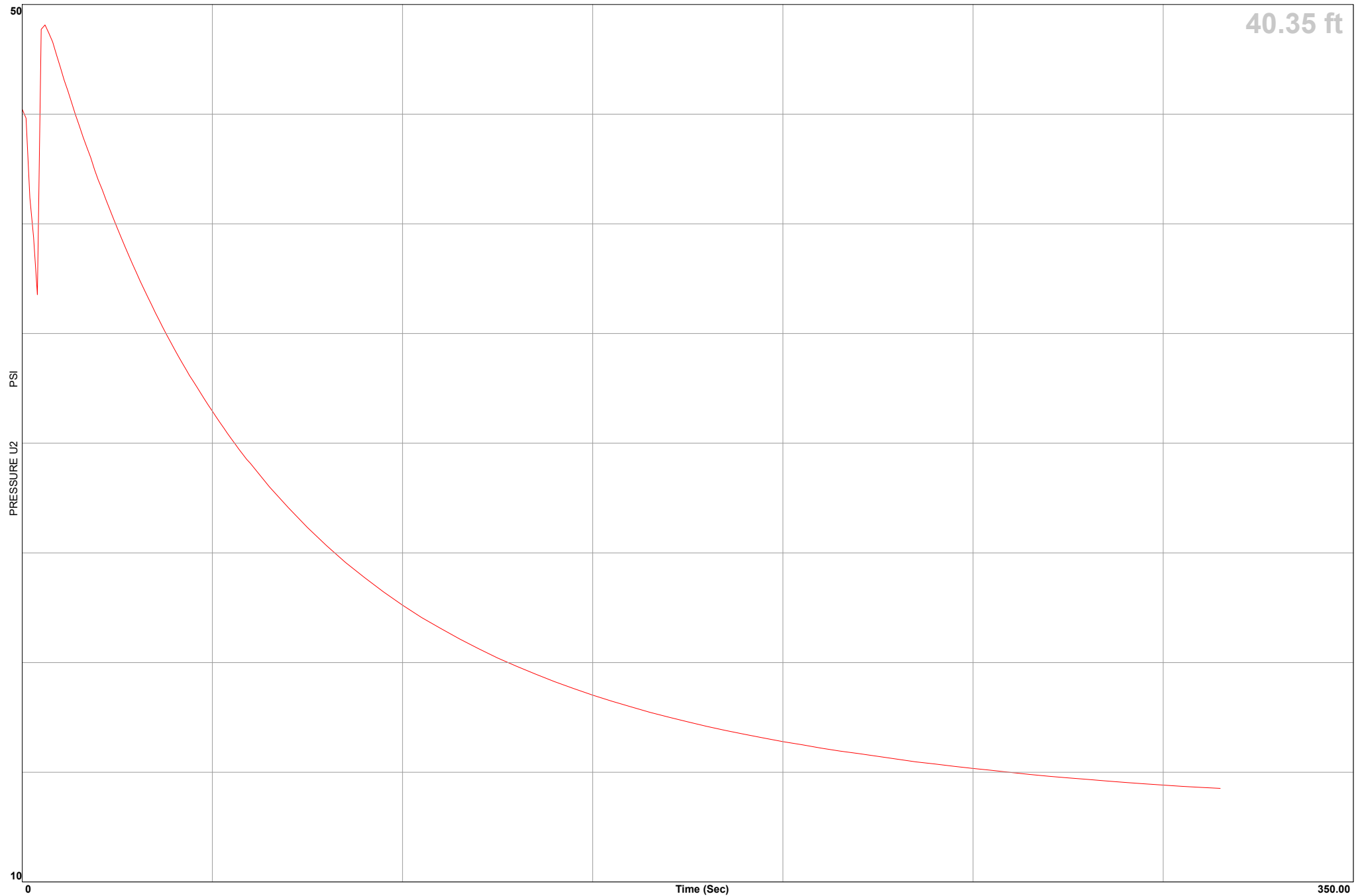
S²Soil behavior type and SPT based on data from UBC-1983



Cornerstone Earth Group

Location	El Camino High School Modernization		Operator	JM - SS
Job Number	1517-1-5		Cone Number	El Camino High School Modernization
Hole Number	CPT-01		Date and Time	12/29/2025 9:02:59 AM
Equilized Pressure	14.2		EST GW Depth During Test	7.4

GPS _____



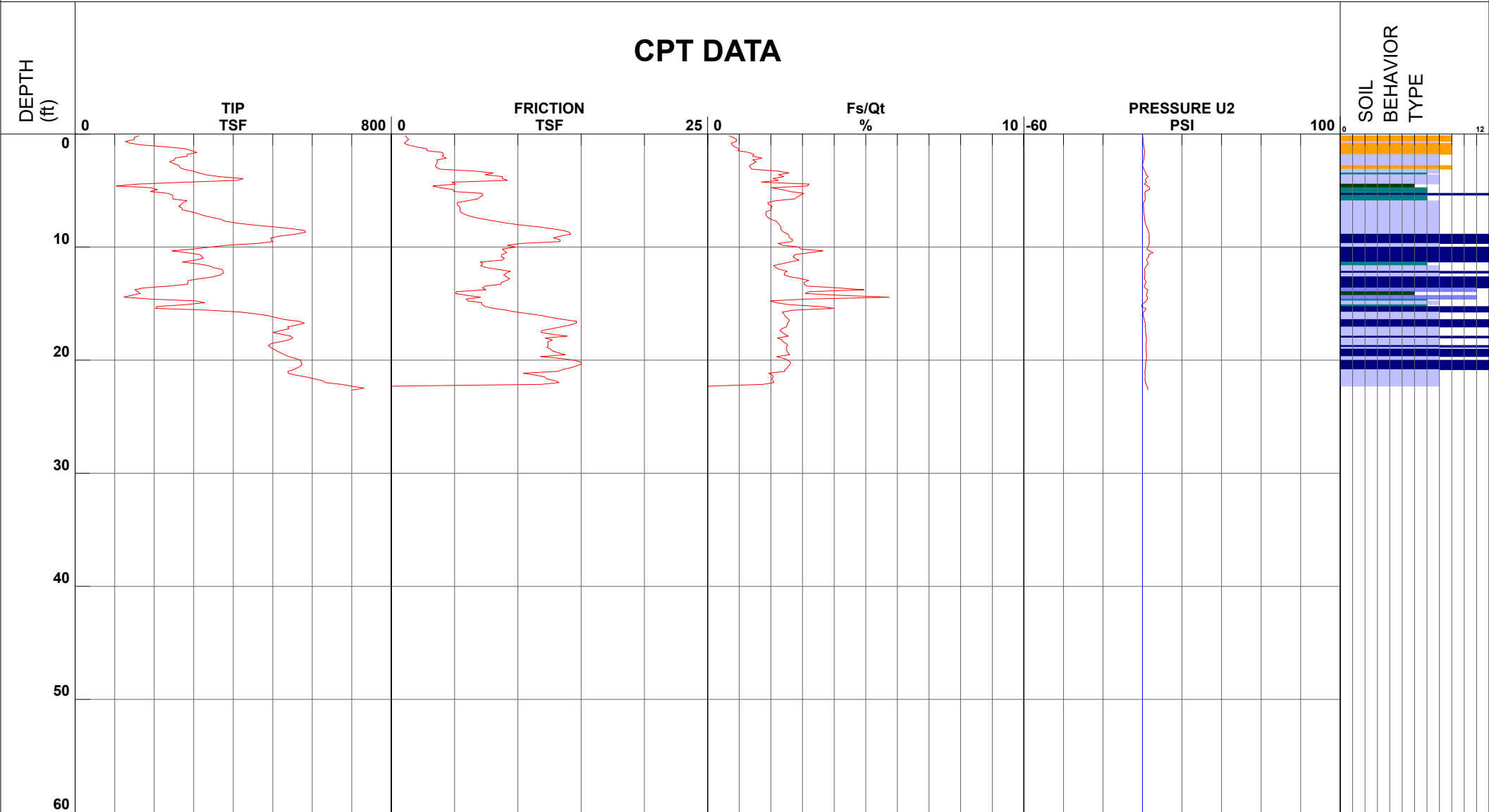


Cornerstone Earth Group

Project El Camino High School Modernization Operator JM - SS
Job Number 1517-1-5 Cone Number DDG1587
Hole Number CPT-02 Date and Time 12/29/2025 10:39:23 AM
EST GW Depth During Test 23.00 ft

Filename SDF(081).cpt
GPS
Maximum Depth 22.64 ft

Net Area Ratio .8



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

Cone Size 15cm²

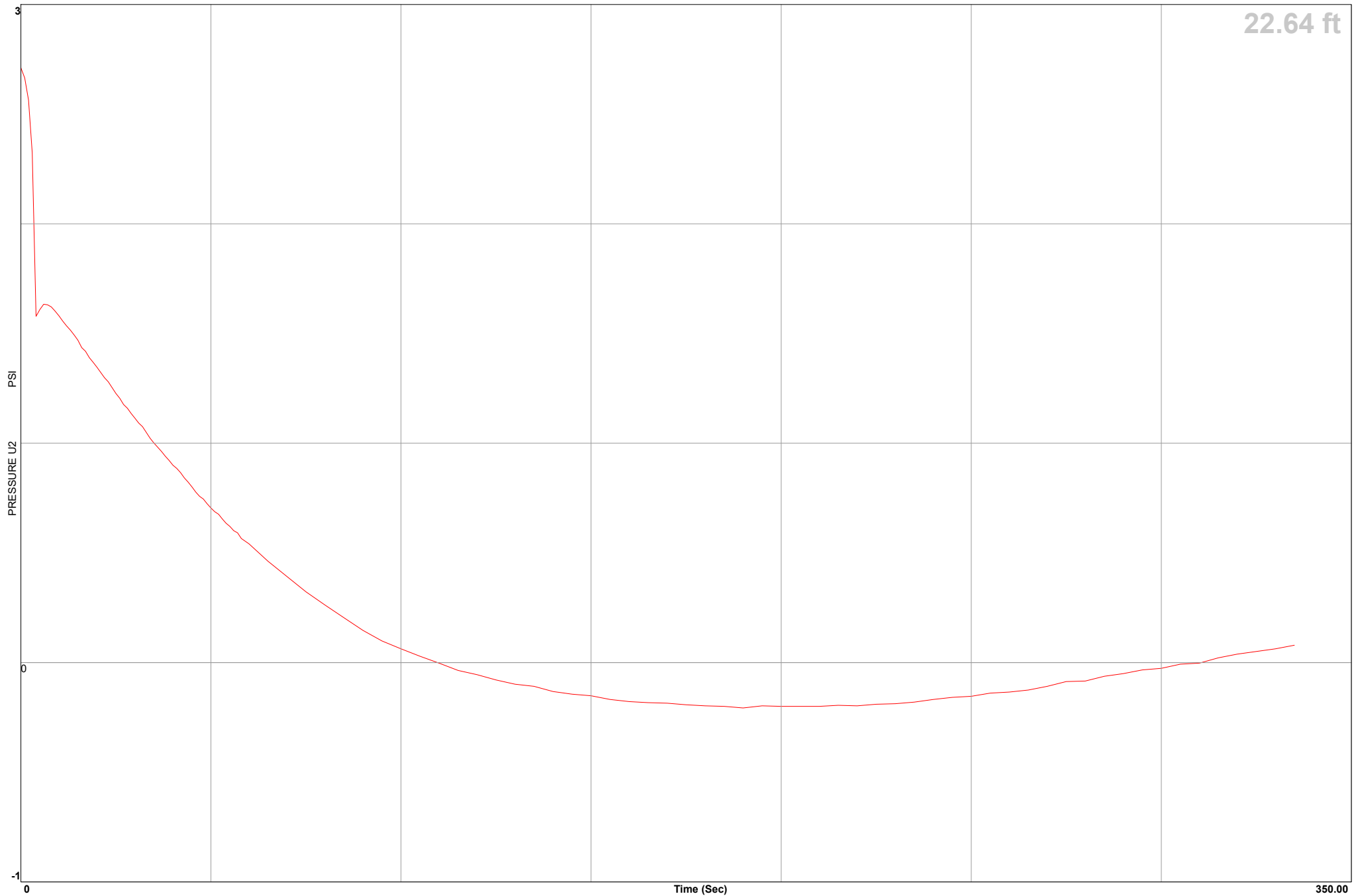
S*Soil behavior type and SPT based on data from UBC-1983



Cornerstone Earth Group

Location	El Camino High School Modernization	Operator	JM - SS
Job Number	1517-1-5	Cone Number	El Camino High School Modernization
Hole Number	CPT-02	Date and Time	12/29/2025 10:39:23 AM
Equilized Pressure	.0	EST GW Depth During Test	22.4

GPS _____



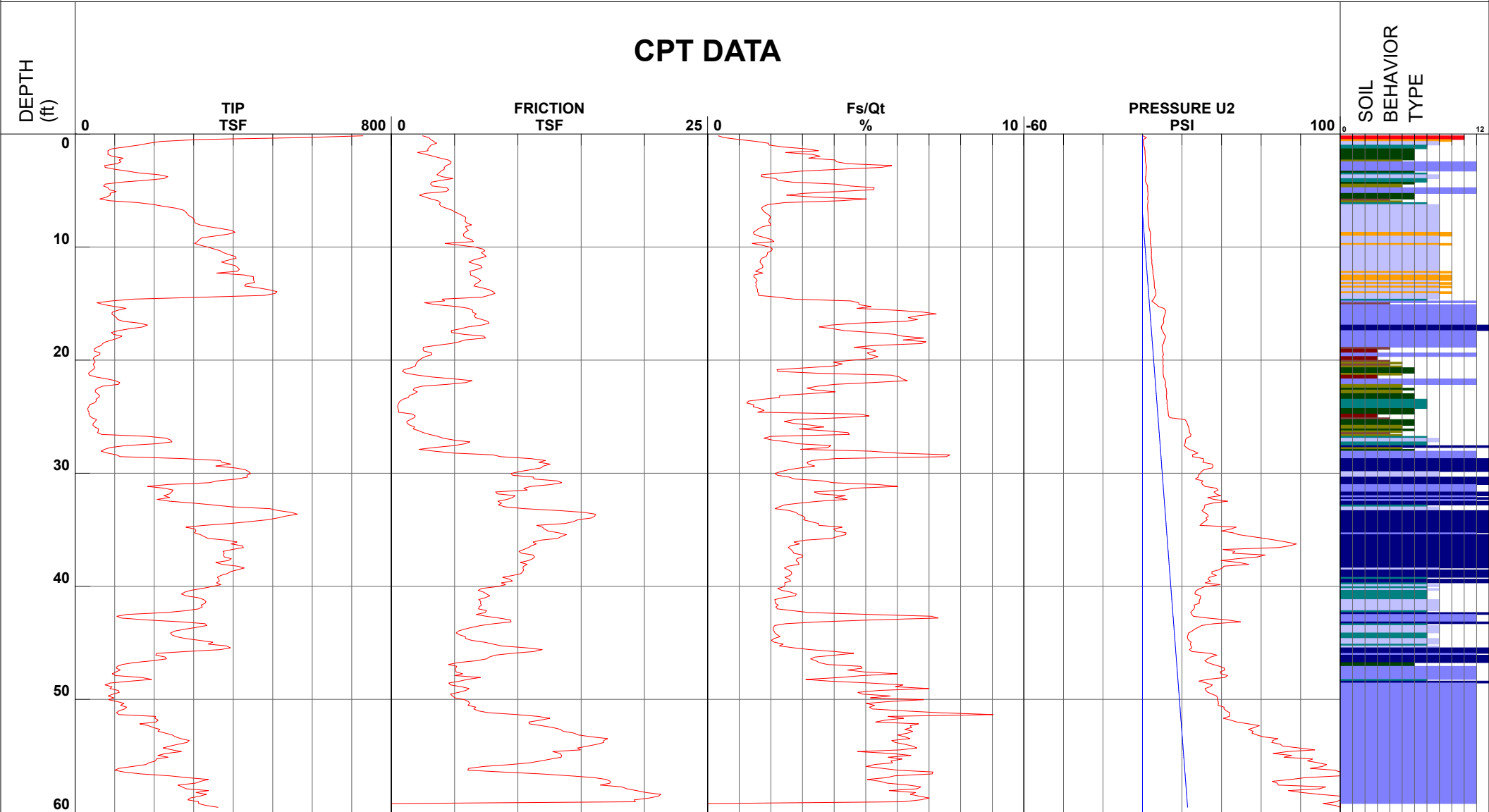


Cornerstone Earth Group

Project El Camino High School Modernization Operator JM - SS
Job Number 1517-1-5 Cone Number DDG1587
Hole Number CPT-03A Date and Time 12/29/2025 11:19:15 AM
EST GW Depth During Test 7.00 ft

Filename SDF(082).cpt
GPS
Maximum Depth 59.55 ft

Net Area Ratio .8



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

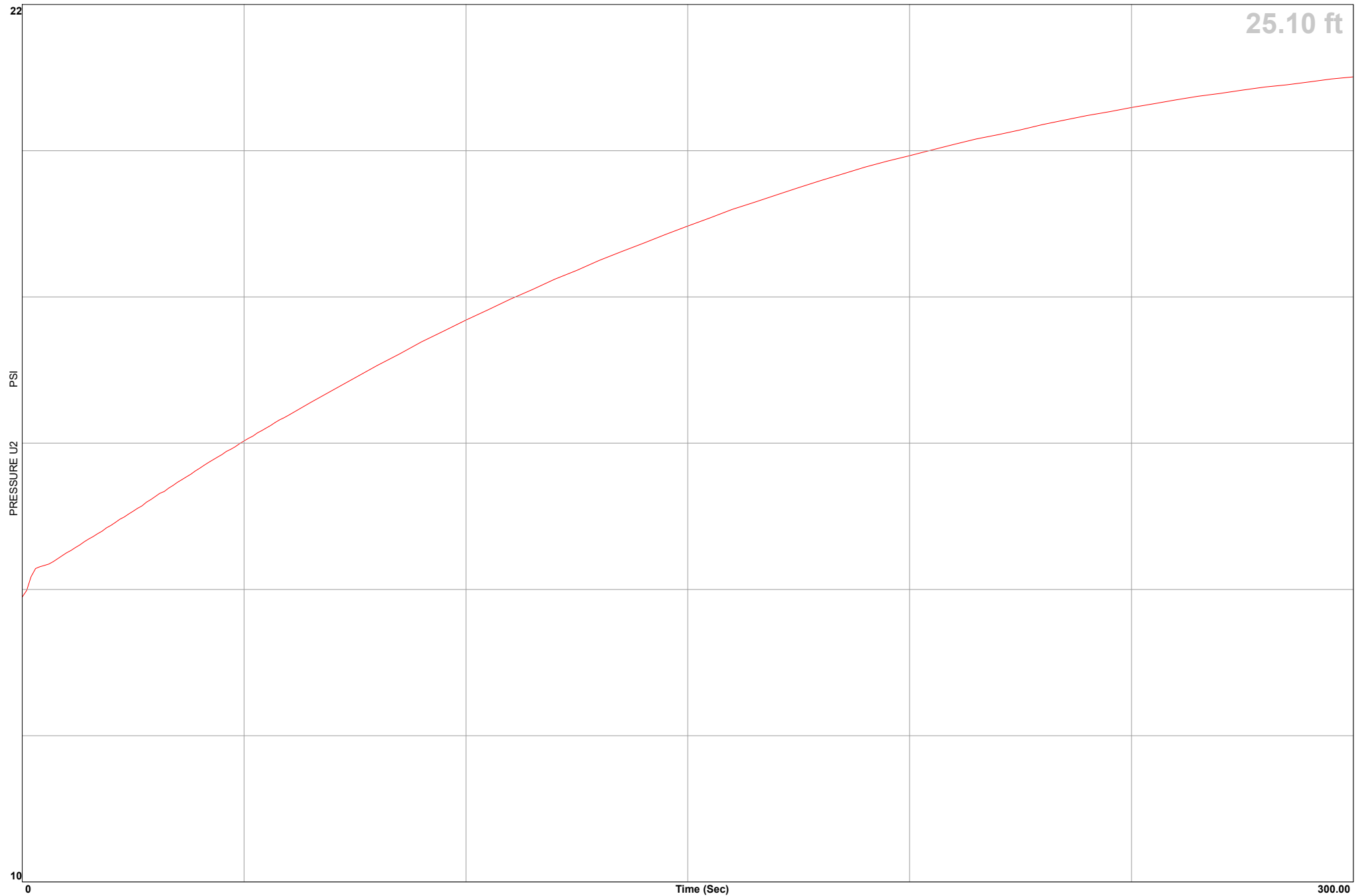
Cone Size 15cm²

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



Cornerstone Earth Group

Location	El Camino High School Modernization		Operator	JM - SS	
Job Number	1517-1-5		Cone Number	El Camino High School Modernization	GPS
Hole Number	CPT-03A		Date and Time	12/29/2025 11:19:15 AM	
Equilized Pressure	21.0		EST GW Depth During Test	+23.4	



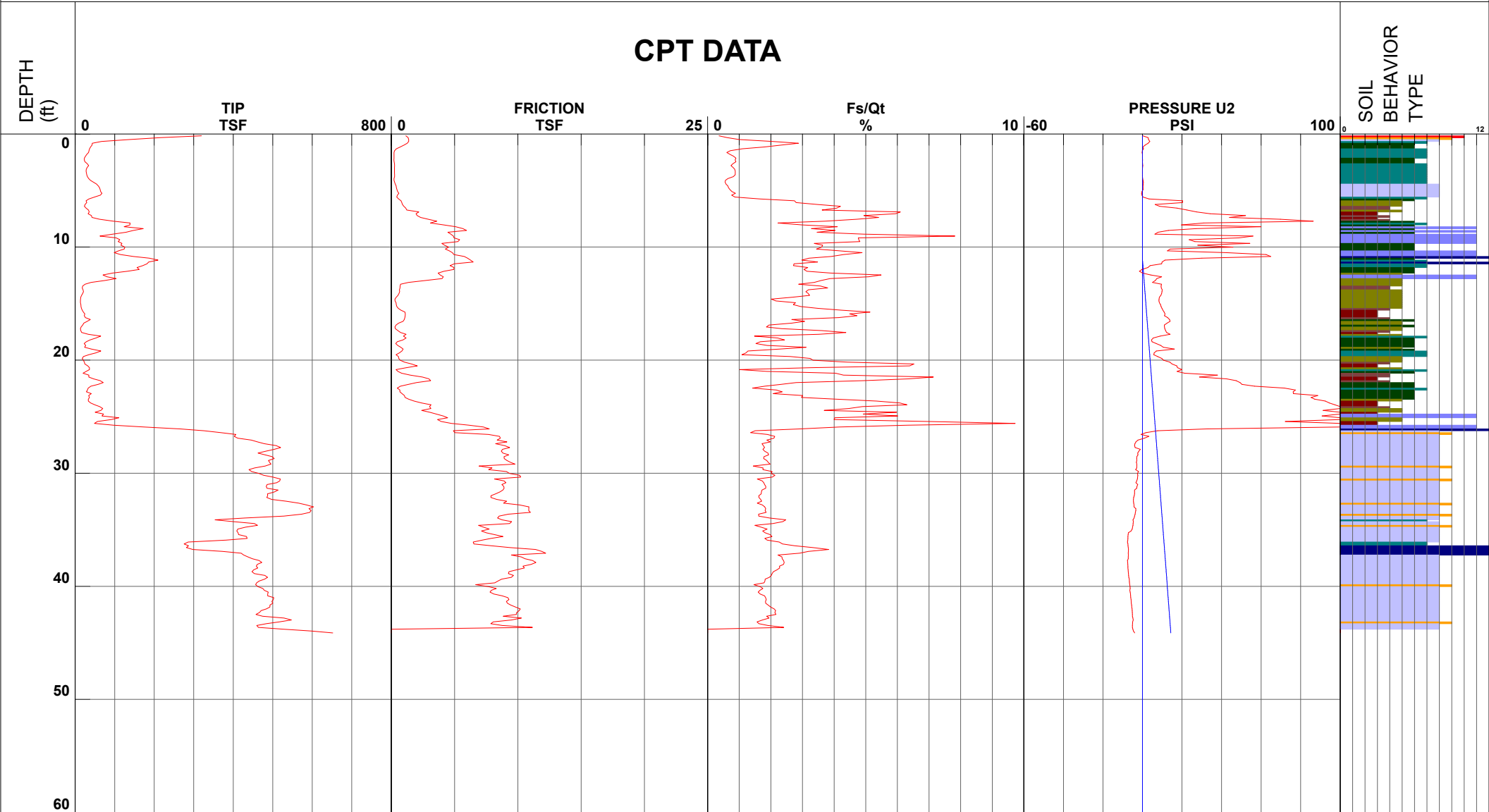


Cornerstone Earth Group

Project El Camino High School Modernization Operator JM - SS
Job Number 1517-1-5 Cone Number DDG1587
Hole Number CPT-04 Date and Time 12/29/2025 9:52:07 AM
EST GW Depth During Test 11.09 ft

Filename SDF(080).cpt
GPS _____
Maximum Depth 44.13 ft

Net Area Ratio .8



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

Cone Size 15cm²

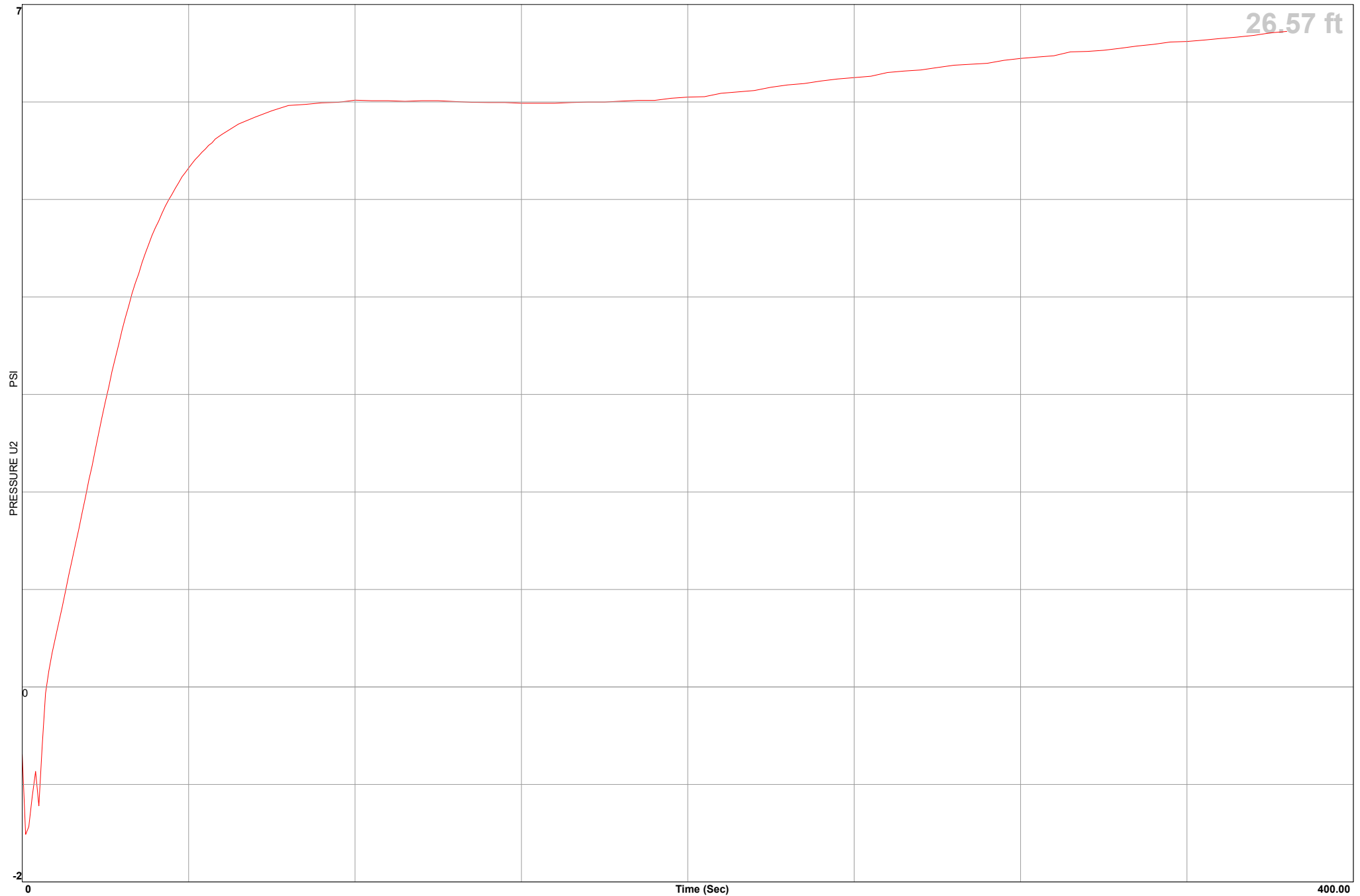
S*Soil behavior type and SPT based on data from UBC-1983



Cornerstone Earth Group

Location	El Camino High School Modernization	Operator	JM - SS
Job Number	1517-1-5	Cone Number	El Camino High School Modernization
Hole Number	CPT-04	Date and Time	12/29/2025 9:52:07 AM
Equilized Pressure	6.7	EST GW Depth During Test	11.0

GPS _____



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 61 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 37 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 7 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

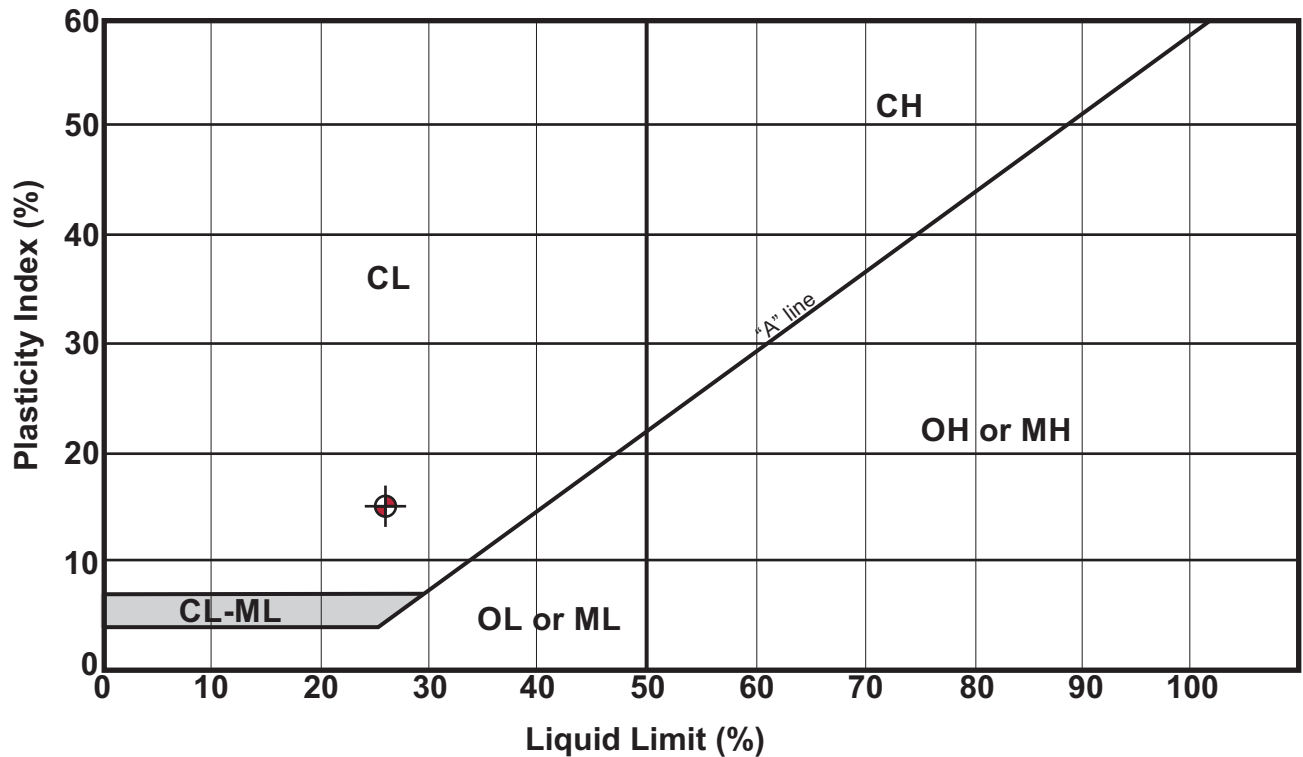
Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification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

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

R-value: An R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 73 at an exudation pressure of 300 pounds per square inch.

Corrosion: Soluble sulfate determinations (ASTM D4327), resistivity tests (ASTM G57), chloride determinations (ASTM D4327), and pH determinations (ASTM G51) were performed on 3 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-2	4.0	10	determined nonplastic			—	Silty Sand (SM) [Qc]
⊗	EB-6	2.0	16	27	12	15	—	Sandy Lean Clay (CL)
	EB-9	4.0	28	determined nonplastic			—	Silty Sand (SM) [Qc]

Samples prepared in accordance with ASTM D421

CORNERSTONE
EARTH GROUP

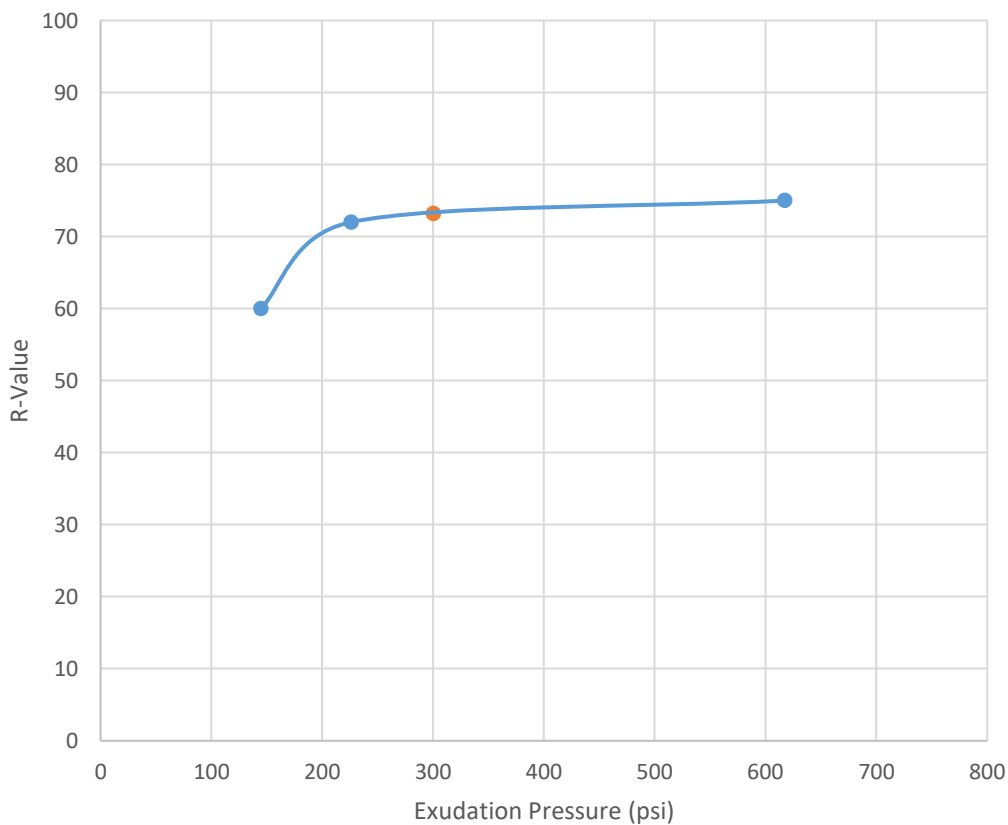
Date Tested	1/13,14/2026
Tested By	BBA, FL

[illegible]

R-Value of Treated and Untreated Bases

CTM 301

Project Name:	El Camino High School Modernization	Sample Date: 1/5/2026
Project Number:	1517-1-5	Test Date: 1/12/2026
Project Address :	--	Report Date: 1/13/2026
City/State:	--	Sampled By: Others
Sample Location:	EB-8 @ 0-5	Lab Number: M-2829
Material:	P.G. fine Sand to Silty Sand (SM)	



Specimen Number	1	2	3
Dry Density of Briq. (lbs/ft ³)	118.8	120.3	121.8
Moisture @ Compaction	14.5%	13.5%	12.6%
Expansion Dial Reading	0	0	0
R-Value	60	72	75
R-Value @ 300 psi			73
Material Description	Silty Sand / Sand (SM)		
Specification			

Reported By:

Suzanne Morgan
Laboratory Manager

APPENDIX C: PREVIOUS INVESTIGATIONS



CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 1 OF 1

DATE STARTED 7/9/24 DATE COMPLETED 7/9/24

DRILLING CONTRACTOR Exploration Geoservices, Inc.

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

LOGGED BY BTM

NOTES

PROJECT NAME El Camino High School Kitchen Modernization

PROJECT NUMBER 1517-1-1

PROJECT LOCATION South San Francisco, CA

GROUND ELEVATION 86 FT +/- BORING DEPTH 24.5 ft.

LATITUDE 37.665570° LONGITUDE -122.441675°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 12 ft.

▼ AT END OF DRILLING 8 ft.

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	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				
86.0	0		2 inches asphalt concrete over 2 inches aggregate base							○ HAND PENETROMETER				
85.6			Silty Sand (SM) medium dense, moist, dark brown to brown, fine to medium sand	17	MC-1B	110	11			△ TORVANE				
				23	MC-2B	113	15			● UNCONFINED COMPRESSION				
81.5	5		Sandy Lean Clay (CL) stiff, moist, brown, fine to medium sand, low plasticity	11	MC-3B	110	16			▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL				
				8	SPT					1.0	2.0	3.0	4.0	
				13	MC-5B	110	18							
	10			26	SPT									
75.0			Clayey Sand (SC) medium dense, moist, brown, fine to medium sand											
73.0			Silty Sand (SM) medium dense, moist, gray, fine to medium sand	17	SPT-7		20							
	15			13	SPT-8		18							
				19	SPT-9		17		20					
67.5			Silty Sand (SM) [Qc] very dense, moist, gray with reddish brown mottles, fine to medium sand											
	20			60	SPT-10		16							
				50	SPT-11		18							
				6"										
61.5	25		Bottom of Boring at 24.5 feet.											



CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 2

DATE STARTED 7/9/24 DATE COMPLETED 7/9/24

DRILLING CONTRACTOR Exploration Geoservices, Inc.

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

LOGGED BY BTM

NOTES

PROJECT NAME El Camino High School Kitchen Modernization

PROJECT NUMBER 1517-1-1

PROJECT LOCATION South San Francisco, CA

GROUND ELEVATION 85.5 FT +/- BORING DEPTH 50 ft.

LATITUDE 37.665455° LONGITUDE -122.441779°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING 18 ft.

▼ AT END OF DRILLING 12 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				
85.5	0		2 inches asphalt concrete over 6 inches aggregate base							○ HAND PENETROMETER				
84.8			Silty Sand with Gravel (SM) [Fill] medium dense, moist, dark brown, fine to coarse sand, fine subangular gravel, trace brick fragments	56	MC-1B	127	9	NP	26					
81.8			Silty Sand (SM) medium dense, moist, dark brown to brown, fine to medium sand NP = Non-plastic	25	MC-2B	115	10							
5				27	MC-3B	114	12							
77.5			Sandy Lean Clay (CL) very stiff, moist, gray brown, fine to medium sand, low plasticity brick fragment?	20	MC-4B	106	20					○		
75.5	10		Clayey Sand (SC) medium dense, moist, brown, fine to medium sand	24	SPT-5		17							
74.0			Silty Sand (SM) [Qc] medium dense, moist, brown and gray mottled, fine to medium sand becomes dense	25	SPT-6		17							
				42	SPT-7		18							
15														
68.5			Clayey Sand (SC) [Qc] dense, moist, brown and gray mottled, fine to medium sand	34	SPT-8		21							
20														
63.5			Sandy Lean Clay (CL) [Qc] very stiff, moist, gray with reddish brown mottles, fine to medium sand, low plasticity	39	SPT							○		
25														
59.5														

Continued Next Page



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BORING NUMBER B-3

PAGE 1 OF 1

CLIENT South San Francisco Unified School District

PROJECT NAME El Camino High School

PROJECT NUMBER 91-02829-A

PROJECT LOCATION 1320 Mission Road, South San Francisco, CA

DATE STARTED 4/5/12 **COMPLETED** 4/5/12

GROUND ELEVATION 102 ft **HOLE SIZE** 8"

DRILLING CONTRACTOR Exp Geoservices

GROUND WATER LEVELS:

DRILLING METHOD HSA

AT TIME OF DRILLING ---

LOGGED BY RS **CHECKED BY** GH

AT END OF DRILLING 17.50 ft / Elev 84.50 ft

NOTES

AFTER DRILLING ---

GEOTECH BH COLUMNS - STANDARD TEMP GDT - 5/31/12 13:21 - R:\GEOTECH PROJECTS BY NUMBER\2800-2899\91-02821-A TO 91-02829-A SSFUSD PHASE II\91-02829-A EL CAMINO HSEL CAMINO HS.GPJ

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0												
		2" THICK ASPHALT CONCRETE :										
		6" THICK AGGREGATE BASE :										
		(SC) CLAYEY SAND : yellowish brown, moist, dense.	MC 1		8-13-22 (35)		127	11				
		(SM) SILTY SAND : gray, moist, medium dense, w/ traces of clay and ferrous staining.	MC 2		4-8-9 (17)		107	20				
5												
		(SM) SILTY SAND : brown and gray mottled, damp, medium dense to dense, w/ ferrous staining.	MC 3		7-13-15 (28)							
10												
15												
		Groundwater encountered at 17.5 ft.										
		(SP) FINE SAND : gray, wet, very dense, w/ ferrous staining.	SPT 4		20-23-30 (53)							
20												

Bottom of borehole at 20.0 feet.



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BORING NUMBER B-4

PAGE 1 OF 1

CLIENT South San Francisco Unified School District

PROJECT NAME El Camino High School

PROJECT NUMBER 91-02829-A

PROJECT LOCATION 1320 Mission Road, South San Francisco, CA

DATE STARTED 4/5/12 **COMPLETED** 4/5/12

GROUND ELEVATION 102 ft **HOLE SIZE** 8"

DRILLING CONTRACTOR Exp Geoservices

GROUND WATER LEVELS:

DRILLING METHOD HSA

AT TIME OF DRILLING ---

LOGGED BY RS **CHECKED BY** GH

AT END OF DRILLING 18.00 ft / Elev 84.00 ft

NOTES

AFTER DRILLING ---

GEOTECH BH COLUMNS - STANDARD TEMP GDT - 5/31/12 13:21 - R:\GEOTECH PROJECTS BY NUMBER\2800-2899\91-02821-A TO 91-02829-A SSFUSD PHASE II\91-02829-A EL CAMINO HSEL CAMINO HS.GPJ

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		2" THICK ASPHALT CONCRETE : 6" THICK AGGREGATE BASE : (SP) FINE SAND : yellowish brown/dark gray, damp, dense, w/ ferrous staining.	MC 1		8-16-20 (36)							
5		(SP) FINE SAND : dark gray, dry to damp, dense.	MC 2		4-12-20 (32)		109	6				
10		(SP) FINE SAND : yellowish gray, damp, medium dense, w/ traces of clay.	SPT 3		10-10-13 (23)							
15		hard pan between 11'-12' becomes moist.	SPT 4		7-7-12 (19)							
20		Groundwater encountered at 18 ft. becomes wet, dense, w/ traces of gravel.	SPT 5		18-23-19 (42)							

Bottom of borehole at 20.0 feet.

APPENDIX D: LIQUEFACTION ANALYSES CALCULATIONS

CPT No. 1

PGA (A_{max}) 0.95

Total Settlement: 0.00 (Inches)

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Depth (ft)	Qc (tsf)	f _c (tsf)	C _{uc} (psf)	In situ σ' (psf)	Q	F (%)	I _c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q _{ch} near interfaces (soil layer)	Thin Layer Factor (K _u)	Interpreted Q _{ch}	C _u	Q _{c1N}	Q _{c1N-C5}	Stress Reduction Coeff. f _s	CSR	K _s for Sand	CRR _N >7.5, σ'vc = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε _{vc}	Settlement (Inches)
0.160	576.050	0.644	19.2	19.2	5715.767	0.112	0.39		Unsaturated	0.0			544.47	1.70	925.60	925.60	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.330	324.540	1.617	39.6	39.6	2242.158	0.498	0.93		Unsaturated	0.0			306.75	1.70	521.47	521.47	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.490	208.070	1.818	58.8	58.8	1179.591	0.874	1.23		Unsaturated	0.0			168.66	1.70	334.33	334.33	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.660	177.900	1.806	79.2	79.2	868.938	1.055	1.34		Unsaturated	0.0			168.15	1.70	285.85	285.85	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.820	137.090	1.742	98.4	98.4	600.654	1.271	1.49		Unsaturated	0.0			129.57	1.70	220.28	220.28	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.980	159.460	1.601	117.6	117.6	639.087	1.004	1.39		Unsaturated	0.0			150.72	1.70	256.22	256.22	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.150	156.310	1.420	138.0	138.0	578.286	0.909	1.37		Unsaturated	0.0			147.74	1.70	251.16	251.16	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.310	130.680	1.279	157.2	157.2	452.891	0.979	1.46		Unsaturated	0.0			123.52	1.70	209.98	209.98	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.480	100.500	1.088	177.6	177.6	327.582	1.084	1.58		Unsaturated	0.0			94.99	1.70	161.48	161.48	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.640	78.260	0.963	196.8	196.8	242.244	1.232	1.70		Unsaturated	0.0			73.97	1.70	125.75	125.75	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.800	65.050	0.626	216.0	216.0	192.119	0.963	1.69		Unsaturated	0.0			61.48	1.70	104.52	104.52	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.970	57.730	0.642	236.4	236.4	162.915	1.113	1.79		Unsaturated	5.8			54.57	1.70	92.76	93.24	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.130	54.150	0.978	255.6	255.6	146.914	1.255	1.85		Unsaturated	11.3			51.18	1.70	37.01	97.93	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.300	56.030	0.821	276.0	276.0	146.274	1.470	1.90		Unsaturated	15.4			52.96	1.70	90.03	113.29	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.460	64.920	1.052	295.2	295.2	163.909	1.624	1.90		Unsaturated	15.3			61.36	1.70	104.31	128.53	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.620	85.390	1.637	314.4	314.4	208.996	1.921	1.89		Unsaturated	14.4			80.71	1.70	137.21	161.35	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.790	89.860	1.930	340.0	340.0	213.126	1.952	1.93		Unsaturated	17.2			84.93	1.70	144.39	178.69	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.950	84.330	2.281	354.0	354.0	194.464	2.710	2.03		Unsaturated	25.5			79.71	1.70	135.50	190.41	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.120	65.370	2.813	374.4	374.4	146.466	4.316	2.27		Unsaturated	44.4			61.79	1.70	105.04	175.56	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.280	72.650	2.748	393.6	393.6	158.782	3.792	2.20		Unsaturated	39.1			68.67	1.70	116.73	186.06	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.440	81.850	2.446	412.8	412.8	174.713	2.996	2.09		Unsaturated	30.5			77.36	1.70	131.52	194.26	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.610	96.390	2.212	433.2	433.2	200.901	2.300	1.97		Unsaturated	20.3			91.11	1.69	153.97	199.01	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.770	89.520	2.194	452.4	452.4	182.529	2.457	2.01		Unsaturated	24.1			84.61	1.68	142.11	195.10	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.940	76.790	2.330	472.8	472.8	153.073	3.044	2.13		Unsaturated	33.7			72.58	1.68	122.11	187.08	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.100	58.680	2.448	492.0	492.0	114.500	4.190	2.32		Unsaturated	48.6			55.44	1.70	94.26	164.53	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.270	49.010	2.332	512.4	512.4	93.643	4.783	2.42		Unsaturated	56.6			46.32	1.70	82.75	148.36	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.430	53.440	2.293	531.6	531.6	100.272	4.313	2.37		Unsaturated	52.3			50.51	1.70	85.87	155.67	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.590	53.810	1.805	550.8	550.8	99.177	3.371	2.29		Unsaturated	45.9			50.86	1.70	86.46	153.17	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.760	53.730	1.940	571.2	571.2	97.226	3.630	2.32		Unsaturated	48.3			50.78	1.68	85.51	153.31	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.920	62.150	2.048	590.4	590.4	110.681	3.311	2.25		Unsaturated	43.0			58.74	1.63	95.73	162.95	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.090	74.580	2.075	611.3	611.3	121.222	3.214	2.14		Unsaturated	34.7			70.49	1.58	134.91	174.88	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.250	81.630	1.485	631.3	615.7	142.486	1.826	1.98		Unsaturated	21.5			77.16	1.59	122.83	166.84	1.00	0.615	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.410	115.380	1.649	651.3	625.7	199.988	1.433	1.81		Unsaturated	7.5			109.05	1.56	170.54	173.29	1.00	0.615	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.580	129.480	1.906	672.6	636.3	222.593	1.476	1.79		Unsaturated	5.9			122.38	1.52	185.88	186.57	1.00	0.615	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.740	97.460	2.853	753.0	738.0	165.084	2.364	2.03		Unsaturated	46.4			92.12	1.50	117.58	192.80	1.00	0.615	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.910	116.580	2.786	713.8	657.0	197.148	2.397	1.98		Unsaturated	21.8			110.19	1.45	159.60	209.76	1.00	0.615	1.00	n.a.	n.a.	n.a.	0.00	0.00
6.070	116.400	3.404	733.8	667.0	195.343	2.934	2.06		Unsaturated	27.7			110.02	1.42	156.39	219.33	0.99	0.614	1.00	n.a.	n.a.	n.a.	0.00	0.00
6.230	107.890	3.249	753.8	677.0	179.655	3.022	1.99		Unsaturated	30.2			101.98	1.43	146.00	211.21	0.99	0.614	1.00	n.a.	n.a.	n.a.	0.00	0.00
6.400	92.450	3.196	775.0	687.6	152.642	3.471	2.18		Unsaturated	37.4			87.38	1.45	127.08	197.25	0.99	0.614	1.00	n.a.	n.a.	n.a.	0.00	0.00
6.560	98.600	3.158	795.0	697.7	161.748	3.214	2.14		Unsaturated	34.7			93.25	1.44	134.01	202.04	0.99	0.614	1.00	n.a.	n.a.	n.a.	0.00	0.00
6.730	142.130	3.304	816.3	708.3	231.527	2.332	1.93		Unsaturated	17.8			134.34	1.38	185.67	226.60	0.99	0.613	1.00	n.a.	n.a.	n.a.	0.00	0.00
6.890	133.000	3.488	836.3	718.3	215.080	2.631	2.00		Unsaturated	22.7			125.71	1.37	172.85	227.39	0.99	0.613	1.00	n.a.	n.a.	n.a.	0.00	0.00
7.050	105.510	3.553	856.3	728.3	169.292	3.381	2.14		Unsaturated	34.6			99.73	1.40	139.81	209.74	0.99	0.613	1.00	n.a.	n.a.	n.a.	0.00	0.00
7.220	87.040	3.853	882.0	738.0	136.582	4.449	2.48		Unsaturated	46.4			82.27	1.43	127.58	192.80	0.99	0.612	1.00	n.a.	n.a.	n.a.	0.00	0.00
7.380	106.750	3.939	897.5	749.0	168.878	3.423	2.15		Unsaturated	35.0			100.90	1.39	140.06	210.53	0.99	0.613	1.00	n.a.	n.a.	n.a.	0.00	0.00
7.550	101.970	4.231	918.8	759.6	160.134	4.168	2.23		Unsaturated	41.7			96.38	1.38	133.46	209.01	0.99	0.612	1.00	n.a.	n.a.	n.a.	0.00	0.00
7.710	113.110	4.339	938.8	769.6	176.531	3.852	2.18		Unsaturated	37.5			106.91	1.36	145.39	219.88	0.99	0.612	1.00	n.a.	n.a.	n.a.	0.00	0.00
7.870	114.450	4.287	958.8	779.7	177.465	3.761	2.17		Unsaturated	36.7			108.18	1.35	146.45	220.34	0.99	0.612	1.00	n.a.	n.a.	n.a.	0.00	0.00
8.040	131.280	4.107	980.0	790.3	227.278	3.140	2.07		Unsaturated	28.9			124.08	1.33	164.99	231.90	0.99	0.612	1.00	n.a.	n.a.	n.a.	0.00	0.00
8.200	189.690	3.626	1000.0	803.3	290.763	1.917	1.81		Unsaturated	7.7			179.29	1.31	235.52	239.31	0.99	0.612	1.00	n.a.	n.a.	n.a.	0.00	0.00
8.370	215.150	1.520	1021.3	811.0	327.704	1.002	1.55		Unsaturated	0.0			203.36	1.29	261.90	261.90	0.99	0.611	1.00	n.a.	n.a.	n.a.	0.00	0.00
8.530	239.320	2.252	1041.3	821.0	362.359	0.943	1.50		Unsaturated	0.0			226.20	1.28	290.38	290.38	0.99	0.611	1.00	n.a.	n.a.	n.a.	0.00	0.00
8.690	261.800	2.707	1061.3	831.0	394.004	0.936	1.45		Unsaturated	0.0			247.45	1.26	316.65	316.65	0.99	0.610	1.00	n.a.	n.a.	n.a.	0.00	0.00
8.860	270.700	3.377	1082.5	841.6	404.883	1.250	1.57		Unsaturated	0.0			255.86	1.28	326.31	326.31	0.99	0.611	1.00	n.a.	n.a.	n.a.		

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Depth (ft)	Qc (tsf)	f_c (tsf)	G_{vc} (psf)	Insitu G^* (psf)	Q	F (%)	I_c	Layer "Plastic" $PI > 7$	Flag Soil Type	Fines (%)	Qch near interfaces (soil layer)	Thin Layer Factor (K_{L1})	Interpreted Qch	C _N	Q _{c1N}	Q _{c1N-C5}	Stress Reduction Coeff. f_d	CSR	K_0 for Sand	CRR _{0.75} $\sigma'_{vc} = 1 \text{ atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ϵ_v	Settlement (Inches)
21.880	249.280	4.498	2722.5	1662.9	264.327	1.814	1.81		Sand	8.0			235.61	1.07	251.08	255.82	0.95	0.815	1.072	266.621	628.963	772.13	0.00	0.00
22.150	228.860	4.673	2743.8	1673.6	241.772	2.054	1.88		Sand	13.3			216.31	1.06	230.12	256.26	0.95	0.816	1.070	282.130	664.362	814.08	0.00	0.00
22.310	203.510	5.135	2763.8	1683.6	214.180	2.540	1.98		Sand	21.8			192.35	1.06	204.31	261.08	0.95	0.817	1.069	534.865	1257.398	1538.11	0.00	0.00
22.470	176.420	5.473	2783.8	1693.6	184.916	3.127	2.09		Sand	30.6			166.75	1.06	177.05	249.26	0.95	0.819	1.067	119.357	280.126	342.09	0.00	0.00
22.640	116.260	4.950	2805.0	1704.3	120.966	4.310	2.32		Sand	48.3			109.89	1.08	118.20	194.53	0.95	0.820	1.053	1.407	3.261	3.98	0.00	0.00
22.800	84.460	4.775	2825.0	1714.3	87.208	5.750	2.50		Sand	63.1			79.83	1.08	86.59	160.58	0.95	0.822	1.037	0.377	0.704	0.86	0.01	0.00
22.970	141.170	4.934	2846.3	1724.9	146.295	3.530	2.20		Sand	38.8			133.43	1.06	142.08	217.12	0.95	0.823	1.061	5.737	13.394	16.27	0.00	0.00
23.130	191.520	6.239	2866.3	1734.9	198.419	3.282	2.09		Sand	30.6			181.02	1.05	190.76	265.78	0.95	0.824	1.060	1039.057	2422.090	2938.20	0.00	0.00
23.290	187.920	6.721	2886.3	1745.0	194.091	3.604	2.13		Sand	33.7			177.62	1.05	186.89	266.10	0.95	0.826	1.058	1087.875	2531.754	3066.50	0.00	0.00
23.460	162.000	8.026	2907.5	1755.6	166.594	4.999	2.29		Sand	46.1			153.12	1.05	161.08	247.12	0.95	0.827	1.056	93.151	216.413	261.70	0.00	0.00
23.620	168.070	7.873	2927.5	1765.6	172.391	4.725	2.26		Sand	43.8			158.86	1.05	166.68	252.34	0.95	0.828	1.054	172.561	400.252	483.29	0.00	0.00
23.790	185.410	7.859	2948.8	1776.3	189.752	4.473	2.20		Sand	39.1			175.25	1.05	183.53	268.77	0.95	0.829	1.053	1617.832	3746.113	4516.26	0.00	0.00
23.950	207.790	8.221	2968.8	1786.3	212.231	3.985	2.15		Sand	35.0			196.40	1.05	205.38	290.49	0.95	0.831	1.051	6859.558	#####	19097.11	0.00	0.00
24.110	212.460	8.531	2988.8	1796.3	216.419	4.024	2.15		Sand	35.0			200.81	1.04	209.68	295.88	0.95	0.832	1.049	#####	#####	563441.30	0.00	0.00
24.280	209.330	8.031	3010.0	1806.9	212.569	3.864	2.14		Sand	34.0			197.85	1.04	206.27	290.21	0.95	0.833	1.047	6469.506	#####	179700.63	0.00	0.00
24.440	209.330	7.282	3030.0	1816.9	211.972	3.504	2.10		Sand	31.2			197.85	1.04	205.97	285.26	0.95	0.834	1.046	25425.377	#####	70171.48	0.00	0.00
24.610	276.410	5.417	3051.3	1827.6	270.585	1.971	1.83		Sand	9.2			271.55	1.04	271.55	280.88	0.95	0.835	1.044	11593.036	#####	31871.98	0.00	0.00
24.770	316.480	5.359	3071.3	1837.9	319.433	1.702	1.74		Sand	2.4			299.13	1.04	310.47	310.47	0.94	0.837	1.042	#####	#####	#####	0.00	0.00
24.930	336.360	5.813	3091.3	1847.6	338.665	1.736	1.74		Sand	1.9			317.92	1.04	329.50	329.50	0.94	0.838	1.041	#####	#####	#####	0.00	0.00
25.100	334.830	6.601	3112.5	1858.3	336.140	1.981	1.79		Sand	5.9			316.47	1.03	327.51	328.50	0.94	0.839	1.039	#####	#####	#####	0.00	0.00
25.260	331.390	6.951	3132.5	1868.3	331.767	2.107	1.81		Sand	8.0			313.22	1.03	323.98	329.21	0.94	0.840	1.037	#####	#####	#####	0.00	0.00
25.430	324.740	7.269	3153.8	1878.9	324.146	2.249	1.84		Sand	10.3			306.94	1.03	316.71	332.13	0.94	0.841	1.036	#####	#####	#####	0.00	0.00
25.590	340.740	7.518	3173.8	1888.9	338.281	2.217	1.83		Sand	9.1			322.06	1.03	331.85	341.71	0.94	0.842	1.034	#####	#####	#####	0.00	0.00
25.750	353.530	7.880	3193.8	1899.0	351.136	2.239	1.82		Sand	8.8			334.15	1.03	343.83	352.58	0.94	0.843	1.032	#####	#####	#####	0.00	0.00
25.920	374.180	8.466	3215.0	1909.6	376.692	2.272	1.82		Sand	8.3			353.67	1.03	363.37	370.40	0.94	0.844	1.031	#####	#####	#####	0.00	0.00
26.080	384.580	8.963	3235.0	1919.6	380.034	2.236	1.80		Sand	7.3			363.50	1.03	372.96	376.99	0.94	0.845	1.029	#####	#####	#####	0.00	0.00
26.250	372.760	8.337	3256.3	1930.3	367.277	2.246	1.81		Sand	8.1			352.33	1.02	360.97	367.27	0.94	0.846	1.028	#####	#####	#####	0.00	0.00
26.410	358.010	7.864	3276.3	1940.3	351.759	2.207	1.82		Sand	8.3			338.38	1.02	346.21	353.14	0.94	0.847	1.026	#####	#####	#####	0.00	0.00
26.570	344.970	7.464	3296.3	1950.3	338.006	2.174	1.82		Sand	8.6			326.06	1.02	333.15	340.92	0.94	0.848	1.024	#####	#####	#####	0.00	0.00
26.740	335.170	7.054	3317.5	1960.9	327.455	2.115	1.82		Sand	8.3			316.80	1.02	323.22	329.93	0.94	0.849	1.023	#####	#####	#####	0.00	0.00
26.900	326.080	6.912	3337.5	1970.9	317.710	2.131	1.83		Sand	9.1			303.20	1.02	314.63	323.63	0.94	0.850	1.021	#####	#####	#####	0.00	0.00
27.070	325.970	7.131	3358.8	1981.6	316.738	2.199	1.84		Sand	10.1			308.10	1.02	313.48	327.56	0.94	0.851	1.020	#####	#####	#####	0.00	0.00
27.230	331.110	7.100	3378.8	1991.6	320.939	2.155	1.83		Sand	9.2			312.96	1.02	318.00	328.33	0.94	0.851	1.018	#####	#####	#####	0.00	0.00
27.400	327.060	6.888	3400.0	2002.2	316.139	2.117	1.82		Sand	9.0			309.13	1.01	313.67	322.85	0.94	0.852	1.017	#####	#####	#####	0.00	0.00
27.560	317.890	6.812	3420.0	2012.3	306.221	2.156	1.84		Sand	10.1			300.24	1.01	304.24	318.28	0.94	0.853	1.015	#####	#####	#####	0.00	0.00
27.720	323.490	7.193	3440.0	2022.3	311.099	2.236	1.85		Sand	10.9			309.75	1.01	309.43	327.52	0.94	0.854	1.014	#####	#####	#####	0.00	0.00
27.890	349.620	7.871	3461.3	2032.9	335.759	2.261	1.84		Sand	9.8			330.74	1.01	334.25	347.76	0.93	0.855	1.012	#####	#####	#####	0.00	0.00
28.050	376.120	8.451	3481.3	2042.9	360.128	2.257	1.82		Sand	8.6			355.50	1.01	358.81	366.97	0.93	0.856	1.011	#####	#####	#####	0.00	0.00
28.220	385.710	8.053	3502.5	2053.6	368.385	2.097	1.79		Sand	6.0			364.57	1.01	367.46	368.00	0.93	0.856	1.009	#####	#####	#####	0.00	0.00
28.380	388.490	8.224	3522.5	2063.6	370.141	2.127	1.79		Sand	6.1			367.19	1.01	369.63	371.27	0.93	0.857	1.007	#####	#####	#####	0.00	0.00
28.540	390.450	8.481	3542.5	2073.6	371.108	2.182	1.80		Sand	7.0			369.05	1.01	371.02	374.16	0.93	0.858	1.006	#####	#####	#####	0.00	0.00
28.710	384.450	8.883	3563.8	2084.2	364.435	2.321	1.83		Sand	9.2			363.37	1.00	364.83	375.92	0.93	0.859	1.005	#####	#####	#####	0.00	0.00
28.870	402.210	9.729	3583.8	2094.3	380.426	2.430	1.84		Sand	9.8			380.16	1.00	381.20	395.97	0.93	0.859	1.003	#####	#####	#####	0.00	0.00
29.040	421.020	10.376	3605.0	2104.9	397.279	2.475	1.83		Sand	9.7			397.94	1.00	398.49	412.85	0.93	0.860	1.002	#####	#####	#####	0.00	0.00
29.200	433.150	10.685	3625.0	2114.9	407.795	2.479	1.83		Sand	9.3			409.40	1.00	409.46	422.07	0.93	0.861	1.000	#####	#####	#####	0.00	0.00
29.360	435.900	10.704	3645.0	2124.9	409.418	2.466	1.83		Sand	9.1			412.00	1.00	411.55	423.00	0.93	0.861	0.999	#####	#####	#####	0.00	0.00
29.530	423.650	10.252	3666.3	2135.6	396.861	2.430	1.83		Sand	9.1			400.43	1.00	399.45	411.01	0.93	0.862	0.997	#####	#####	#####	0.00	0.00
29.690	402.570	8.608	3686.3	2145.6	376.138	2.148	1.79		Sand	6.3			380.50	1.00	379.11	380.81	0.93	0.863	0.996	#####	#####	#####	0.00	0.00
29.860	403.520	8.553	3707.5	2155.2	378.088	2.129	1.80		Sand	6.1			381.40	1.00	379.51	380.80	0.93	0.864	0.994	#####	#####	#####	0.00	0.00
30.020	412.970	9.017	3727.5	2166.3	383.662	2.217	1.80		Sand	6.9			389.95	0.99	387.55	390.56	0.93	0.864	0.993	#####	#####	#####	0.00	0.00
30.180	419.340	9.942	3747.5	2176.3	389.079	2.382	1.82		Sand	8.9			396.35	0.99	393.43	403.47	0.93	0.865	0.992	#####	#####	#####	0.00	0.00
30.350	441.620	11.200	3768.8	2186.9	408.835	2.547	1.84		Sand	10.1			417.41	0.9										

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Depth (ft)	q _c (tsf)	f _s (tsf)	C _{vc} (psf)	In situ σ' (psf)	Q	F (%)	I _c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q _h near interfaces (soil layer)	Thin Layer Factor (K _u)	Interpreted q _h	C _N	Q _{uN}	Q _{uN-CS}	Stress Reduction Coeff. f _s	CSR	K _s for Sand	CRR _u >7.5, σ'vc = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε _{vc}	Settlement (Inches)
0.160	161.350	1.105	19.2	19.2	1600.902	0.685	1.09		Unsaturated	0.0			152.50	1.70	259.26	259.26	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.330	148.690	1.219	39.6	39.6	1027.185	0.820	1.22		Unsaturated	0.0			140.54	1.70	238.92	238.92	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.490	150.050	1.376	59.8	59.8	850.618	0.917	1.30		Unsaturated	0.0			141.82	1.70	241.10	241.10	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.660	127.110	1.157	79.2	79.2	620.803	0.910	1.36		Unsaturated	0.0			120.14	1.70	204.24	204.24	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.820	140.250	1.043	98.4	98.4	614.505	0.744	1.29		Unsaturated	0.0			132.56	1.70	225.35	225.35	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.980	170.920	1.324	117.6	117.6	685.034	0.775	1.28		Unsaturated	0.0			161.55	1.70	274.64	274.64	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.150	234.060	2.021	138.0	138.0	866.028	0.864	1.27		Unsaturated	0.0			221.23	1.70	376.09	376.09	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.310	277.680	2.790	157.2	157.2	962.648	1.005	1.32		Unsaturated	0.0			262.46	1.70	446.18	446.18	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.480	295.420	2.822	177.6	177.6	963.518	0.956	1.29		Unsaturated	0.0			279.22	1.70	474.68	474.68	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.640	307.670	4.040	196.8	196.8	953.248	1.134	1.43		Unsaturated	0.0			290.80	1.70	494.37	494.37	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.800	283.570	4.137	216.0	216.0	836.572	1.459	1.49		Unsaturated	0.0			268.02	1.70	455.64	455.64	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.970	282.760	4.027	236.4	236.4	799.254	1.425	1.49		Unsaturated	0.0			267.26	1.70	454.34	454.34	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.130	252.630	4.338	255.2	255.2	686.084	1.718	1.59		Sand	0.0			239.78	1.70	405.95	405.93	1.00	0.638	1.000	#####	#####	#####	0.00	0.00
2.300	251.950	3.602	276.0	276.0	659.013	1.430	1.52		Sand	0.0			238.14	1.70	404.83	404.83	1.00	0.662	1.100	#####	#####	#####	0.00	0.00
2.460	238.820	3.658	295.2	295.2	603.971	1.533	1.57		Sand	0.0			225.73	1.68	379.54	379.54	1.00	0.683	1.100	#####	#####	#####	0.00	0.00
2.620	250.510	3.592	314.4	314.4	613.880	1.435	1.57		Sand	0.0			236.78	1.65	391.56	391.56	1.00	0.703	1.100	#####	#####	#####	0.00	0.00
2.790	263.160	3.494	337.6	337.6	624.914	1.325	1.50		Sand	0.0			247.71	1.63	404.56	404.56	1.00	0.845	1.100	#####	#####	#####	0.00	0.00
2.950	264.760	3.554	354.0	354.0	611.410	1.343	1.51		Sand	0.0			250.25	1.60	401.08	401.08	1.00	0.740	1.100	#####	#####	#####	0.00	0.00
3.120	274.420	3.828	374.4	374.4	616.203	1.396	1.52		Sand	0.0			259.38	1.58	409.61	409.61	1.00	0.757	1.100	#####	#####	#####	0.00	0.00
3.280	297.730	6.804	393.6	393.6	652.049	2.220	1.70		Sand	0.0			281.41	1.56	438.58	438.58	1.00	0.772	1.100	#####	#####	#####	0.00	0.00
3.440	312.650	8.048	412.8	412.8	668.612	2.576	1.75		Sand	3.3			295.51	1.54	454.81	454.81	1.00	0.786	1.100	#####	#####	#####	0.00	0.00
3.610	330.690	7.436	433.2	433.2	690.342	2.250	1.69		Sand	0.0			312.51	1.52	474.97	474.97	1.00	0.799	1.100	#####	#####	#####	0.00	0.00
3.770	364.540	8.775	452.4	452.4	744.709	2.409	1.71		Sand	0.0			344.56	1.50	517.63	517.63	1.00	0.811	1.100	#####	#####	#####	0.00	0.00
3.940	424.860	8.775	472.8	472.8	849.058	2.067	1.63		Sand	0.0			401.57	1.48	596.30	596.30	1.00	0.824	1.100	#####	#####	#####	0.00	0.00
4.100	410.950	9.811	492.0	492.0	805.042	2.235	1.67		Sand	0.0			388.42	1.47	570.75	570.75	1.00	0.834	1.100	#####	#####	#####	0.00	0.00
4.270	281.770	4.763	512.4	512.4	540.714	1.699	1.63		Sand	0.0			266.32	1.45	387.17	387.17	1.00	0.845	1.100	#####	#####	#####	0.00	0.00
4.430	157.900	5.058	531.6	531.6	297.256	3.209	1.67		Sand	22.5			149.24	1.44	214.87	275.44	1.00	0.855	1.100	4615.443	#####	13068.69	0.00	0.00
4.590	104.310	3.286	550.8	550.8	192.732	3.159	2.09		Sand	30.1			98.59	1.52	149.58	215.30	1.00	0.864	1.100	5.023	12.55	14.07	0.00	0.00
4.760	191.690	3.871	571.2	571.2	348.201	2.022	1.79		Sand	5.9			181.18	1.41	255.95	256.78	1.00	0.873	1.100	301.573	729.80	836.05	0.00	0.00
4.920	208.160	4.897	590.4	590.4	319.146	2.356	1.83		Sand	9.3			196.75	1.40	275.53	285.16	1.00	0.881	1.100	2464.717	#####	66560.37	0.00	0.00
5.090	180.050	4.953	610.8	610.8	333.804	2.319	1.89		Sand	14.3			179.63	1.39	249.31	281.34	1.00	0.901	1.100	12556.34	#####	34167.63	0.00	0.00
5.250	234.670	7.146	630.0	630.0	405.953	3.049	1.91		Sand	15.8			221.81	1.38	305.34	349.78	1.00	0.897	1.100	#####	#####	#####	0.00	0.00
5.410	246.660	7.229	649.2	649.2	420.349	2.935	1.89		Sand	14.0			233.14	1.37	318.41	354.43	1.00	0.904	1.100	#####	#####	#####	0.00	0.00
5.740	247.740	6.698	688.8	688.8	409.412	2.720	1.87		Sand	12.4			233.91	1.34	314.52	341.24	1.00	0.917	1.100	#####	#####	#####	0.00	0.00
5.910	262.560	6.855	685.6	685.6	402.737	2.127	1.75		Sand	7.0			262.56	1.37	362.35	362.35	1.00	0.924	1.100	#####	#####	#####	0.00	0.00
6.070	273.740	5.212	728.4	728.4	440.400	1.906	1.71		Sand	0.0			258.73	1.32	342.80	342.80	0.99	0.930	1.100	#####	#####	#####	0.00	0.00
6.230	268.530	5.200	747.6	747.6	426.408	1.939	1.73		Sand	1.1			253.81	1.32	333.98	333.98	0.99	0.935	1.100	#####	#####	#####	0.00	0.00
6.400	262.250	5.340	768.0	768.0	410.838	2.039	1.75		Sand	3.2			247.87	1.31	323.86	323.86	0.99	0.941	1.100	#####	#####	#####	0.00	0.00
6.560	270.510	5.450	787.2	787.2	418.582	2.018	1.74		Sand	2.6			255.68	1.30	331.89	331.89	0.99	0.946	1.100	#####	#####	#####	0.00	0.00
6.730	271.240	5.414	807.6	807.6	414.363	1.999	1.74		Sand	2.5			256.37	1.29	330.55	330.55	0.99	0.951	1.100	#####	#####	#####	0.00	0.00
6.890	295.380	5.444	826.8	826.8	446.010	1.846	1.70		Sand	0.0			279.19	1.28	357.74	357.74	0.99	0.956	1.100	#####	#####	#####	0.00	0.00
7.050	311.320	5.717	846.0	846.0	464.733	1.839	1.69		Sand	0.0			294.25	1.27	374.77	374.77	0.99	0.960	1.100	#####	#####	#####	0.00	0.00
7.220	325.460	6.039	866.4	866.4	480.100	1.858	1.69		Sand	0.0			307.62	1.27	389.33	389.33	0.99	0.965	1.100	#####	#####	#####	0.00	0.00
7.380	346.260	6.865	885.6	885.6	505.245	1.907	1.68		Sand	0.0			327.28	1.26	411.83	411.83	0.99	0.969	1.100	#####	#####	#####	0.00	0.00
7.550	368.550	7.348	906.0	906.0	531.705	1.996	1.69		Sand	0.0			348.35	1.25	435.71	435.71	0.99	0.973	1.100	#####	#####	#####	0.00	0.00
7.710	378.870	8.225	925.2	925.2	540.896	2.174	1.72		Sand	0.8			358.10	1.24	445.44	445.44	0.99	0.977	1.100	#####	#####	#####	0.00	0.00
7.870	407.870	9.045	944.4	944.4	576.385	2.220	1.72		Sand	0.6			385.51	1.24	476.95	476.95	0.99	0.981	1.100	#####	#####	#####	0.00	0.00
8.040	447.020	10.167	964.8	964.8	625.045	2.277	1.72		Sand	0.2			422.51	1.23	519.79	519.79	0.99	0.985	1.100	#####	#####	#####	0.00	0.00
8.200	498.090	11.525	984.0	984.0	689.680	2.316	1.71		Sand	0.0			476.17	1.22	576.17	576.17	0.99	0.988	1.100	#####	#####	#####	0.00	0.00
8.370	547.040	12.653	1004.4	1004.4	749.789	2.315	1.69		Sand	0.0			517.05	1.22	629.37	629.37	0.99	0.992	1.100	#####	#####	#####	0.00	0.00
8.530	580.240	13.548	1023.6	1023.6	787.828	2.337	1.69		Sand	0.0			548.43	1.21	664.24	664.24	0.99	0.995	1.100	#####	#####	#####	0.00	0.00
8.690	583.740	14.089	1042.8	1042.8	785.241	2.416	1.70		Sand	0.0			551.74	1.21	664.98	664.98	0.99	0.998	1.100	#####	#####	#####	0.00	0.00
8.860	557.680	14.174	1063.2	1063.2	742.097	2.544	1.73		Sand	0.6			523.66	1.20	632.06	632.06	0.99	1.004	1.100	#####	#####	#####	0.00	0.00
9.020	518.310	13.390	1082.4	1082.4	6																			



CPT No.

2

PGA (A_{max})

0.95

Total Settlement: 0.00 (Inches)

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Depth (ft)	Q_c (tsf)	f_c (tsf)	σ'_{vc} (psf)	In situ σ'_{vc} (psf)	Q	F (%)	I_c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q_{ch} near interfaces (soft layer)	Thin Layer Factor (K_{th})	Interpreted q_{ch}	C_N	q_{c1N}	q_{c1N-CS}	Stress Reduction Coeff. f_d	CSR	K_s for Sand	$CRR_{d=7.5}$, $\sigma'_{vc} = 1 \text{ atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ϵ_v	Settlement (Inches)
22.150	674.080	11.837	2658.0	2658.0	567.347	1.760	1.63		Sand	0.0			637.13	0.94	599.92	599.92	0.95	1.081	0.932	#####	#####	#####	0.00	0.00

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Depth (ft)	Qc (tsf)	f _c (tsf)	C _u (psf)	In situ C _v (psf)	Q	F (%)	lc	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q _N near interfaces (soil layer)	Thin Layer Factor (K _u)	Interpreted Q _N	C _N	Q _N	Q _N CS	Stress Reduction Coeff. f _s	CSR	K _e for Sand	CRR _{7.5} , σ _v ' = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain E _v	Settlement (Inches)
0.160	552.510	1.180	19.2	19.2	5482.191	0.214	0.61		Unsaturated	0.0			522.22	1.70	887.78	887.78	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.330	451.390	1.630	39.6	39.6	3118.584	0.361	0.78		Unsaturated	0.0			426.64	1.70	725.30	725.30	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.490	260.260	1.896	58.8	58.8	1475.509	0.729	1.12		Unsaturated	0.0			245.99	1.70	418.19	418.19	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.660	183.020	2.346	79.2	79.2	859.952	1.292	1.43		Unsaturated	0.0			172.99	1.70	294.08	294.08	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.820	181.370	2.795	98.4	98.4	794.735	1.541	1.52		Unsaturated	0.0			171.43	1.70	291.43	291.43	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.980	152.720	2.300	117.6	117.6	612.064	1.506	1.56		Unsaturated	0.0			144.35	1.70	245.39	245.39	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.150	111.810	2.646	138.0	138.0	413.566	2.368	1.81		Unsaturated	7.7			105.68	1.70	179.66	182.90	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.310	87.250	2.940	157.2	157.2	302.287	3.258	2.00		Unsaturated	22.6			82.47	1.70	140.19	189.54	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.480	89.870	3.043	177.6	177.6	292.911	3.389	2.02		Unsaturated	24.4			84.94	1.70	144.40	198.44	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.640	87.140	3.247	196.8	196.8	269.766	3.731	2.07		Unsaturated	28.7			82.36	1.70	140.02	201.58	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.800	89.770	3.683	216.0	216.0	265.249	4.108	2.11		Unsaturated	31.9			84.85	1.70	144.24	211.62	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.970	98.790	3.747	236.4	236.4	279.024	3.798	2.07		Unsaturated	28.7			93.37	1.70	156.74	223.89	1.00	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.130	104.420	3.762	255.6	255.6	283.624	3.608	2.05		Unsaturated	26.8			98.70	1.70	167.78	231.16	1.00	0.638	1.100	18.223	44.100	69.16	0.00	0.00
2.300	92.050	2.892	276.0	276.0	240.541	3.147	2.03		Sand	25.7			87.00	1.70	147.91	205.38	1.00	0.662	1.100	2.594	6.279	9.48	0.00	0.00
2.460	74.290	1.956	295.2	295.2	187.621	2.639	2.03		Sand	25.5			70.22	1.70	119.37	171.36	1.00	0.683	1.100	0.527	1.132	1.66	0.00	0.00
2.620	46.330	2.634	314.4	314.4	113.218	5.704	2.43		Sand	57.5			43.79	1.70	74.44	143.19	1.00	0.703	1.100	0.249	0.437	0.62	0.02	0.04
2.790	53.680	3.851	334.8	334.8	106.922	6.922	2.47		Sand	60.3			50.74	1.70	86.25	159.26	1.00	0.845	1.100	0.364	0.713	0.99	0.01	0.02
2.950	89.060	4.176	354.0	354.0	205.395	4.698	2.22		Sand	40.4			84.18	1.70	143.10	219.95	1.00	0.740	1.100	7.107	17.198	23.25	0.00	0.00
3.120	136.430	4.183	374.4	374.4	306.138	3.071	1.97		Sand	20.6			128.95	1.58	203.64	256.79	1.00	0.757	1.100	302.071	731.013	96.10	0.00	0.00
3.280	173.750	3.807	393.6	393.6	380.345	2.193	1.80		Sand	6.8			164.22	1.56	255.95	257.93	1.00	0.772	1.100	350.754	848.826	1100.13	0.00	0.00
3.440	185.650	3.557	412.8	412.8	396.839	1.918	1.74		Sand	2.0			175.47	1.54	270.06	270.06	1.00	0.786	1.100	198.047	4762.673	6062.84	0.00	0.00
3.610	198.420	3.592	433.2	433.2	414.037	1.812	1.71		Sand	0.0			187.54	1.52	284.99	284.99	1.00	0.789	1.100	24183.093	73220.00	0.00	0.00	0.00
3.770	227.550	3.682	452.4	452.4	464.682	1.620	1.64		Sand	0.0			215.08	1.50	323.11	323.11	1.00	0.811	1.100	#####	#####	#####	0.00	0.00
3.940	242.500	4.104	472.8	472.8	484.419	1.694	1.65		Sand	0.0			229.21	1.48	340.36	340.36	1.00	0.824	1.100	#####	#####	#####	0.00	0.00
4.100	227.770	5.139	492.0	492.0	445.981	2.258	1.77		Sand	0.0			215.28	1.47	316.34	316.62	1.00	0.834	1.100	#####	#####	#####	0.00	0.00
4.270	185.790	4.022	512.4	512.4	317.947	2.430	1.90		Sand	13.0			156.70	1.46	228.26	252.74	1.00	0.845	1.100	181.184	438.486	518.85	0.00	0.00
4.430	111.910	3.296	531.6	531.6	210.531	2.952	2.04		Sand	26.4			105.78	1.52	160.49	221.63	1.00	0.855	1.100	1.105	19.613	22.95	0.00	0.00
4.590	72.960	3.363	550.8	550.8	134.653	4.627	2.31		Sand	48.1			68.96	1.60	110.52	184.73	1.00	0.864	1.100	0.884	2.112	2.45	0.00	0.00
4.760	83.310	3.788	571.2	571.2	151.037	4.562	2.28		Sand	45.4			78.74	1.55	121.81	197.26	1.00	0.873	1.100	1.625	3.932	4.50	0.00	0.00
4.920	99.800	4.731	590.4	590.4	178.158	4.472	2.25		Sand	43.4			94.39	1.48	139.54	218.01	1.00	0.881	1.100	6.131	14.837	16.84	0.00	0.00
5.090	102.340	5.061	610.8	610.8	170.002	4.960	2.27		Sand	44.5			97.73	1.46	140.95	220.64	1.00	0.889	1.100	7.493	18.133	20.39	0.00	0.00
5.250	108.040	3.988	630.0	630.0	186.603	3.702	2.15		Sand	35.3			102.12	1.44	147.55	220.03	1.00	0.897	1.100	7.150	17.302	19.29	0.00	0.00
5.410	106.190	4.009	649.2	649.2	180.650	3.787	2.17		Sand	36.5			100.37	1.44	144.25	217.46	1.00	0.904	1.100	5.885	14.241	15.76	0.00	0.00
5.580	95.600	2.366	669.6	669.6	160.066	2.484	2.05		Sand	27.2			90.36	1.48	133.78	191.47	1.00	0.911	1.100	1.207	2.920	3.21	0.00	0.00
5.740	84.150	2.167	688.3	688.3	135.945	2.5854	2.10		Sand	13.0			79.54	1.46	118.76	179.99	1.00	0.917	1.100	1.726	1.688	1.82	0.00	0.00
5.910	58.800	2.536	709.2	709.2	95.420	4.339	2.38		Sand	53.5			55.58	1.54	85.49	155.74	1.00	0.924	1.100	0.331	0.634	0.69	0.02	0.04
6.070	89.690	3.177	728.4	728.4	143.931	3.556	2.20		Sand	39.3			84.77	1.44	121.78	192.49	0.99	0.930	1.100	1.269	3.071	3.30	0.00	0.00
6.230	178.640	3.601	747.6	747.6	283.183	2.022	1.83		Sand	9.7			208.68	1.34	226.56	236.78	0.99	0.935	1.100	31.070	75.189	80.40	0.00	0.00
6.400	219.670	3.416	768.0	768.0	344.035	1.558	1.69		Sand	0.0			267.63	1.31	271.27	271.27	0.99	0.941	1.100	2372.658	5742.316	6103.04	0.00	0.00
6.560	240.490	3.625	787.2	787.2	372.062	1.501	1.66		Sand	0.0			277.31	1.30	295.06	295.06	0.99	0.946	1.100	#####	#####	#####	0.00	0.00
6.730	259.850	4.040	807.6	807.6	396.397	1.561	1.66		Sand	0.0			245.60	1.29	316.66	316.66	0.99	0.951	1.100	#####	#####	#####	0.00	0.00
6.890	270.110	4.269	826.8	826.8	407.800	1.583	1.66		Sand	0.0			255.30	1.28	327.13	327.13	0.99	0.956	1.100	#####	#####	#####	0.00	0.00
7.050	274.030	4.553	846.3	843.1	409.687	1.684	1.68		Sand	0.0			259.01	1.27	330.17	330.17	0.99	0.960	1.100	#####	#####	#####	0.00	0.00
7.220	289.220	4.553	867.5	867.5	398.552	1.594	1.67		Sand	0.0			254.46	1.27	330.17	330.17	0.99	0.960	1.100	#####	#####	#####	0.00	0.00
7.380	287.160	4.344	887.5	883.8	394.585	1.629	1.68		Sand	0.0			252.51	1.27	318.85	318.85	0.99	0.969	1.100	#####	#####	#####	0.00	0.00
7.550	257.370	4.014	908.8	874.4	377.746	1.562	1.67		Sand	0.0			243.26	1.26	307.13	307.13	0.99	0.973	1.100	#####	#####	6100968.46	0.00	0.00
7.710	249.500	3.812	928.8	884.4	364.081	1.531	1.67		Sand	0.0			235.82	1.26	296.85	296.85	0.99	0.977	1.100	#####	#####	616079.11	0.00	0.00
7.870	265.740	4.117	948.8	894.5	387.085	1.546	1.66		Sand	0.0			252.12	1.26	316.42	316.42	0.99	0.981	1.100	#####	#####	#####	0.00	0.00
8.040	282.240	4.434	970.0	905.1	407.188	1.514	1.66		Sand	0.0			265.77	1.25	333.76	333.76	0.99	0.985	1.100	#####	#####	#####	0.00	0.00
8.200	278.870	4.304	990.0	915.1	400.095	1.546	1.66		Sand	0.0			263.58	1.25	328.82	328.82	0.99	0.988	1.100	#####	#####	#####	0.00	0.00
8.370	273.990	3.943	1011.3	925.8	390.800	1.442	1.63		Sand	0.0			258.97	1.24	322.08	322.08	0.99	0.992	1.100	#####	#####	#####	0.00	0.00
8.530	273.320	3.827	1031.3	935.8	387.737	1.403	1.63		Sand	0.0			258.34	1.24	320.38	320.38	0.99	0.995	1.100	#####	#####	#####	0.00	0

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Depth (ft)	Qc (tsf)	f _c (tsf)	C _u (psf)	Insitu C* (psf)	Q ₂₀	F (%)	lc	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q _h near interfaces (soil layer)	Thin Layer Factor (K _u)	Interpreted Q _h	C _N	Q _{h-N}	Q _{h-NCS}	Stress Reduction Coeff. f _s	CSR	K _e for Sand	CRR _{7.5} , σ _{vc} = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε _v	Settlement (Inches)
21.880	116.190	6.941	2712.5	1777.7	118.415	6.044	2.44	Dense	Dense Sand	58.2			109.82	1.06	116.39	197.06	0.95	1.081	1.044	1.607	3.692	3.41	0.00	0.00
22.150	129.830	7.683	2733.8	1788.4	132.075	5.981	2.41	Dense	Dense Sand	55.8			122.71	1.05	129.35	212.56	0.95	1.081	1.050	4.111	9.571	8.85	0.00	0.00
22.210	120.570	7.378	2753.8	1798.4	122.202	6.190	2.44	Dense	Dense Sand	58.3			113.96	1.05	120.18	201.93	0.95	1.081	1.044	2.140	4.846	4.48	0.00	0.00
22.470	119.520	6.452	2773.8	1808.4	120.780	5.462	2.40	Dense	Dense Sand	55.0			112.97	1.05	118.99	198.99	0.95	1.081	1.041	1.786	4.089	3.78	0.00	0.00
22.640	103.230	5.752	2795.0	1819.1	103.809	5.648	2.45	Dense	Dense Sand	59.0			97.57	1.06	102.98	180.20	0.95	1.082	1.032	0.732	1.581	1.46	0.00	0.00
22.800	96.950	5.511	2815.0	1829.1	97.130	5.768	2.47	Dense	Dense Sand	61.0			91.64	1.06	96.68	172.83	0.95	1.082	1.029	0.555	1.127	1.04	0.01	0.00
22.970	107.580	4.388	2836.3	1839.7	107.613	4.133	2.33	Dense	Dense Sand	49.6			101.68	1.05	106.87	180.97	0.95	1.082	1.030	0.755	1.638	1.51	0.00	0.00
23.130	116.390	3.230	2856.3	1849.7	116.217	2.809	2.18	Dense	Dense Sand	37.5			110.01	1.05	115.36	182.86	0.95	1.082	1.029	0.816	1.797	1.66	0.00	0.00
23.290	85.940	2.811	2876.3	1859.8	85.194	3.090	2.30	Dense	Dense Sand	47.1			81.23	1.05	85.52	152.66	0.95	1.082	1.021	0.307	0.533	0.49	0.02	0.00
23.460	67.730	1.977	2897.5	1870.4	66.634	2.982	2.36	Dense	Dense Sand	52.0			64.02	1.05	67.54	132.30	0.95	1.082	1.017	0.205	0.310	0.29	0.02	0.00
23.620	71.190	1.140	2917.5	1880.4	69.915	1.634	2.17	Dense	Dense Sand	36.4			67.29	1.05	70.90	127.08	0.95	1.082	1.016	0.189	0.277	0.26	0.02	0.00
23.790	76.010	0.720	2938.8	1891.1	74.527	0.966	2.00	Dense	Dense Sand	23.1			71.84	1.05	75.70	116.76	0.95	1.082	1.014	0.162	0.223	0.21	0.03	0.00
23.950	56.260	0.006	2959.8	1901.1	54.626	1.105	2.14	Dense	Dense Sand	34.5			53.19	1.05	56.00	107.21	0.95	1.081	1.012	0.148	0.194	0.19	0.03	0.00
24.110	46.570	0.520	2978.8	1911.1	44.835	1.153	2.22	Dense	Dense Sand	41.0			44.02	1.05	46.31	100.03	0.95	1.081	1.011	0.137	0.175	0.16	0.03	0.00
24.280	41.710	0.499	3000.0	1921.7	39.880	1.242	2.28	Dense	Dense Sand	45.8			39.42	1.05	41.39	96.50	0.95	1.081	1.010	0.133	0.167	0.15	0.03	0.00
24.440	37.750	0.429	3020.0	1931.7	35.850	1.183	2.31	Dense	Dense Sand	47.9			35.68	1.05	37.40	92.43	0.95	1.081	1.009	0.128	0.159	0.15	0.03	0.00
24.610	37.670	0.269	3040.0	1941.7	35.610	1.105	2.35	Dense	Dense Sand	46.7			35.60	1.05	37.22	91.68	0.95	1.081	1.008	0.127	0.157	0.15	0.04	0.00
24.770	40.440	0.425	3061.3	1952.4	38.286	1.092	2.27	Dense	Dense Sand	44.4			38.22	1.04	39.83	93.87	0.94	1.081	1.008	0.130	0.161	0.15	0.03	0.00
24.930	37.540	0.399	3081.3	1962.4	35.332	1.109	2.30	Dense	Dense Sand	47.1			35.48	1.04	36.89	91.44	0.94	1.081	1.007	0.127	0.156	0.14	0.04	0.00
25.100	38.530	0.445	3102.5	1973.1	36.195	1.204	2.31	Dense	Dense Sand	47.9			36.42	1.04	37.75	92.91	0.94	1.081	1.007	0.129	0.159	0.15	0.03	0.00
25.260	37.680	0.502	3122.5	1983.1	35.264	1.389	2.36	Dense	Dense Sand	51.5			35.61	1.03	36.82	93.17	0.94	1.081	1.007	0.129	0.160	0.15	0.03	0.00
25.430	41.430	0.629	3143.0	1993.7	41.722	1.564	2.31	Dense	Dense Sand	47.4			41.98	1.03	45.30	102.92	0.94	1.079	1.002	0.140	0.179	0.17	0.03	0.00
25.590	42.840	0.766	3163.8	2003.7	40.074	1.856	2.39	Dense	Dense Sand	54.0			40.49	1.03	41.61	100.14	0.94	1.081	1.006	0.137	0.175	0.16	0.03	0.00
25.750	49.290	0.683	3183.8	2013.8	46.214	1.432	2.27	Dense	Dense Sand	44.6			46.59	1.02	47.73	103.85	0.94	1.080	1.005	0.143	0.184	0.17	0.03	0.00
25.920	58.640	0.702	3205.0	2024.4	55.117	1.230	2.17	Dense	Dense Sand	36.5			55.43	1.02	56.61	109.63	0.94	1.080	1.005	0.151	0.200	0.19	0.03	0.00
26.080	56.380	7.786	3225.0	2034.4	52.793	1.435	2.22	Dense	Dense Sand	41.0			54.39	1.02	54.30	109.86	0.94	1.080	1.005	0.152	0.201	0.19	0.03	0.00
26.250	50.890	0.092	3246.3	2045.1	47.367	1.405	2.26	Dense	Dense Sand	43.7			48.10	1.02	48.01	104.74	0.94	1.080	1.004	0.144	0.186	0.17	0.03	0.00
26.410	51.850	0.664	3266.3	2055.1	48.162	1.322	2.23	Dense	Dense Sand	41.8			49.01	1.01	49.71	104.74	0.94	1.080	1.003	0.144	0.186	0.17	0.03	0.00
26.570	44.510	0.649	3286.3	2065.1	41.013	1.513	2.33	Dense	Dense Sand	49.0			42.07	1.01	42.58	99.48	0.94	1.080	1.003	0.137	0.172	0.16	0.03	0.00
26.740	46.360	0.637	3307.5	2075.7	42.963	1.424	2.30	Dense	Dense Sand	46.7			43.82	1.01	44.24	100.51	0.94	1.080	1.002	0.138	0.175	0.16	0.03	0.00
26.900	47.590	0.626	3327.5	2085.7	43.722	1.522	2.31	Dense	Dense Sand	47.4			44.98	1.01	45.30	102.92	0.94	1.079	1.002	0.140	0.179	0.17	0.03	0.00
27.070	51.090	0.745	3348.8	2096.4	46.925	1.508	2.28	Dense	Dense Sand	45.3			48.29	1.00	48.51	105.17	0.94	1.079	1.001	0.144	0.186	0.17	0.03	0.00
27.230	57.290	0.778	3368.8	2106.4	52.677	1.400	2.22	Dense	Dense Sand	40.5			54.15	1.00	54.27	109.62	0.94	1.079	1.001	0.151	0.199	0.18	0.03	0.00
27.400	61.900	0.825	3390.0	2117.0	56.286	1.519	2.22	Dense	Dense Sand	40.5			57.90	1.00	57.89	114.10	0.94	1.079	1.000	0.159	0.214	0.20	0.03	0.00
27.560	61.230	1.216	3410.0	2127.1	56.115	2.042	2.30	Dense	Dense Sand	47.2			57.73	1.00	57.73	117.75	0.94	1.079	0.999	0.166	0.228	0.21	0.03	0.00
27.720	70.430	0.836	3430.3	2137.1	66.547	2.303	2.30	Dense	Dense Sand	47.2			68.57	0.99	68.28	128.52	0.94	1.079	0.998	0.183	0.280	0.26	0.00	0.00
27.890	83.310	3.334	3451.3	2147.7	76.634	4.081	2.42	Dense	Dense Sand	56.8			78.84	0.99	78.36	147.93	0.93	1.078	0.998	0.275	0.452	0.42	0.02	0.00
28.050	99.340	9.609	3471.3	2157.7	91.357	5.747	2.49	Dense	Dense Sand	62.1			93.75	0.99	93.21	168.77	0.93	1.078	0.996	0.483	0.922	0.86	0.01	0.00
28.220	156.320	9.834	3492.5	2168.4	144.325	6.362	2.41		Sand	55.9			147.89	0.99	146.72	234.74	0.93	1.078	0.993	25.477	55.639	51.63	0.00	0.00
28.380	234.820	9.832	3513.0	2178.4	217.466	4.218	2.17		Sand	55.9			222.00	0.99	220.31	310.69	0.93	1.077	0.991	110.89	440.22	400.22	0.00	0.00
28.540	305.170	10.911	3532.5	2188.4	281.987	3.596	2.05		Sand	26.8			288.44	0.99	285.89	371.19	0.93	1.077	0.990	*****	*****	*****	0.00	0.00
28.710	386.980	12.631	3553.8	2199.0	357.145	2.739	1.96		Sand	20.1			365.77	0.99	362.07	434.75	0.93	1.077	0.988	*****	*****	*****	0.00	0.00
28.870	381.440	11.938	3573.8	2209.1	351.201	3.145	1.95		Sand	19.0			360.53	0.99	356.46	423.45	0.93	1.077	0.987	*****	*****	*****	0.00	0.00
29.040	396.360	11.534	3593.0	2219.7	356.179	2.923	1.92		Sand	16.2			374.63	0.99	369.83	423.03	0.93	1.076	0.986	*****	*****	*****	0.00	0.00
29.200	388.380	11.951	3613.0	2229.7	355.941	1.957	1.92		Sand	9.9			362.05	0.99	362.05	415.66	0.93	1.076	0.984	*****	*****	*****	0.00	0.00
29.360	405.430	10.765	3635.0	2239.7	370.799	2.667	1.88		Sand	13.1			383.20	0.99	377.50	411.99	0.93	1.076	0.983	*****	*****	*****	0.00	0.00
29.530	414.070	10.248	3656.3	2250.4	377.830	2.486	1.85		Sand	10.7			391.37	0.98	385.06	404.56	0.93	1.076	0.982	*****	*****	*****	0.00	0.00
29.690	434.760	10.081	3676.3	2260.4	395.904	2.329	1.81		Sand	7.9			410.93	0.98	403.83	409.90	0.93	1.075	0.980	*****	*****	*****	0.00	0.00
29.860	459.870	9.921	3697.5	2271.0	414.174	2.242	1.79		Sand	4.6			436.66	0.98	428.53	427.73	0.93	1.075	0.979	*****	*****	*****	0.00	0.00
30.020	475.200	10.217	3717.5	2281.1	430.902	2.158	1.76		Sand	4.2			449.15	0.98	440.34	440.40	0.93	1.075	0.977	*****	*****	*****	0.00	0.00
30.180	478.250	11.287	3737.5	2291.1	432.721	2.369	1.80		Sand	7.0			452.03											

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Depth (ft)	Qc (tsf)	f _c (tsf)	σ _{vc} (psf)	Insitu σ _{vc} (psf)	Q	F (%)	Is	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q _N near interface (soft layer)	Thin Layer Factor (F _{ti})	Interpreted Q _N	C _N	Q _{c1N}	Q _{c1NCS}	Stress Reduction Coeff. f _d	CSR	K _e for Sand	CRR _{avg} 7.5, σ _{vc} = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε _v	Settlement (inches)
43.800	369.790	11.260	5440.0	3143.7	284.644	3.068	1.99		Sand	21.9			349.52	0.90	314.86	388.75	0.88	1.036	0.881	0.000	0.000	0.000	0.00	0.00
43.960	396.240	11.661	5460.0	3153.7	304.662	2.963	1.96		Sand	19.7			374.52	0.90	337.09	404.57	0.88	1.035	0.880	0.000	0.000	0.000	0.00	0.00
44.130	417.200	12.275	5481.3	3164.3	320.341	2.962	1.95		Sand	18.8			394.33	0.90	354.61	420.00	0.88	1.034	0.879	0.000	0.000	0.000	0.00	0.00
44.290	444.200	12.435	5501.3	3174.4	340.663	2.817	1.92		Sand	16.2			419.85	0.90	377.24	430.97	0.87	1.034	0.878	0.000	0.000	0.000	0.00	0.00
44.460	473.790	13.123	5522.5	3185.0	362.882	2.786	1.90		Sand	14.8			447.82	0.90	402.02	449.22	0.87	1.033	0.877	0.000	0.000	0.000	0.00	0.00
44.620	490.990	13.554	5542.5	3195.0	375.535	2.776	1.89		Sand	14.1			464.07	0.90	416.27	460.09	0.87	1.033	0.876	0.000	0.000	0.000	0.00	0.00
44.780	488.160	11.928	5562.5	3205.0	372.767	2.458	1.84		Sand	10.5			461.40	0.90	413.53	433.26	0.87	1.032	0.875	0.000	0.000	0.000	0.00	0.00
44.950	461.130	11.615	5583.8	3215.7	351.417	2.534	1.87		Sand	12.5			435.85	0.90	390.29	421.38	0.87	1.032	0.874	0.000	0.000	0.000	0.00	0.00
45.110	440.810	10.781	5603.8	3225.7	335.308	2.461	1.87		Sand	12.4			416.64	0.89	372.78	402.52	0.87	1.031	0.874	0.000	0.000	0.000	0.00	0.00
45.280	385.690	10.129	5625.0	3236.3	292.621	2.645	1.92		Sand	17.0			364.55	0.89	325.89	378.80	0.87	1.030	0.873	0.000	0.000	0.000	0.00	0.00
45.440	384.240	9.639	5645.0	3246.3	291.055	2.527	1.91		Sand	15.7			363.18	0.89	324.40	370.43	0.87	1.030	0.872	0.000	0.000	0.000	0.00	0.00
45.600	363.600	9.084	5665.0	3256.4	274.874	2.518	1.92		Sand	16.7			343.67	0.89	306.72	356.05	0.87	1.029	0.871	0.000	0.000	0.000	0.00	0.00
45.770	339.470	8.412	5686.3	3267.0	256.063	2.499	1.94		Sand	17.8			320.86	0.89	286.12	338.65	0.87	1.029	0.870	0.000	0.000	0.000	0.00	0.00
45.930	317.280	7.989	5706.3	3277.0	238.810	2.541	1.96		Sand	19.6			299.89	0.89	267.20	325.30	0.87	1.028	0.869	0.000	0.000	0.000	0.00	0.00
46.100	310.670	7.260	5727.5	3287.7	233.403	2.359	1.94		Sand	17.9			293.64	0.89	261.41	311.71	0.87	1.027	0.868	0.000	0.000	0.000	0.00	0.00
46.260	300.060	7.243	5747.5	3297.7	225.007	2.437	1.96		Sand	19.6			283.61	0.89	252.28	308.30	0.87	1.027	0.867	0.000	0.000	0.000	0.00	0.00
46.420	285.620	7.928	5767.5	3307.7	213.743	2.804	2.02		Sand	24.6			269.96	0.89	239.95	310.89	0.87	1.026	0.866	0.000	0.000	0.000	0.00	0.00
46.590	245.710	8.511	5788.8	3318.3	183.089	3.505	2.14		Sand	34.0			232.24	0.89	206.25	290.18	0.87	1.025	0.865	0.000	0.000	0.000	0.00	0.00
46.750	186.760	8.219	5808.8	3328.4	138.559	4.470	2.29		Sand	46.5			176.52	0.88	155.55	240.44	0.87	1.025	0.864	45.005	85.557	83.48	0.00	0.00
46.920	170.750	9.064	5830.0	3339.0	126.284	5.400	2.38		Sand	53.7			161.39	0.87	141.01	226.42	0.86	1.024	0.863	12.006	22.798	22.26	0.00	0.00
47.080	197.770	7.134	5850.0	3349.0	146.387	3.661	2.21		Sand	39.8			186.93	0.88	164.96	246.47	0.86	1.024	0.862	86.531	164.148	160.35	0.00	0.00
47.240	147.540	4.913	5870.0	3359.0	108.480	3.397	2.26		Sand	44.1			139.45	0.86	119.24	193.15	0.86	1.023	0.868	1.312	2.561	2.50	0.00	0.00
47.410	80.420	4.829	5891.3	3369.7	45.983	6.233	2.71		Clay	79.5			76.01	0.88	n.a.	n.a.	0.86	1.023	n.a.	n.a.	n.a.	0.00	0.00	0.00
47.570	74.060	4.626	5911.3	3379.7	42.078	6.506	2.75		Clay	82.7			70.00	0.88	n.a.	n.a.	0.86	1.022	n.a.	n.a.	n.a.	0.00	0.00	0.00
47.740	95.650	4.721	5932.5	3390.3	69.208	5.094	2.52		Sand	64.9	127.78		127.78	0.85	108.66	189.49	0.86	1.021	0.890	1.007	2.148	2.10	0.00	0.00
47.900	135.190	3.987	5952.5	3400.3	98.580	3.016	2.25		Sand	43.1			127.78	0.84	107.72	178.00	0.86	1.021	0.902	0.671	1.245	1.22	0.00	0.00
48.060	103.040	3.950	5972.5	3410.4	74.491	3.948	2.42		Sand	56.5	127.78		127.78	0.85	108.18	185.89	0.86	1.020	0.893	0.931	1.821	1.79	0.00	0.00
48.230	77.440	3.798	5993.8	3421.0	43.521	5.102	2.66		Clay	75.7			73.19	0.88	n.a.	n.a.	0.86	1.019	n.a.	n.a.	n.a.	0.00	0.00	0.00
48.390	100.250	3.223	6013.8	3431.0	72.180	3.315	2.37		Sand	52.8		1.8	170.56	0.87	148.61	235.56	0.86	1.019	0.855	27.563	51.846	50.89	0.00	0.00
48.560	80.390	3.168	6035.0	3441.7	57.342	4.094	2.51		Sand	63.6	94.75		170.55	0.87	148.92	240.78	0.86	1.018	0.854	46.659	87.670	86.11	0.00	0.00
48.720	62.980	2.563	6055.0	3451.7	34.738	4.275	2.67		Clay	76.9			59.53	0.88	n.a.	n.a.	0.86	1.017	n.a.	n.a.	n.a.	0.00	0.00	0.00
48.880	60.810	2.509	6075.0	3461.7	33.378	4.344	2.69		Clay	78.3			57.48	0.88	n.a.	n.a.	0.86	1.017	n.a.	n.a.	n.a.	0.00	0.00	0.00
49.050	59.010	1.964	6096.3	3472.3	32.233	3.510	2.64		Clay	74.1			55.78	0.88	n.a.	n.a.	0.86	1.016	n.a.	n.a.	n.a.	0.00	0.00	0.00
49.210	57.580	1.878	6116.3	3482.3	31.313	3.444	2.64		Clay	74.4			54.42	0.88	n.a.	n.a.	0.86	1.016	n.a.	n.a.	n.a.	0.00	0.00	0.00
49.380	60.310	2.389	6137.5	3493.0	32.775	4.173	2.58		Clay	77.8			57.00	0.88	n.a.	n.a.	0.86	1.015	n.a.	n.a.	n.a.	0.00	0.00	0.00
49.540	59.410	2.660	6157.5	3503.0	32.162	4.722	2.73		Clay	81.2			56.15	0.88	n.a.	n.a.	0.85	1.014	n.a.	n.a.	n.a.	0.00	0.00	0.00
49.700	67.620	3.253	6177.5	3513.0	36.738	5.041	2.71		Clay	79.5			63.91	0.87	n.a.	n.a.	0.85	1.014	n.a.	n.a.	n.a.	0.00	0.00	0.00
49.870	71.620	3.534	6198.8	3523.7	38.892	5.158	2.70		Clay	78.7			67.69	0.87	n.a.	n.a.	0.85	1.013	n.a.	n.a.	n.a.	0.00	0.00	0.00
50.030	84.080	3.157	6218.8	3533.7	59.222	3.899	2.48		Sand	61.6			79.47	0.80	63.52	130.51	0.85	1.012	0.930	0.199	0.272	0.27	0.02	0.00
50.200	79.760	2.797	6240.0	3544.3	55.971	3.650	2.48		Sand	61.3			75.39	0.79	59.90	125.78	0.85	1.012	0.933	0.185	0.247	0.24	0.03	0.00
50.360	72.230	2.652	6260.0	3554.3	38.882	3.836	2.61		Clay	71.5			68.27	0.87	n.a.	n.a.	0.85	1.011	n.a.	n.a.	n.a.	0.00	0.00	0.00
50.520	64.220	1.763	6280.0	3564.4	44.482	2.886	2.48		Sand	61.3			60.70	0.78	47.32	109.66	0.85	1.010	0.940	0.151	0.187	0.19	0.03	0.00

CPT No. 4

PGA (A_{max}) 0.95

Total Settlement: 0.37 (Inches)

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Depth (ft)	Qc (tsf)	f _s (tsf)	C _{vc} (psf)	In situ σ' (psf)	Q	F (%)	I _c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q ₀ h near interfaces (soil layer)	Thin Layer Factor (K _u)	Interpreted Q ₀ h	C _N	Q ₀ N	Q ₀ N-C5	Stress Reduction Coeff. f _s	CSR	K _s for Sand	CRR _N >7.5, σ' _{vc} = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain E _v	Settlement (Inches)
0.160	319.720	1.149	19.2	19.2	3172.330	0.360	0.78		Unsaturated	0.0			302.19	1.70	513.73	513.73	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.330	216.390	1.331	39.6	39.6	1494.934	0.615	1.05		Unsaturated	0.0			204.53	1.70	347.70	347.70	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.490	137.530	1.369	58.8	58.8	778.324	0.996	1.35		Unsaturated	0.0			170.20	1.70	220.98	220.98	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.660	66.900	1.371	79.2	79.2	326.647	2.051	1.81		Unsaturated	5.5			63.23	1.70	107.50	109.74	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.820	43.670	1.253	98.4	98.4	191.191	2.873	2.06		Unsaturated	27.5			41.28	1.70	70.17	116.44	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
0.980	42.900	1.000	117.6	117.6	171.763	2.334	2.01		Unsaturated	23.9			40.55	1.70	68.93	109.40	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.150	38.910	0.698	138.0	138.0	143.755	1.797	1.97		Unsaturated	20.9			36.78	1.70	62.52	96.35	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.310	36.510	0.400	157.2	157.2	126.334	1.098	1.86		Unsaturated	11.9			34.51	1.70	58.66	69.83	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.480	35.210	0.253	177.6	177.6	114.583	0.719	1.77		Unsaturated	5.0			33.28	1.70	56.58	56.70	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.640	33.950	0.205	196.8	196.8	104.915	0.606	1.76		Unsaturated	4.0			32.09	1.70	54.55	54.57	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.800	29.470	0.209	216.0	216.0	86.862	0.711	1.87		Unsaturated	12.5			27.85	1.70	47.35	59.66	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
1.970	26.210	0.214	236.4	236.4	73.782	0.820	1.96		Unsaturated	20.0			24.77	1.70	42.11	71.33	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.130	23.380	0.209	255.6	255.6	63.235	0.972	2.04		Unsaturated	28.3			22.10	1.70	37.57	73.05	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.300	23.110	0.201	276.0	276.0	60.120	0.873	2.05		Unsaturated	27.1			21.84	1.70	37.13	76.60	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.460	25.020	0.224	295.2	295.2	62.941	0.901	2.04		Unsaturated	26.4			23.65	1.70	40.20	79.39	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.620	30.400	0.244	314.4	314.4	74.157	0.806	1.96		Unsaturated	19.5			28.73	1.70	48.85	78.00	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.790	34.290	0.253	333.2	333.2	81.081	0.742	1.90		Unsaturated	15.3			32.41	1.70	55.10	75.21	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
2.950	30.460	0.244	354.0	354.0	69.979	0.806	1.98		Unsaturated	21.2			28.79	1.70	49.94	81.30	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.120	28.300	0.242	374.4	374.4	63.170	0.861	2.03		Unsaturated	25.4			26.75	1.70	45.47	84.22	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.280	27.900	0.243	393.6	393.6	60.712	0.878	2.05		Unsaturated	26.9			26.37	1.70	44.83	85.52	1.00	0.618	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.440	26.530	0.248	412.8	412.8	60.611	0.877	2.05		Unsaturated	27.0			26.97	1.70	45.84	86.76	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.610	29.490	0.258	433.2	433.2	61.121	0.882	2.05		Unsaturated	28.8			27.87	1.70	48.88	88.40	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.770	31.170	0.253	452.4	452.4	63.253	0.819	2.02		Unsaturated	24.4			29.46	1.70	50.08	88.09	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
3.940	34.240	0.219	472.8	472.8	67.992	0.644	1.93		Unsaturated	17.7			32.36	1.70	55.02	80.95	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.100	38.860	0.219	492.0	492.0	75.689	0.597	1.87		Unsaturated	12.2			36.73	1.70	62.44	74.74	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.270	46.480	0.257	512.4	512.4	89.784	0.556	1.80		Unsaturated	7.1			43.93	1.70	74.68	76.25	1.00	0.617	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.430	54.320	0.315	531.6	531.6	101.932	0.582	1.76		Unsaturated	4.0			51.34	1.70	87.28	67.30	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.590	58.730	0.378	550.8	550.8	108.291	0.646	1.77		Unsaturated	4.4			55.51	1.70	94.37	94.41	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.760	61.560	0.404	571.2	571.2	111.470	0.660	1.76		Unsaturated	5.0			58.19	1.70	98.91	98.93	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
4.920	63.510	0.447	590.4	590.4	113.114	0.708	1.78		Unsaturated	5.0			60.03	1.70	103.05	102.21	1.00	0.616	1.00	n.a.	n.a.	n.a.	0.00	0.00
5.090	64.540	0.496	610.8	610.8	113.003	0.772	1.80		Sand	6.9			61.00	1.70	103.70	105.13	1.00	0.616	1.00	0.144	0.235	0.33	0.03	0.06
5.250	68.010	0.598	630.0	630.0	117.262	0.883	1.82		Sand	8.8			64.28	1.70	109.28	114.17	1.00	0.631	1.00	0.159	0.236	0.37	0.03	0.05
5.410	64.920	0.486	649.2	649.2	106.890	0.763	1.81		Sand	7.7	64.28		64.28	1.70	109.28	111.93	1.00	0.640	1.00	0.155	0.227	0.35	0.03	0.06
5.580	48.000	0.422	669.6	669.6	81.751	0.867	1.94		Sand	18.3	64.28		64.28	1.63	104.70	137.79	1.00	0.650	1.00	0.225	0.381	0.59	0.02	0.04
5.740	34.750	0.577	688.3	688.3	167.7	0.773	2.24		Sand	42.3	64.28		64.28	1.53	98.29	165.06	0.99	0.678	1.00	0.438	0.932	1.37	0.00	0.00
5.910	28.080	0.763	709.2	709.2	45.285	2.751	2.46		Sand	59.0	64.28		64.28	1.50	96.23	171.82	1.00	0.668	1.00	0.536	1.155	1.73	0.00	0.00
6.070	29.830	0.827	728.4	728.4	47.468	2.807	2.45		Sand	59.7	64.28		64.28	1.49	95.48	170.59	0.99	0.676	1.00	0.514	1.097	1.62	0.00	0.00
6.230	25.020	0.876	747.6	747.6	39.605	3.516	2.57		Sand	68.9	64.28		64.28	1.47	94.33	172.15	0.99	0.684	1.00	0.542	1.171	1.71	0.00	0.00
6.400	23.610	0.987	768.0	768.0	44.627	4.248	2.59		plastic Clay	70.5			22.32	1.31	n.a.	n.a.	0.99	0.692	n.a.	n.a.	n.a.	n.a.	0.00	0.00
6.560	28.040	1.136	787.2	787.2	42.842	4.773	2.60		Sand	70.7	32.17		69.88	1.58	102.47	158.61	0.99	0.699	1.00	0.160	0.245	0.35	0.03	0.05
6.730	30.400	1.221	807.6	807.6	51.461	3.630	2.50		Sand	63.2			32.17	1.57	50.50	114.24	0.99	0.707	1.00	0.159	0.236	0.33	0.03	0.05
6.890	36.100	1.276	826.8	826.8	65.118	0.697	2.60		plastic Clay	71.1			34.12	1.28	n.a.	n.a.	0.99	0.714	n.a.	n.a.	n.a.	n.a.	0.00	0.00
7.050	33.230	1.945	846.0	846.0	59.909	5.929	2.62		plastic Clay	72.6			31.41	1.27	n.a.	n.a.	0.99	0.721	n.a.	n.a.	n.a.	n.a.	0.00	0.00
7.220	41.940	2.335	865.4	865.4	66.997	3.903	2.55		Sand	65.7	131.83		131.83	1.27	165.95	264.96	0.99	0.728	1.00	922.070	2231.409	3086.37	0.00	0.00
7.380	43.490	2.318	885.6	885.6	62.892	3.595	2.57		Sand	68.5	131.83		131.83	1.25	165.89	264.36	0.99	0.734	1.00	846.013	2047.352	2788.99	0.00	0.00
7.550	62.950	2.919	906.0	906.0	90.275	4.671	2.42		Sand	56.7	131.83		131.83	1.25	164.89	258.32	0.99	0.741	1.00	369.116	893.260	1206.03	0.00	0.00
7.710	97.930	3.589	925.2	925.2	139.320	6.882	2.22		Sand	41.0	131.83		131.83	1.25	164.96	247.62	0.99	0.747	1.00	98.693	238.838	319.87	0.00	0.00
7.870	139.480	3.090	944.4	944.4	196.688	2.223	1.96		Sand	19.7			131.83	1.28	169.32	214.89	0.99	0.753	1.00	4.878	11.805	15.69	0.00	0.00
8.040	138.950	4.137	964.8	964.8	212.921	2.987	2.07		Sand	28.4			165.00	1.33	266.00	230.85	0.99	0.759	1.00	17.719	42.880	56.53	0.00	0.00
8.200	124.840	5.081	984.0	984.0	172.351	4.086	2.21		Sand	39.6			118.00	1.26	148.14	225.46	0.99	0.764	1.00	11.073	26.797	35.07	0.00	0.00
8.370	172.840	5.661	1004.4	1004.4	236.428	3.285	2.05		Sand	27.3			163.36	1.22	198.85	269.03	0.99	0.770	1.00	1680.475	4086.749	5282.97	0.00	0.00
8.530	147.470	5.934	1023.6	1023.6	199.710	4.038	2.17		Sand	36.5			139.39	1.22	169.57	248.55	0.99	0.775	1.00	109.897	265.950	343.16	0.00	0.00
8.690	129.120	4.465	1042.8	1042.8	173.144	4.472	2.15		Sand	49.4			150.84	1.29	204.61	232.61	0.99	0.780	1.00	1.504	22.969	29.48	0.00	

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Depth (ft)	Qc (tsf)	f _u (tsf)	C _u (psf)	In situ C _u (psf)	Q	F (%)	I _c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q _u near interfaces (soil layer)	Thin Layer Factor (K _u)	Interpreted Q _u	C _u	Q _{u-N}	Q _{u-15}	Stress Reduction Coeff. f _s	CSR	K _u for Sand	CRR _{0.01} >7.5, cyclic 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain E _v	Settlement (Inches)
21.980	71.880	1.994	2692.5	2007.3	68.448	2.826	2.34	plastic	Clay	50.0	67.94	1.01	n.a.	n.a.	n.a.	0.95	0.964	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
22.150	58.910	1.352	2713.8	2018.0	55.703	2.348	2.35	plastic	Clay	50.7	55.68	1.01	n.a.	n.a.	n.a.	0.95	0.965	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
22.310	40.760	0.722	2733.8	2028.0	38.033	1.833	2.40	plastic	Clay	55.2	38.53	1.01	n.a.	n.a.	n.a.	0.95	0.966	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
22.470	34.150	0.471	2753.8	2038.0	31.563	1.436	2.40	plastic	Clay	55.3	32.28	1.01	n.a.	n.a.	n.a.	0.95	0.968	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
22.640	32.320	0.678	2775.0	2048.7	29.713	2.191	2.53	plastic	Clay	65.8	30.55	1.01	n.a.	n.a.	n.a.	0.95	0.967	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
22.800	29.650	0.673	2795.0	2058.7	27.073	2.382	2.59	plastic	Clay	70.1	28.02	1.01	n.a.	n.a.	n.a.	0.95	0.968	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
22.970	42.070	0.849	2816.3	2069.3	38.864	2.088	2.43	plastic	Clay	57.4	39.76	1.01	n.a.	n.a.	n.a.	0.95	0.968	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
23.130	35.590	1.031	2836.3	2079.3	32.582	3.018	2.59	plastic	Clay	70.4	33.64	1.00	n.a.	n.a.	n.a.	0.95	0.969	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
23.290	37.700	1.082	2856.3	2089.4	34.501	2.982	2.57	plastic	Clay	68.6	35.63	1.00	n.a.	n.a.	n.a.	0.95	0.970	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
23.460	34.850	1.361	2877.5	2100.0	31.820	4.075	2.69	plastic	Clay	78.0	32.94	1.00	n.a.	n.a.	n.a.	0.95	0.970	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
23.620	34.980	1.797	2897.5	2110.0	31.783	3.538	2.77	plastic	Clay	84.6	33.06	1.00	n.a.	n.a.	n.a.	0.95	0.971	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
23.790	40.760	2.396	2918.8	2120.7	37.085	6.098	2.76	plastic	Clay	84.0	38.53	1.00	n.a.	n.a.	n.a.	0.95	0.972	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
23.950	51.180	3.137	2938.8	2130.7	46.662	6.310	2.71	plastic	Clay	79.5	48.37	1.00	n.a.	n.a.	n.a.	0.95	0.972	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
24.110	63.400	3.025	2958.8	2140.7	58.188	4.885	2.56	plastic	Clay	67.8	59.92	1.00	n.a.	n.a.	n.a.	0.95	0.973	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
24.280	71.540	3.127	2980.0	2151.3	65.664	4.464	2.50	plastic	Clay	62.6	67.62	1.00	n.a.	n.a.	n.a.	0.95	0.973	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
24.440	66.480	2.401	3000.0	2161.3	60.770	3.694	2.46	plastic	Clay	59.6	62.84	0.99	n.a.	n.a.	n.a.	0.95	0.974	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
24.610	52.230	0.940	3020.0	2171.3	47.970	3.971	2.69	plastic	Clay	78.1	49.37	0.99	n.a.	n.a.	n.a.	0.95	0.974	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
24.770	73.670	3.542	3041.3	2182.0	67.155	4.909	2.52	plastic	Clay	64.6	69.63	0.99	n.a.	n.a.	n.a.	0.94	0.975	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
24.930	69.280	4.085	3061.3	2192.0	61.815	6.300	2.61	plastic	Clay	71.9	65.48	0.99	n.a.	n.a.	n.a.	0.94	0.976	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
25.100	113.780	4.461	3082.5	2202.7	103.978	3.975	2.33	plastic	Clay	49.3	107.54	0.99	n.a.	n.a.	n.a.	0.94	0.977	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
25.260	92.990	3.653	3102.5	2212.7	84.517	3.995	2.39	plastic	Clay	54.0	87.89	0.99	n.a.	n.a.	n.a.	0.94	0.977	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
25.430	57.670	0.940	3123.8	2223.3	50.472	3.022	2.71	plastic	Clay	60.5	54.51	0.99	n.a.	n.a.	n.a.	0.94	0.977	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
25.590	50.760	4.742	3143.8	2233.3	44.049	6.640	2.86	plastic	Clay	92.0	47.98	0.99	n.a.	n.a.	n.a.	0.94	0.977	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
25.750	99.570	6.136	3163.8	2243.4	89.949	6.262	2.52	plastic	Clay	64.8	94.11	0.98	n.a.	n.a.	n.a.	0.94	0.978	n.a.	n.a.	n.a.	n.a.	0.00	0.00	
25.920	177.460	7.280	3185.0	2254.0	161.058	4.139	2.23	plastic	Sand	41.3	167.73	0.98	164.89	342.03	0.94	0.978	0.981	101.749	219.604	224.46	0.00	0.00		
26.080	206.080	7.746	3201.3	2264.0	241.670	2.929	2.01	plastic	Sand	44.4	257.49	0.98	247.05	315.93	0.94	0.979	0.980	167.7254	167.7254	0.00	0.00			
26.250	325.700	4.902	3226.3	2274.7	295.445	1.513	1.72	plastic	Sand	0.6	307.84	0.98	302.03	302.03	0.94	0.979	0.978	0.978	0.978	0.978	0.978	0.00	0.00	
26.410	368.990	5.016	3246.3	2284.7	334.165	1.365	1.65	plastic	Sand	0.0	348.76	0.98	341.78	341.78	0.94	0.980	0.977	0.977	0.977	0.977	0.977	0.00	0.00	
26.570	406.690	7.685	3266.3	2294.7	367.644	1.897	1.75	plastic	Sand	3.0	384.40	0.98	376.26	376.26	0.94	0.980	0.976	0.976	0.976	0.976	0.976	0.00	0.00	
26.740	402.600	8.545	3287.5	2305.3	363.081	2.131	1.80	plastic	Sand	6.7	380.53	0.98	372.02	374.43	0.94	0.980	0.974	0.974	0.974	0.974	0.974	0.00	0.00	
26.900	411.190	8.617	3307.5	2315.3	370.047	2.104	1.79	plastic	Sand	0.0	378.53	0.98	378.53	380.72	0.94	0.981	0.973	0.973	0.973	0.973	0.973	0.00	0.00	
27.070	445.890	8.355	3328.8	2326.0	400.472	1.881	1.73	plastic	Sand	1.2	421.45	0.98	411.06	411.06	0.94	0.981	0.972	0.972	0.972	0.972	0.972	0.00	0.00	
27.230	461.560	9.139	3348.8	2336.0	413.700	1.987	1.74	plastic	Sand	2.3	436.26	0.97	425.02	425.02	0.94	0.981	0.970	0.970	0.970	0.970	0.970	0.00	0.00	
27.400	478.700	8.233	3370.0	2346.6	428.135	1.726	1.68	plastic	Sand	0.0	452.48	0.97	440.27	440.27	0.94	0.982	0.969	0.969	0.969	0.969	0.969	0.00	0.00	
27.560	508.360	8.854	3390.0	2356.7	453.771	1.767	1.68	plastic	Sand	0.0	480.48	0.97	467.02	467.02	0.94	0.982	0.968	0.968	0.968	0.968	0.968	0.00	0.00	
27.720	520.150	9.357	3410.0	2366.7	463.346	1.805	1.68	plastic	Sand	0.0	497.64	0.97	477.33	477.33	0.94	0.983	0.965	0.965	0.965	0.965	0.965	0.00	0.00	
27.890	503.570	7.888	3431.3	2377.3	447.514	1.751	1.68	plastic	Sand	0.0	475.96	0.97	461.56	461.56	0.93	0.983	0.965	0.965	0.965	0.965	0.965	0.00	0.00	
28.050	483.210	8.690	3451.3	2387.3	428.448	1.805	1.70	plastic	Sand	0.0	456.72	0.97	442.41	442.41	0.93	0.983	0.964	0.964	0.964	0.964	0.964	0.00	0.00	
28.220	462.660	8.815	3475.2	2398.0	400.152	1.913	1.73	plastic	Sand	1.4	437.20	0.97	423.01	423.01	0.93	0.983	0.962	0.962	0.962	0.962	0.962	0.00	0.00	
28.380	480.590	9.278	3495.5	2408.0	424.240	1.938	1.73	plastic	Sand	1.1	454.54	0.97	438.99	438.99	0.93	0.983	0.961	0.961	0.961	0.961	0.961	0.00	0.00	
28.540	498.530	8.897	3512.5	2418.0	439.240	1.791	1.69	plastic	Sand	0.0	471.20	0.97	454.90	454.90	0.93	0.984	0.960	0.960	0.960	0.960	0.960	0.00	0.00	
28.710	503.720	8.924	3533.8	2428.6	442.847	1.778	1.68	plastic	Sand	0.0	476.11	0.96	459.11	459.11	0.93	0.984	0.959	0.959	0.959	0.959	0.959	0.00	0.00	
28.870	489.180	9.138	3553.8	2438.7	429.126	1.875	1.71	plastic	Sand	0.0	462.36	0.96	445.37	445.37	0.93	0.984	0.957	0.957	0.957	0.957	0.957	0.00	0.00	
29.040	491.910	9.173	3575.0	2449.3	450.842	1.920	1.72	plastic	Sand	0.6	464.94	0.96	447.34	447.34	0.93	0.984	0.956	0.956	0.956	0.956	0.956	0.00	0.00	
29.200	486.230	9.776	3595.0	2459.3	432.698	1.981	1.73	plastic	Sand	1.5	468.08	0.96	448.88	448.88	0.93	0.984	0.953	0.953	0.953	0.953	0.953	0.00	0.00	
29.360	480.930	6.934	3615.0	2469.3	419.207	1.447	1.62	plastic	Sand	0.0	454.57	0.96	436.42	436.42	0.93	0.985	0.954	0.954	0.954	0.954	0.954	0.00	0.00	
29.530	453.830	7.957	3636.3	2480.0	394.637	1.760	1.71	plastic	Sand	0.0	428.95	0.96	411.36	411.36	0.93	0.985	0.952	0.952	0.952	0.952	0.952	0.00	0.00	
29.690	439.690	7.690	3656.3	2490.0	381.514	1.756	1.71	plastic	Sand	0.0	415.59	0.96	398.12	398.12	0.93	0.985	0.951	0.951	0.951	0.951	0.951	0.00	0.00	
29.860	446.520	9.101	3677.5	2500.6	386.350	1.924	1.77	plastic	Sand	4.4	422.94	0.96	403.85	403.85	0.93	0.985	0.949	0.949	0.949	0.949	0.949	0.00	0.00	
30.020	465.110	9.336	3697.5	2510.7	401.980	2.015	1.75	plastic	Sand	3.2	439.61	0.96	420.22	420.22	0.93	0.985	0.949	0.949	0.949	0.949	0.949	0.00	0.00	
30.180	476.720	10.142	3717.5	2520.7	411.226	2.136	1.77	plastic	Sand	4.6	450.59	0.95	430.26	430.43	0.93	0.985	0.948	0.948	0.948	0.948	0.948	0.00	0.00	
30.350	500.100	10.221	3738.8	2531.3	430.824	2.050	1.74	plastic	Sand	2.6	472.98	0.95	451.14	451.14	0.93	0.986	0.946	0.946	0.946	0.946	0.946	0.00	0.00	
30.510	515.220	9																						

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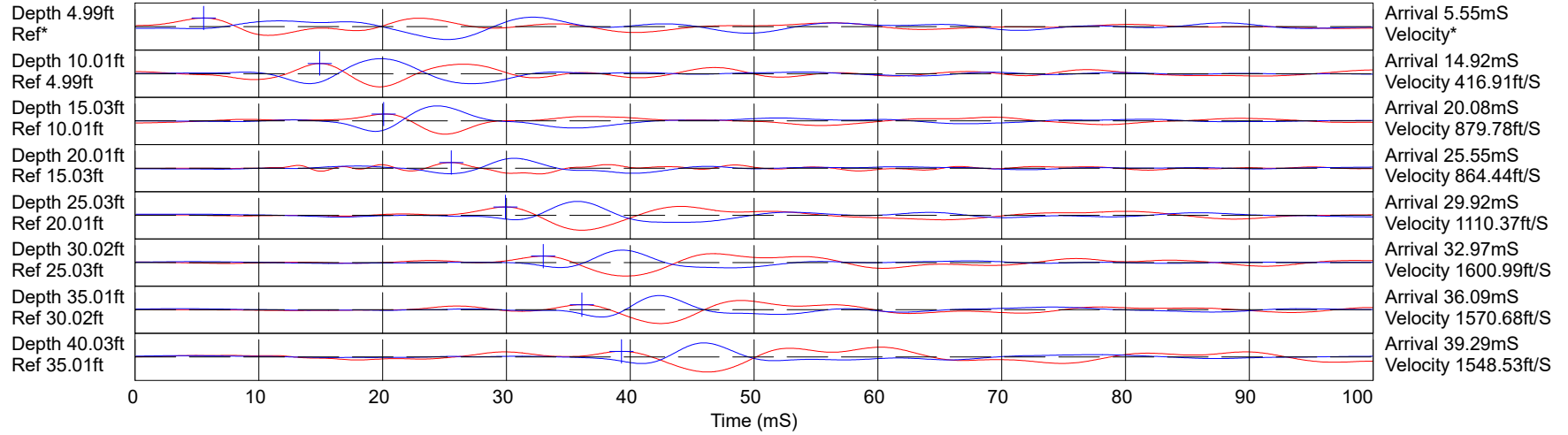
Depth (ft)	Q _c (tsf)	f _s (tsf)	σ _{vc} (psf)	Insitu σ _v (psf)	Q	F (%)	I _c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	Q _{ch} near interfaces (soft layer)	Thin Layer Factor (K _{LL})	Interpreted Q _{ch}	C _N	Q _{c1N}	Q _{c1N-C5}	Stress Reduction Coeff. f _d	CSR	K _σ for Sand	CRR _{0.0075} σ _{vc} = 1 atm	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε _v	Settlement (inches)	
43.800	369.790	11.260	5420.0	3373.3	274.794	3.067	1.99		Sand	22.5	349.52	0.88	309.05	384.47	0.88	0.974	0.860	0.974	0.860	0.974	0.860	0.974	0.860	0.00	0.00
43.960	396.240	11.661	5440.0	3383.3	294.150	2.963	1.97		Sand	20.3	374.52	0.88	330.90	400.36	0.88	0.974	0.859	0.974	0.859	0.974	0.859	0.974	0.859	0.00	0.00
44.130	417.200	12.275	5461.3	3393.9	309.323	2.962	1.95		Sand	19.4	394.33	0.88	348.12	415.68	0.88	0.973	0.858	0.973	0.858	0.973	0.858	0.973	0.858	0.00	0.00
44.290	444.200	12.435	5481.3	3404.0	328.981	2.817	1.92		Sand	16.8	419.85	0.88	370.36	429.89	0.87	0.973	0.857	0.973	0.857	0.973	0.857	0.973	0.857	0.00	0.00
44.460	473.790	13.123	5502.5	3414.6	350.477	2.786	1.90		Sand	15.4	447.82	0.88	394.70	445.03	0.87	0.973	0.856	0.973	0.856	0.973	0.856	0.973	0.856	0.00	0.00
44.620	490.990	13.554	5522.5	3424.6	362.735	2.776	1.90		Sand	14.7	464.07	0.88	408.72	455.80	0.87	0.972	0.856	0.972	0.856	0.972	0.856	0.972	0.856	0.00	0.00
44.780	488.160	11.928	5542.5	3434.6	360.099	2.457	1.85		Sand	11.1	461.40	0.88	406.05	429.19	0.87	0.972	0.855	0.972	0.855	0.972	0.855	0.972	0.855	0.00	0.00
44.950	461.130	11.615	5563.8	3445.3	339.513	2.534	1.88		Sand	13.1	435.85	0.88	383.25	417.79	0.87	0.972	0.854	0.972	0.854	0.972	0.854	0.972	0.854	0.00	0.00
45.110	440.810	10.781	5583.8	3455.3	323.983	2.461	1.88		Sand	13.0	416.64	0.88	366.08	399.24	0.87	0.971	0.853	0.971	0.853	0.971	0.853	0.971	0.853	0.00	0.00
45.280	385.690	10.129	5605.0	3465.9	282.770	2.645	1.93		Sand	17.6	364.55	0.88	320.05	375.52	0.87	0.971	0.852	0.971	0.852	0.971	0.852	0.971	0.852	0.00	0.00
45.440	384.240	9.639	5625.0	3475.9	281.286	2.527	1.92		Sand	16.4	363.18	0.88	318.60	367.46	0.87	0.971	0.851	0.971	0.851	0.971	0.851	0.971	0.851	0.00	0.00
45.600	363.600	9.084	5645.0	3486.0	265.675	2.518	1.93		Sand	17.3	343.67	0.88	301.26	353.20	0.87	0.970	0.850	0.970	0.850	0.970	0.850	0.970	0.850	0.00	0.00
45.770	339.470	8.412	5666.3	3496.6	247.520	2.499	1.94		Sand	18.4	320.86	0.88	281.04	335.91	0.87	0.970	0.849	0.970	0.849	0.970	0.849	0.970	0.849	0.00	0.00
45.930	317.280	7.989	5686.3	3506.6	230.867	2.541	1.97		Sand	20.3	299.89	0.88	262.47	322.52	0.87	0.969	0.848	0.969	0.848	0.969	0.848	0.969	0.848	0.00	0.00
46.100	310.670	7.260	5707.5	3517.3	225.664	2.359	1.95		Sand	18.6	293.64	0.87	256.80	309.39	0.87	0.969	0.848	0.969	0.848	0.969	0.848	0.969	0.848	0.00	0.00
46.260	300.060	7.243	5727.5	3527.3	217.568	2.437	1.97		Sand	20.3	283.61	0.87	247.84	305.62	0.87	0.969	0.847	0.969	0.847	0.969	0.847	0.969	0.847	0.00	0.00
46.420	285.620	7.528	5747.5	3537.3	206.687	2.804	2.03		Sand	25.3	269.96	0.87	235.74	307.76	0.87	0.968	0.846	0.968	0.846	0.968	0.846	0.968	0.846	0.00	0.00
46.590	245.710	8.511	5768.8	3547.9	177.247	3.505	2.15		Sand	34.7	232.24	0.87	202.64	286.75	0.87	0.968	0.845	0.968	0.845	0.968	0.845	0.968	0.845	0.00	0.00
46.750	186.760	8.219	5788.8	3558.0	134.021	4.470	2.30		Sand	47.2	176.52	0.86	152.35	236.88	0.87	0.968	0.844	0.968	0.844	0.968	0.844	0.968	0.844	0.00	0.00
46.920	170.750	9.064	5810.0	3568.6	122.161	5.400	2.39		Sand	54.4	161.39	0.85	137.95	222.86	0.86	0.967	0.843	0.967	0.843	0.967	0.843	0.967	0.843	0.00	0.00
47.080	197.770	7.134	5830.0	3578.6	141.621	3.661	2.22		Sand	40.5	186.93	0.86	161.67	243.06	0.86	0.967	0.842	0.967	0.842	0.967	0.842	0.967	0.842	0.00	0.00
47.240	147.540	4.913	5850.0	3588.6	104.959	3.397	2.27		Sand	44.8	139.45	0.83	116.34	190.01	0.86	0.966	0.876	0.966	0.876	0.966	0.876	0.966	0.876	0.00	0.00
47.410	80.420	4.829	5871.3	3599.3	43.056	6.232	2.73		Clay	81.1	76.01	0.87	n.a.	n.a.	0.86	0.966	n.a.	0.966	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
47.570	74.060	4.626	5891.3	3609.3	39.406	6.505	2.77		Clay	84.2	70.00	0.87	n.a.	n.a.	0.86	0.965	n.a.	0.965	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
47.740	95.650	4.721	5912.5	3619.9	51.213	5.093	2.61		Clay	71.8	127.78	0.87	n.a.	n.a.	0.86	0.965	n.a.	0.965	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
47.900	135.190	3.987	5932.5	3629.9	95.418	3.016	2.26		Sand	43.8	127.78	0.82	104.97	175.08	0.86	0.965	0.891	0.965	0.891	0.965	0.891	0.965	0.891	0.01	0.01
48.060	103.040	3.950	5952.5	3640.0	72.111	3.948	2.43		Sand	57.3	127.78	0.83	105.50	182.77	0.86	0.964	0.882	0.964	0.882	0.964	0.882	0.964	0.882	0.00	0.00
48.230	77.440	3.798	5973.8	3650.6	40.790	5.101	2.68		Clay	77.3	73.19	0.87	n.a.	n.a.	0.86	0.964	n.a.	0.964	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
48.390	100.250	3.223	5993.8	3660.6	69.887	3.314	2.38		Sand	53.5	170.56	0.85	145.56	232.09	0.86	0.963	0.836	0.963	0.836	0.963	0.836	0.963	0.836	0.00	0.00
48.560	80.390	3.168	6015.0	3671.3	42.156	4.094	2.60		Clay	71.0	170.55	0.86	n.a.	n.a.	0.86	0.963	n.a.	0.963	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
48.720	62.980	2.563	6035.0	3681.3	32.577	4.274	2.69		Clay	78.5	59.53	0.86	n.a.	n.a.	0.86	0.963	n.a.	0.963	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
48.880	60.810	2.509	6055.0	3691.3	31.308	4.343	2.71		Clay	79.9	57.48	0.86	n.a.	n.a.	0.86	0.962	n.a.	0.962	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
49.050	59.010	1.964	6076.3	3701.9	30.239	3.509	2.66		Clay	75.8	55.78	0.86	n.a.	n.a.	0.86	0.962	n.a.	0.962	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
49.210	57.580	1.878	6096.3	3711.9	29.382	3.443	2.66		Clay	76.1	54.42	0.86	n.a.	n.a.	0.86	0.961	n.a.	0.961	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
49.380	60.310	2.389	6117.5	3722.6	30.758	4.172	2.70		Clay	79.4	57.00	0.86	n.a.	n.a.	0.86	0.961	n.a.	0.961	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
49.540	59.410	2.860	6137.5	3732.6	30.189	4.722	2.75		Clay	82.8	56.15	0.86	n.a.	n.a.	0.85	0.960	n.a.	0.960	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
49.700	67.620	3.253	6157.5	3742.6	34.490	5.040	2.73		Clay	81.1	63.91	0.86	n.a.	n.a.	0.85	0.960	n.a.	0.960	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
49.870	71.620	3.534	6178.8	3753.3	36.518	5.157	2.72		Clay	80.2	67.69	0.86	n.a.	n.a.	0.85	0.960	n.a.	0.960	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
50.030	84.080	3.157	6198.8	3763.3	57.394	3.898	2.49		Sand	62.3	79.47	0.78	61.65	128.32	0.85	0.959	0.923	0.959	0.923	0.959	0.923	0.959	0.923	0.02	0.02
50.200	79.760	2.797	6220.0	3773.9	54.249	3.649	2.49		Sand	62.0	75.39	0.77	59.12	123.71	0.85	0.959	0.926	0.959	0.926	0.959	0.926	0.959	0.926	0.03	0.03
50.360	72.230	2.652	6240.0	3783.9	36.528	3.838	2.63		Clay	73.0	68.27	0.86	n.a.	n.a.	0.85	0.958	n.a.	0.958	n.a.	n.a.	n.a.	n.a.	n.a.	0.00	0.00
50.520	64.220	1.763	6260.0	3794.0	43.122	2.886	2.49		Sand	62.1	60.70	0.76	45.84	107.97	0.85	0.958	0.934	0.958	0.934	0.958	0.934	0.958	0.934	0.19	0.03

APPENDIX E: SHEAR WAVE ANALYSIS CALCULATIONS

CPT-01

Cornerstone Earth Group

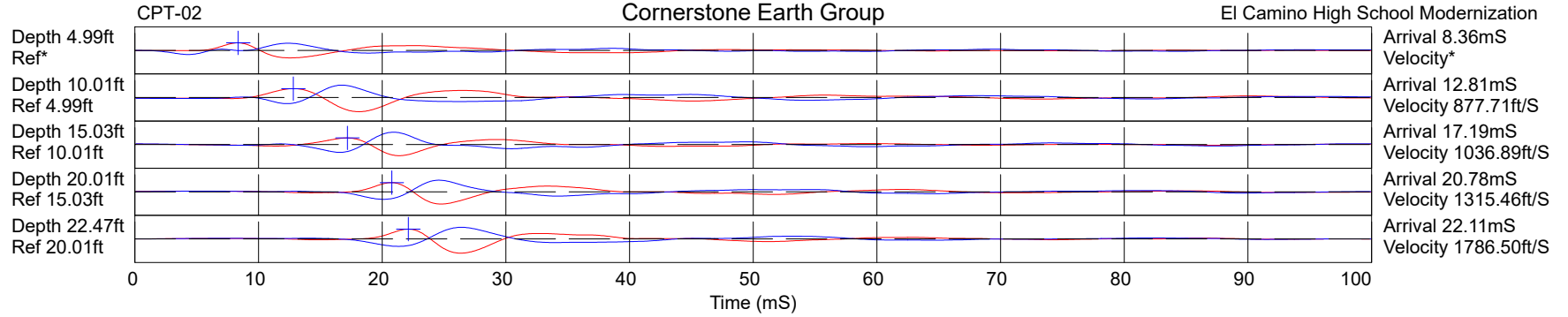
El Camino High School Modernization



Hammer to Rod String Distance (ft): 5.83

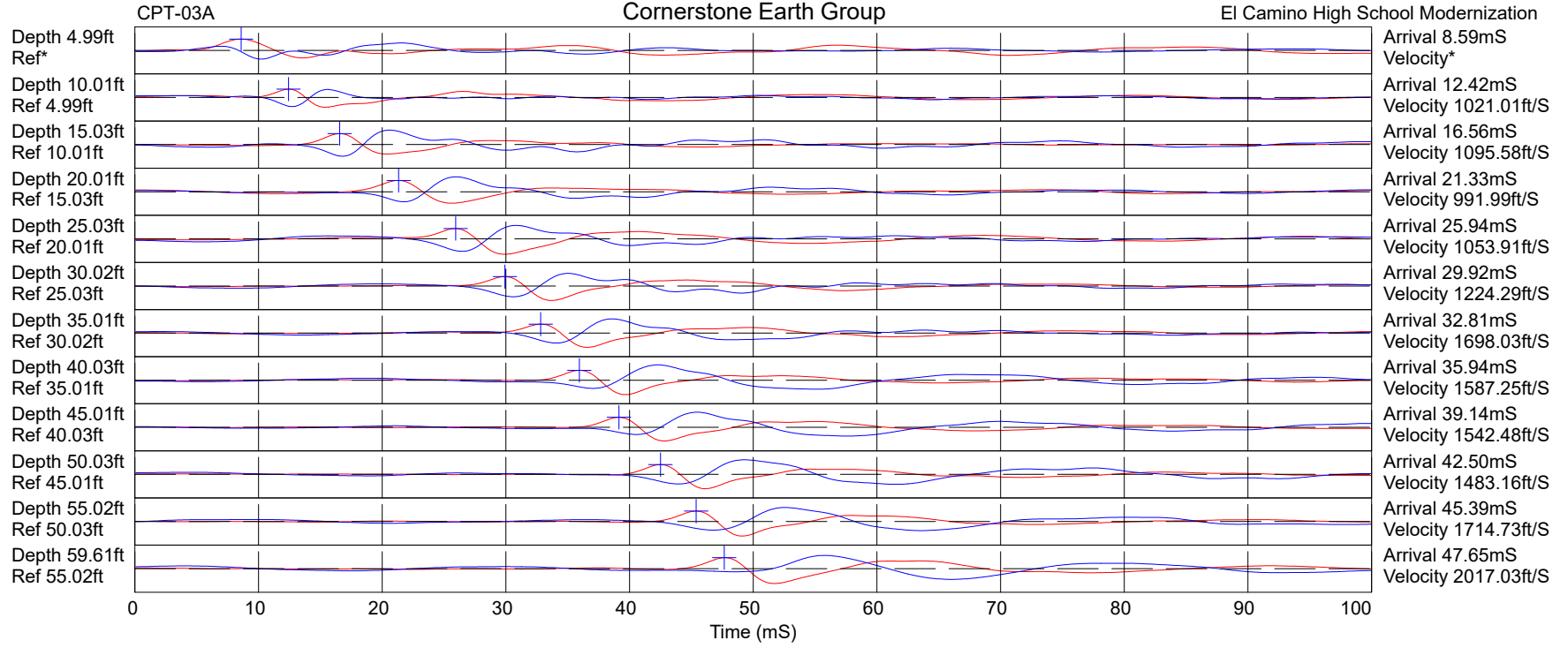
* = Not Determined

COMMENT:



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Average Shear Wave Velocity, v_s			
Depth (ft)	d_i (ft)	v_s (ft/s)	d_i/v_s (sec)
4.99	4.99	450	0.011089
10.01	5.02	1021.01	0.004917
15.03	5.02	1095.58	0.004582
20.01	4.98	991.99	0.00502
25.03	5.02	1053.91	0.004763
30.02	4.99	1224.29	0.004076
35.01	4.99	1698.03	0.002939
40.03	5.02	1587.25	0.003163
45.01	4.98	1542.48	0.003229
50.03	5.02	1483.16	0.003385
55.02	4.99	1714.73	0.00291
59.61	4.59	2017.03	0.002276
100	40.39	2017.03	0.020024
Totals	100		0.072372

ASSUMED

Job Name: El Camino HS Mod

Job Number: 1517-1-5

CPT Number: 3A

$v_s = 1382$ ft/s

$v_s = 421$ m/s

Site Class

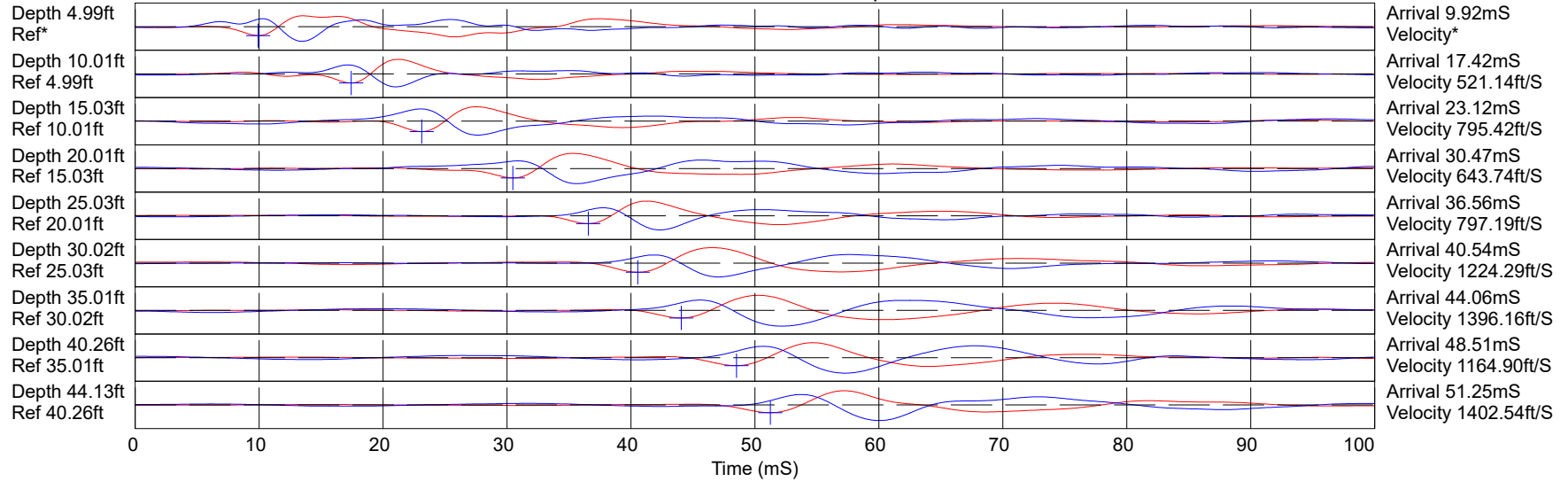
CD

Dense sand or very stiff clay

CPT-04

Cornerstone Earth Group

El Camino High School Modernization



Hammer to Rod String Distance (ft): 5.83

* = Not Determined

COMMENT:

Average Shear Wave Velocity, v_s			
Depth (ft)	d_i (ft)	v_s (ft/s)	d_i/v_s (sec)
4.99	4.99	450	0.011089
10.01	5.02	521.14	0.009633
15.03	5.02	795.42	0.006311
20.01	4.98	643.74	0.007736
25.03	5.02	797.19	0.006297
30.02	4.99	1224.29	0.004076
35.01	4.99	1396.16	0.003574
40.26	5.25	1164.9	0.004507
44.13	3.87	1402.54	0.002759
100	55.87	1402.54	0.039835
Totals	100		0.095817

Job Name: El Camino HS Mod

Job Number: 1517-1-5

CPT Number: 4

$$v_s = 1044 \text{ ft/s}$$
$$v_s = 318 \text{ m/s}$$

Site Class

CD	Dense sand or very stiff clay
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