CORNERSTONE EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation and Geologic Hazard Evaluation
PROJECT NAME	Piedmont Middle School New Gymnasium
LOCATION	955 Piedmont Road San Jose, California
CLIENT	Berryessa Union School District
PROJECT NUMBER	1332-2-2
DATE	September 12, 2022

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Client Address	**1376 Piedmont Road** **San Jose, California**
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Type of Services	**Geotechnical Investigation and Geologic Hazards Evaluation**
Project Name	**Piedmont Middle School New Gymnasium**
Location	**955 Piedmont Road San Jose, California**

SECTION 1: INTRODUCTION

This geotechnical investigation and geologic hazards evaluation report was prepared for the sole use of Berryessa Union School District for the Piedmont Middle School New Gymnasium project located at 955 Piedmont Road in San Jose, 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 site plan titled “Piedmont Middle School, 955 Piedmont Road, San Jose, CA 95132, New Modular Gymnasium”, prepared by McKim Design Group, dated May 11, 2022.
- A geotechnical report titled “Geotechnical Engineering Investigation, New Gymnasium, Piedmont Middle School, 955 Piedmont Road, San Jose, CA 95132”, prepared by CTE Cal, Inc., dated August 6, 2021.
- Topographic plans of the existing grades in the vicinity of the planned improvements titled “Topographic Survey Piedmont MS, 955 Piedmont Rd, San Jose, California” Sheets 1 and 2, prepared by Carroll Engineering Engineers and Surveyors, dated April 28, 2021.
- A geotechnical report titled “Geotechnical Site Characterization, Solar Array Projects – Ten School Sites, Berryessa Union School District, San Jose, California”, prepared by BSK Associates Engineers and Laboratories, dated April 18, 2016.
- A geotechnical report titled “Geologic Hazards Evaluation and Geotechnical Engineering Study, Berryessa Union School District, New Central Kitchen Facility, 945 Piedmont Road, San Jose, California”, prepared by Earth Systems Pacific, dated June 24, 2015.

CTE Cal, Inc. (CTE) previously prepared a geotechnical report (referenced above) for the new gymnasium project. Based on their report, CTE concluded there is a potential for lateral spreading to occur at the site and impact the proposed improvements. At the request of the

District, Cornerstone was retained to review this prior subsurface data, perform supplemental engineering analysis, to prepare an updated geotechnical investigation and geologic hazards evaluation report for the project, including re-evaluating the potential for lateral spreading. The conclusions presented in this report are based on previous field and laboratory programs performed by CTE and others.

1.1 PROJECT DESCRIPTION

The project site is within the Piedmont Middle School campus located at 955 Piedmont Road in San Jose, California. The site is currently occupied by 12 at-grade, 1- to 2-story administrative and/or classroom buildings (Buildings A through L), a kitchen building, five relocatable (modular) buildings (Buildings M, N, P, Q, and S), a corporate yard and associated buildings, various shade structures, paved parking with solar shade canopy arrays, pick up and drop off areas, and courtyard areas, paved sports courts, natural turf sports and playing fields with backstops, an unpaved running track, concrete sidewalks, and landscape areas. We understand that a new modular gymnasium building and fire access lane are currently planned for the northwest portion of the site.

We understand the project will include the demolition of the existing Building L. Based on the plans prepared by McKim Design Group (MDG) dated May 11, 2022, we understand the new gymnasium will have a footprint of approximately 7,500 square feet and will partially overlap the footprint of Building L to be demolished. Appurtenant fire access, utilities, landscaping and other improvements necessary for site development are also planned.

The site is bounded by Piedmont Road to the east, Flanders Drive to the north, and Penitencia Creek Park to the south and west.

Structural loads are not currently known for the proposed structure; however, structural loads are anticipated to be typical of similar type structures. In addition, grading is anticipated to include cuts/fills of approximately 1 to 3 feet for construction of the new building pad and utility installation.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated August 10, 2022 and consisted of reviews of the above referenced documents, including previously performed field and laboratory programs used to evaluate physical and engineering properties of the subsurface soil, engineering analysis to further evaluate the potential for lateral spreading and to prepare recommendations for site work and grading, building foundations, flatwork, and pavements, and preparation of this report. Brief descriptions of the previously performed exploration and laboratory programs are presented below.

1.3 PREVIOUS EXPLORATION PROGRAMS

As discussed, CTE previously prepared a geotechnical report (CTE, 2021) for the new gymnasium project. BSK Associates (BSK) and Earth Systems Pacific (ESP) previously perform geotechnical investigations at the site in 2016 and 2015, respectively.

CTE's field exploration consisted of three exploratory Borings (B-1 through B-3) drilled on June 24, 2021 with truck-mounted, hollow-stem auger drilling equipment and four Cone Penetration Tests (CPTs) advanced on July 28, 2021. The borings were drilled to depths of approximately 10 to 45 feet; the CPTs were advanced to depths of approximately 7¾ to 51⅓ feet below the existing grades. Refusal to CPT advancement was encountered in CPT-1, 2 and 3 at depths of approximately 8, 17 and 13 feet, respectively.

The approximate locations of the exploratory borings and CPTs performed by CTE are shown on the Site Plan and Geologic Map, Figure 2. Details regarding their field program are included in Appendix A.

1.4 PREVIOUS LABORATORY TESTING PROGRAMS

In addition to visual classification of samples, prior laboratory testing by CTE including moisture contents, dry densities, sieve analyses, Plasticity Index tests, unconfined-unconsolidated triaxial shear strength tests, and soil corrosivity were performed. Results from the previous laboratory programs are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Cornerstone Earth Group was not retained to provide environmental services for this project.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

2.1.1 Regional Geologic Setting

The relatively flat-lying plain along the eastern edge of the San Francisco Bay is bounded by the Santa Cruz Mountains on the west and the San Francisco Bay to the east. 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 trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates, 1) the North

American plate to the east and 2) the Pacific plate to the west. The San Andreas Fault system and its major branches is about 40 miles wide in the Bay area and extends from the San Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the Regional Fault Map, Figure 3. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system has more recently been identified by geologists working in the bay region.

2.1.2 Local Geology

The site is in an area adjacent to the San Francisco Bay where Holocene age (11,000 years or less before present) alluvial fan deposits account for the majority of Quaternary sediment which has been shed from the northwest-trending East Bay Hills located in the eastern portion of the San Jose and nearby Calaveras Reservoir quadrangles (CGS, 2001). The gentle southwest-sloping alluvial plain within the Calaveras Reservoir Quadrangle is covered by Holocene and Pleistocene alluvial fan and associated deposits, most of which been deposited by the various creeks that drain the east foothills (California Geological Survey, 2001).

Published geologic maps covering the general area of the site include those of Graymer et al. (1994), Helley and Graymer (1997), Wentworth et al, (1998), Knudsen et al. (2000), the California Geological Survey ("CGS", 2001), and the Dibblee Geological Foundation (2005). The USGS geologic map is reproduced as the Vicinity Geologic Map, Figure 3. The USGS mapping (1999) shows the site in an area underlain by Holocene alluvial fan deposits ("Qhf"). Wentworth et al., (1998) describe the Qhf unit as; "brown gravelly sand and sandy and clayey gravel, grading upward to sandy and silty clay, moderately dense to dense, coarser near the fan heads and upstream, deposited by flooding streams where they emerge from constrained channels of the uplands." Knudsen et al. (2000) characterize the Qf unit as follows: "Sediment deposited by streams emanating from mountain canyons onto alluvial valley floors or alluvial plains as debris flows, hyper concentrated mudflows, or braided stream flows.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area is 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. A Regional Fault Map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones. Figure 5 also shows regional faults with historical earthquake information superimposed. The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989, and passes approximately 16.3 miles west of the school site. Other major active faults in the Bay area include the Hayward, Calaveras, and the San Gregorio Fault Zone. The range of distances to significant faults is a result of variations in how a source measures distance to a fault. Tables 1A and 1B list all known active faults in order of increasing distance within 100 kilometers (62 miles) of the site. The range of distances to significant faults is a result of variations in how a source measures distance to a fault. Tables 1A and 1B present known active faults in order of increasing distance within 100 kilometers (62 miles) of the site. The fault distances presented in Table 1A are based on the USGS fault and fold database (2006) for the faults our Certified Engineering Geologist considered significant. The fault distances presented in Table 1B are based on the 2014 USGS fault model from the online Unified Hazard Tool (UHT). The seismic characteristics of some faults vary along its length so different segments of the same fault could be listed separately in the table. We have presented two different fault sources to provide a comprehensive look at the faults considered significant to the project site. At this time, the USGS 2014 fault model is only available in the UHT and we are not aware of an option for a full fault distance search. Therefore, our Certified Engineering Geologist has compiled a list of faults considered geologically significant within 100 km (Table 1A). We have also included the faults considered significant in the UHT (Table 1B).

Table 1A: Approximate Fault Distances (USGS 2006 Fault and Fold Database)

Fault Name	Distance (miles)	Distance (kilometers)
Hayward	0.8	1.3
Calaveras	3.6	5.8
Monte Vista-Shannon	11.3	18.2
Northern San Andreas	16.3	26.2
Greenville Connected	18.2	29.3
Las Positas	15.7	25.3
Sargent-Berrocal	18.4	29.6
Great Valley	26.6	42.8
San Gregorio Connected	30.1	48.4
Concord	35.8	57.6
Ortigalita	33.0	53.1
Monterey Bay – Tularcitos	36.0	57.9
Green Valley Connected	49.9	80.3
West Napa	56.6	91.1

Table 1B: Approximate Fault Distances (USGS Unified Hazard Tool, 2014)

Fault Name	Distance (miles)	Distance (kilometers)
Hayward (South)	2.1	3.3
Calaveras	3.6	5.8

SECTION 3: SITE CONDITIONS

3.1 GEOMORPHOLOGY AND RECENT HISTORY

Based on historic aerial imagery (1939 to the present) and topographic maps (1953 to the present) obtained of the area (listed in the Reference section), the site vicinity was occupied by agricultural fields consisting of orchards and row crops before the early 1960s. A farmhouse and large plowed fields were observed on the school site in the 1939, 1940 and 1956 aerial photographs. Agriculture was prominent well into the 1970s before the surrounding land became urbanized. Beginning sometime prior to 1963 construction of the school campus and consisting of four structures is visible in aerial photographs. At that time, development was only sporadic with tract homes present northeast of Piedmont Road and south of Penitencia Creek. During the 1960s, urbanization accelerated such that by 1980, Interstate Highway 680 and areas east of it were filled in with residential and commercial development. All of the existing classroom buildings and modifications to athletic facilities had all been made. Sometime in the 1990s the Berryessa Community Center, lake and Penitencia Creek Park had been developed.

At the time of our geologic reconnaissance, performed by our California-Certified Engineering Geologist on August 17, 2022, some pavements and flatworks within the campus have shown cracking and heaving. Adjacent to Building L and the future gymnasium some grading work had recently been completed. Settlement or displaced utilities, curbs and structures were not observed during the reconnaissance.

3.2 SURFACE DESCRIPTION

As discussed, the project site is within the Piedmont Middle School campus located at 955 Piedmont Road in San Jose, California. The site is currently occupied by 12 at-grade, 1- to 2-story administrative and/or classroom buildings (Buildings A through L), a kitchen building, five relocatable (modular) buildings (Buildings M, N, P, Q, and S), a corporate yard and associated buildings, various shade structures, paved parking with solar shade canopy arrays, pick up and drop off areas, and courtyard areas, paved sports courts, natural turf sports and playing fields with backstops, an unpaved running track, concrete sidewalks, and landscape areas.

The overall topography of the campus is relatively flat with a gentle downward slope to the west. Based on two topographic maps prepared by Carroll Engineering, the overall ground surface of the campus ranges from Elevation 200 to 203 feet based on the North American Vertical Datum 1988 (NAVD 88) with gradual slopes upward toward the eastern portion of the campus. There is an existing stormwater retention basin (lake) approximately 600 feet west of the proposed new gymnasium. Water was no observed in the lake at the time of our site visit and the bottom was observed to be about 6 to 8 feet below the adjacent surrounding grades with the sides sloped at about 3:1 (horizontal;vertical).

Surface pavements were not indicated in the explorations performed by CTE. However, based on our observations, the existing nearby pavements are in fair condition with localized cracking and trench patches.

3.3 GEOLOGY AND SUBSURFACE CONDITIONS

Below the vegetation, the CTE explorations (borings and CPTs) generally encountered Holocene alluvial fan deposits (Qhf). However, based on observations during our geologic site reconnaissance, the elevation of the building pad for Building L was observed to be approximately 1½ to 2 feet above the grade of the adjacent playing field in the vicinity of the backstop. Therefore, undocumented fill potentially underlies Building L. Figure 2, Site Plan and Geologic Map also includes a site geologic map although the thickness and presence of surficial fill is likely variable across the campus. It is noted that undocumented fill may be present in other areas of the project site. The alluvium (Qhf) encountered in CTE's Boring B-1 consisted of medium dense, clayey sand with gravel to a depth of 4½ feet, underlain by very dense, silty sand with gravel and cobbles to a depth of 8 feet, underlain by very dense, gravel with silt and sand to the terminal depth of 15 feet below the existing grade. Boring B-2 encountered loose, silty sand to a depth of 4 feet, underlain by hard, sandy clay to a depth of 7 feet, underlain by very dense, clayey gravel to a depth of 19 feet, underlain by medium dense, clayey sand with gravel to a depth of 33 feet, underlain by very stiff, sandy clay with gravel to a depth of 39 feet, underlain by dense to medium dense clayey sand with gravel to the terminal depth of 46½ feet below the existing grade. Boring B-3 encountered hard, sandy clay to a depth of 4 feet, underlain by very dense silty sand to a depth of 8 feet, underlain by very dense, clayey sand to the terminal depth of 11½ feet below the existing grades.

The California Geological Survey has published a compilation of 2,785 geotechnical laboratory tests conducted on the Qhf mapping unit and compiled by the CGS indicates this unit typically consists of primarily lean clay and clayey sand, with lesser amounts (10%) silty sand and silt, and 16% other constituents (CGS, 2001). The subsurface conditions encountered at the prior exploratory borings and the CPT test holes are generally consistent with that characterization.

CPT-1 through CPT-4 generally indicated soil behavior types (SBTs) consisting of clay, very stiff fine-grained sands, silty sand to sandy silts, sand to silty sand, sand, sandy silt to clayey silt, clayey silt to silty clay, gravelly sand to sand, and sand to clayey sand to a depth of 51⅓ feet below the existing grades. It should be noted that practical refusal was encountered at depths from 7¾ to 16¾ feet below the existing grades at CPT-1 through CPT-3 due to dense sands.

Our geologic Cross Sections A-A' and B-B' (Figures 6 and 7, respectively) were prepared utilizing the site geologic map as well as data from CTE's Borings B-1 through B-3, and CPT-1 through CPT-4. Of note, the boring and CPT locations indicated on the CTE site plan are shown on our Figure 2 and are not within the planned footprint of the new Gymnasium. As such the subsurface profiles depicted in our geologic cross-sections do not necessarily represent those beneath the gymnasium site, including the potential presence of undocumented fill.

3.3.1 Plasticity/Expansion Potential

CTE performed four Plasticity Index (PI) tests on representative samples. Their test results were used to evaluate expansion potential of surficial soil, and the plasticity of the fines in potentially liquefiable layers. The surficial PI tests resulted in PIs ranging from 10 to 20, indicating low to moderate expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Based on the descriptions of the soil encountered within the borings, the in-situ moisture contents within the upper 10 feet are likely to be below the estimated laboratory optimum moistures.

3.4 GROUNDWATER

Groundwater was reportedly encountered in CTE's Boring B-2 at a depth of 24 feet below the existing grade. However, groundwater was not encountered in BSK's (2016) and ESP's (2015) borings that extended to depths of 20 to 30 feet below the existing grades in the southeastern portion of the campus. Pore pressure dissipation (PPD) tests were not performed at the CPT locations for CTE's investigation. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. We understand the measurements were taken at the time of drilling and, therefore, may not represent the stabilized levels that can be higher than the initial levels encountered.

Based on information from the Department of Water Resources Sustainable Groundwater Management Act (SGMA) Data View, data from wells approximately 1 mile south/southwest of the site indicated a historically high groundwater depth in 2018 of 41 feet below the ground surface. Additionally, groundwater level data provided on the GeoTracker website (2022) from monitoring wells located approximately ½ mile north of the site indicated minimum groundwater levels greater than 50 feet below existing grades. In addition, the mapped historic high groundwater depth is greater than 50 feet (CGS, 2001); therefore, a design high groundwater depth of 50 feet was used for our analysis.

3.5 CORROSION SCREENING

CTE tested one sample collected at a depth of 2 feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2A.

Table 2A: 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)
Unknown	2	6.8	2,305	80	65

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

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

3.5.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soils may be considered moderately corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 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 318-19, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-19 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]	W0[3]	C0[4]

1 (F0) "Concrete not exposed to freezing-and-thawing cycles" (ACI 318-19)
2 (S0) "Water soluble sulfate in soil, percent by mass" (ACI 318-19)
3 (W0) "Concrete dry in service" (ACI 318-19)
4 (C0) "Concrete dry or protected from moisture" (ACI 318-19)

In addition, ACI 318-14, Table 19.3.2.1 provides requirements for concrete by exposure class. Table 2C below indicates different requirements that we recommend be followed for the concrete design.

Table 2C: ACI 318-14 Table 19.3.2.1 Requirements for Concrete by Exposure Class

Exposure Class	Maximum water:cement ratio	Minimum Compressive Strength (psi)	Cementitious materials – Types (ASTM C150)	Maximum Water-Soluble Chloride Ion Content (% wt)
F0	N/A	2,500	N/A	N/A
S0 (soil)	N/A	2,500	No type restriction	N/A
W0	N/A	2,500	N/A	N/A
C0	N/A	2,500	N/A	1.00 (0.06)[1]

1 Maximum water-soluble chloride ion content for non-pre-stressed concrete, (value for pre-stressed concrete).

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

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site most notably the Hayward Fault Zone located about ½ mile (approximately 2,800 feet) east of the site. However, no faults are mapped trending through or immediately adjacent to the site (Dibblee Foundation, 2005; ICBO, 1998; Wentworth et al., 1998; USGS Quaternary Fault and Fold Database, 2006). Accordingly, the site is not located within a state-designated Earthquake Fault Zone (CDMG, 1982). A review of aerial photos did not reveal any patterns of photographic tonal features indicative of active faulting nor did our surface reconnaissance reveal any patterns of geomorphic features indicative of faulting. Additionally, CTE subsurface explorations did not reveal any stratigraphic or groundwater patterns that would suggest disruption of the structure or water table by fault offset. It our judgement fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 HISTORICAL GROUND FAILURES

The Qhf geologic unit that underlies the ground surface at the site has experienced liquefaction related ground failure historically prior to the 1989 Loma Prieta Earthquake and has been generally given a moderate susceptibility for liquefaction where the groundwater table is located greater than 10 feet below the ground surface (Knudsen et al., 2000). In the Calaveras Reservoir Quadrangle (California Geological Survey, 2001), the CGS states that the potential for ground failure resulting from liquefaction-induced lateral spreading of alluvial materials, considered by some to be a form of landsliding, is not specifically addressed by the earthquake-induced landslide zone or their report. Based on our review of published mapping by Youd and Hoose (1978), historic ground failures have been documented in the eastern half of the San Francisco Bay Area; however, none are noted on or near the project site. For reasons discussed below, the subsurface soil conditions encountered in our explorations at the site suggest the soils would not be susceptible to sand boils or lateral spreading and we judge that the nearby historical occurrences of these types of ground failure are not a concern for this project.

4.3 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA_M) was estimated following the ground motion hazard analysis procedure presented in Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No.1. For our liquefaction analysis we used a PGA_M of 0.97g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.4 LIQUEFACTION POTENTIAL

As indicated on Figure 8, the site is not located within a State-designated Liquefaction Hazard Zone (CGS, Calaveras Reservoir 7.5-Minute Quadrangle, 2001) or a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). However, we screened the site for liquefaction following by analyzing the results of CPT-4, performed by others to a depth of 50 feet. Other CPTs reached refusal at much shallower depths, as previously discussed. We also reviewed boring logs from explorations performed by others in the area of CPT-4.

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 1997). 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.

As discussed in the “Subsurface” section above, previous exploration primarily encountered unsaturated, medium dense to very dense sands and gravel to a depth of 50 feet. In addition, historic high groundwater is mapped at depths greater than 50 feet (CGS, 2001). Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction, which is in agreement with mapping by the State of California and the County of Santa Clara.

4.5 LATERAL SPREADING

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

Previous analysis by CTE (2021) indicated that a potential for lateral spreading may exist, even though analytical review indicated no deformation. Their conclusions, somewhat based on empirical methods, in our opinion, are not supported by updated analytical methods, nor by screening efforts at the site. Two other firms have also commented on the potential for liquefaction at the site - Earth Systems Pacific (2015) and BSK Associates (2016), both finding low potential for liquefaction or lateral spreading. The site is generally underlain by unsaturated, medium dense to very dense sands and gravels with a low potential for liquefaction. In addition, it is generally over 600 feet to a shallow detention facility of limited width and height, with highly heterogenous subsurface conditions. Because of the geometry and distance to a free face and the fact that liquefaction will not occur, in our opinion, the potential for lateral spreading is very low.

4.6 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Strong seismic shaking can cause unsaturated cohesionless soil to consolidate. Soil most susceptible to seismic settlement are medium dense sand and gravel with relatively low fines contents. As discussed, CTE’s borings generally encountered loose to medium dense, silty sand and clayey sand to a depth of 4½ feet below the existing grades and above the design groundwater depth of 50 feet. Laboratory testing indicated the clayey sand had fines contents ranging from approximately 15 to 46 percent.

We evaluated the potential for seismic settlement/unsaturated sand shaking using the method proposed by Pradel (1998). This analysis method estimates seismically-induced settlement based on the Standard Penetration Test (SPT) blow count or N-value and the design earthquake characteristics. Due to the inherent variability of the SPT and sensitivity of the SPT to test procedures, it is necessary to correct SPT N-values to normalized values. The corrections to the prior SPT data obtained by CTE are presented in Appendix D. Details regarding our seismically-induced settlement analysis, as well as example calculations and results, are also presented in Appendix D.

Based on our analysis, settlement from seismic settlement/unsaturated sand shaking were estimated to be ⅓ to 1½ inches. Differential settlement between borings is estimated to be approximately ¾ inch between B-1 and B-2.

4.7 LANDSLIDING

The site is not located in or adjacent to any mapped landslides (Graymer et al, 1994; Bryant, 1980; Dibblee and Minch, 2005; California Geological Survey, 2016). Accordingly, the site is not located within a county or state regulatory zone for landsliding (Santa Clara County Planning Dept., 2003; California Geological Survey, 2004). Due to the flat-lying nature of the site and the absence of slopes within a few miles of the site, in our opinion, the potential for landsliding to affect the site is negligible.

4.8 SEISMICALLY INDUCED WAVES - TSUNAMIS/SEICHES

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events) and have affected the west coasts during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. More recently the 2011 Japanese earthquake generated a tsunami that damaged docks and boats at the Santa Cruz Yacht harbor, located 33 miles south so the subject site. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 11½ miles inland from the San Francisco Bay shoreline and is approximately 200 feet above mean sea level. Furthermore, the California Emergency Management Agency (CEMA) Tsunami-themed series of maps shows the nearest Tsunami inundation hazard zone as being located at the bay waterfront, approximately 8½ miles west of the campus near Alviso.

The site is located a significant distance from areas expected to be impacted in a tsunami event (ABAG, 2007; CEMA, 2020; Tsunami Modeling Working Group, 2013). Based on the above, it is our opinion that the potential for inundation due to tsunami or seiche is considered to be negligible.

4.9 FLOODING AND RESERVOIR INUNDATION

FEMA flood zone mapping for City of San Jose, California and Incorporated Areas, Panel 060349 Map No. 06085CO088J, (2014) were used to assess the potential for flooding of the site. Based on the noted map, the site is located in a zone designated as "Area of Minimal Flood Hazard Zone X", Figure 5 FEMA Flood Hazard Map. Therefore, the potential flooding hazard for the site is considered to be a minimal risk for this project.

The California Division of Safety of Dams has compiled a database and interactive map depicting Dam Breach Inundation Maps (DSOD, 2020). These maps are intended for planning purposes only. Based on our review of these maps, the site is located within a dam failure inundation area based on its location. Cherry Flat Reservoir, located in the Penitencia Creek watershed is approximately 4¾ miles to the east. In the event of dam failure, the project site would be impacted based on its proximity to Penitencia Creek located 1,250 feet to the south.

4.10 VOLCANIC ERUPTION

The site is located over 200 miles from the nearest potentially or historically active volcano (at Mt. Lassen National Park). Based on our review, the hazards associated with volcanic eruptions (primarily particulate airborne ash) for the school site is very low.

4.11 NATURALLY OCCURRING ASBESTOS

Chrysotile and amphibole asbestos occur naturally in certain geologic settings in the San Francisco Bay area most commonly in serpentinite and other ultramafic rocks. These are igneous and metamorphic rocks with a high content of magnesium and iron minerals. The most common type of asbestos is chrysotile, which is commonly found in serpentinite rock formations. When disturbed by construction, grading, quarrying, or surface mining operations, asbestos-containing dust can be generated. Exposure to asbestos can result in lung cancer, mesothelioma, and asbestosis. In July 2001, the California Air Resources Board approved an Asbestos Airborne Toxic Control measure for Construction, Grading, Quarrying, and Surface Mining activities in areas where naturally occurring asbestos (NOA) will likely be found and to provide best dust mitigation measures and practices. These are mountainous areas or areas of shallow bedrock that could be encountered during construction. The subject site is not underlain by ultramafic rock, nor is it located immediately adjacent to any known deposits of ultramafic rock. These published geologic maps referenced in the current school site evaluation indicate the nearest mapped outcrop of ultramafic rock occurs an elongate, northwest trending band of serpentinite located approximately 1.5 miles east of the site in the "Boulder" neighborhood of East San Jose. of the site (Churchill and Hill, 2000, Graymer et al., 1997; Wentworth et al., 1998; Dibblee and Minch, 2005). Our geologic reconnaissance and subsurface investigation of the subject site revealed only Holocene alluvial fan deposits in the vicinity of the site. These earth materials are unlikely to contain serpentinite or another ultramafic rock with NOA. Based on our experience with other MUSD projects and in the vicinity of the site, there is a potential for native soil deposits to contain NOA although typically in negligible amounts. However, if additional information is desired, specific testing can be performed.

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 dry sand settlement
- Presence of moderately expansive soil
- Potential for undocumented fill
- Soil corrosion potential

5.1.1 Potential for Dry Sand Settlement

We evaluated the potential for seismic settlement/unsaturated sand shaking using the method proposed by Pradel (1998). This analysis method estimates seismically-induced settlement based on the Standard Penetration Test (SPT) blow count or N-value and the design earthquake characteristics. Due to the inherent variability of the SPT and sensitivity of the SPT to test procedures, it is necessary to correct SPT N-values to normalized values. The corrections to the SPT data are presented in Appendix D. Details regarding our seismically-induced settlement analysis, as well as example calculations and results, are also presented in Appendix D.

Based on our analysis, settlement from seismic settlement/unsaturated sand shaking were estimated to be ⅓ to 1½ inches. Differential settlement is estimated to be approximately ¾ inch between adjacent foundation elements or over a horizontal distance of 30 feet.

5.1.2 Presence of Moderately Expansive Soil

Based on the laboratory testing performed by CTE, low to moderately expansive surficial soil was encountered at depths less than 5 feet. Expansive soil can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from the building as well as limiting landscaping watering. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.3 Potential for Undocumented Fill

Although CTE did not indicate that undocumented fill was encountered within their explorations, the elevation of the building pad for existing Building L was observed to be up to 1½ feet higher than the adjacent surrounding grades. In addition, CTE did not perform their explorations within the planned footprint of the new gymnasium; therefore, it is unknown at this time if undocumented fill is present within the planned footprint. Any fills encountered during site grading should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fill meets the “Material for Fill” requirements below, the fill may be reused when backfilling the excavations. 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.

5.1.4 Soil Corrosion Potential

Our testing indicates sulfate exposure at the site is low and therefore no cement-type restrictions to buried concrete. However, the corrosion potential for buried metallic structures, such as metal pipes, is considered moderately corrosive. Based on the results of the preliminary soil corrosion screening, special requirements for corrosion control will likely be required to protect metal pipes and fittings. We recommend a corrosion engineer be engaged to provide recommendations for corrosion protection of metal pipes, if used on this project.

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

We understand that Building L will be demolished for the construction of the new gymnasium. All existing improvements not to be reused for the planned development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

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

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements 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.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Following review, additional mitigation or planned foundation elements may need to be modified.

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. 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. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

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

All undocumented fills, if encountered, should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should 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 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below.

6.4 MITIGATION OF DRY SAND SETTLEMENT

As discussed above, our analysis of the loose to medium dense, silty and clayey sand encountered in the upper 4½ feet of the borings indicated that up to 1½ inches of seismic settlement may occur as the result of a design earthquake event. To reduce the potential for seismic/dry sand settlement to occur and impact the proposed building, we recommend the soil within the new gymnasium footprint be over-excavated 3 feet below existing grades, moisture conditioned, as needed, and replaced in lifts and compacted as engineered fill. Recommendations for compaction and moisture conditioning are provided in the “Compaction” section below.

6.5 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. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type C materials.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 3 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.6 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 12 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soil

On-site soil 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

Import non-expansive material and soil to be used as general fill 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.

6.7.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and variable PI of the on-site soil materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that on-site clayey materials will likely need to be mixed with at least 3 percent quicklime (CaO) or approved equivalent to adequately reduce the PI of the on-site soils to 15 or less. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

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 consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 4: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

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

6.8.1 Construction Moisture Conditioning

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

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 (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.
Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence, or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

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

6.10 SITE DRAINAGE

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

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site. Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are variable and consist of both clayey/silty sands and sandy clay, likely categorized as Hydrologic Soil Group C or D, and are expected to have infiltration rates on the order of 0.2 to 0.5 inches per hour. In our opinion, these clayey soils may significantly limit the infiltration of stormwater.

- Locally, seasonal high ground water is mapped at a depth of 50 feet or greater, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

6.11.1 Storm Water Treatment Design Considerations

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

<u>6.11.1.1 General Bioswale Design Guidelines</u>

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

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

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

6.11.1.2 Bioswale Infiltration Material

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

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12-inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

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

6.11.1.3 Bioswale Construction Adjacent to Pavements

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

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

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

6.12 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately expansive, 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: 2019 CBC SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

7.1 SITE LOCATION AND PROVIDED DATA FOR 2019 CBC SEISMIC DESIGN

The project is located at latitude 37.39319° and longitude -121.84613°, which is based on Google Earth (WGS84) coordinates at the approximate center of the proposed gymnasium within the Piedmont Middle School campus located at 955 Piedmont Road in San Jose, California. We have assumed that a Seismic Importance Factor (I_e) of 1.25 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category III. The building period has not been provided by the project structural engineer.

7.2 SITE CLASSIFICATION – CHAPTER 20 OF ASCE 7-16

Code-based site classification and ground motion attenuation relationships are based on the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile (V_{S30}).

As discussed in Section 3, our explorations generally encountered medium dense to very dense, clayey and silty sand with gravel and gravel with varying amounts of clay and silt and medium stiff to hard, clay deposits to a depth of 51⅓ feet below the existing grades, the maximum depth explored. Shear wave velocity (V_S) measurements were not performed while advancing the CPTs; however, we have estimated a shear wave velocity for the top 30 meters (V_{S30}) from blowcount data from the previous borings of approximately 295 meters per second. In accordance with Table 20.3-1 of ASCE 7-16, we recommend the site be classified as Soil Classification D, which is described as a "stiff soil" profile. Because we used site specific data from the previous explorations and laboratory testing, the site class should be considered as "determined" for the purposes of estimating the seismic design parameters from the code outlined below. Our site-specific ground motion hazard analysis considered a V_{S30} of 281 m/s (926 ft/s).

7.2.1 Code-Based Seismic Design Parameters

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum Considered Earthquake (MCE_R) spectral acceleration parameters (S_S and S_1) were determined using the ATC Hazards by Location website (https://hazards.atcouncil.org).

The mapped acceleration parameters were adjusted for local site conditions based on the average soil conditions for the upper 100 feet (30 meters) of the soil profile. Code-based MCE_R spectral response acceleration parameters adjusted for site effects (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table 5.

In accordance with Section 11.4.8 of ASCE 7-16, structures on Site Class D sites with mapped 1-second period spectral acceleration (S_1) values greater than or equal to 0.2 require a site-specific ground motion hazard analysis be performed in accordance with Section 21.2 of ASCE 7-16. ***Design seismic parameters determined by performing a Ground Motion Hazard Analysis per Section 21.2 of ASCE 7-16 are presented in Table 8. Recommended values in Table 5 should not be used for design unless in the judgement of the structural engineer an exception can be taken in accordance with Section 11.4.8 of ASCE 7-16.*** Values summarized in Table 5 are only used to determine Seismic Design Category and comparison with minimum code requirements for further use in our ground motion hazard analysis (GMHA).

Table 5: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.39319°
Site Longitude	-121.84613°
Risk Category	III
Short Period Mapped Spectral Acceleration – S_S	2.101g
1-second Period Mapped Spectral Acceleration – S_1	0.810g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	*null
Short Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	2.101g
Short Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.401g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	*null
Long-Period Transition – T_L	12 seconds
Site Coefficient – F_{PGA}	1.1
Site Modified Peak Ground Acceleration – PGA_M	0.97g

*null – per section 11.4.8 of ASCE 7-16

7.3 GROUND MOTION HAZARD ANALYSIS

Following Section 11.4.8 of ASCE 7-16, we performed a ground motion hazards analysis (GMHA) in accordance with Chapter 21, Section 21.2 of ASCE 7. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project. Our analyses were performed using the USGS interface Unified Hazard Tool (UHT) based on the UCERF 3 Data Set, Building Seismic Safety Council (BSSC) Scenario Catalog 2014 event set (BSSC 2014), and the 2014 National Seismic Hazard Maps – Source Parameters (NSHMP deterministic event set). Additionally, we utilized the USGS program Response Spectra Plotter with combined models (Combined: WUS 2014 (4.1)).

Our analysis utilized the mean ground motions predicted by four of the Next Generation Attenuation West 2 (NGA-West 2) relationships: Boore-Atkinson (2013), Campbell-Bozorgnia (2013), Chiou-Youngs (2013), and Abrahamson-Silva (2013). Rotation factors (scale factors) were determined as specified in ASCE 7-16 Chapter 21, Section 21.2, to calculate the maximum rotated component of ground motions (ASCE, 2016).

7.3.1 Probabilistic MCE_R

We also performed a probabilistic seismic hazard analysis (PSHA) per ASCE 7-16 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50-year period (2,475-year return period). The probabilistic MCE spectrum was multiplied by Risk Coefficients (CR) to determine the probabilistic MCER. We used Risk Coefficients (CR_S and CR_1) of 0.930 and 0.912, respectively, based on ASCE 7-16 Section 21.2.1.1 – Method 1 and the ATC website. Risk coefficients for the various periods are presented in Table 6, Column 3.

The resulting probabilistic MCE_R is presented on Figure 9 (red line). Spectral ordinates are tabulated in Table 6, Column 6.

7.3.2 Deterministic MCER

We performed deterministic seismic hazard analyses in accordance with ASCE 7-16 Section 21.2.2 and ASCE 7-16 Supplement No. 1. The deterministic MCE_R acceleration response spectrum is calculated as the largest 84th percentile ground motion in the direction of maximum horizontal response for each period for characteristic earthquakes on all known active faults within the region. The largest deterministic ground motion resulted from a M_w 7.58 earthquake on the fully combined Hayward Fault (RC+HN+HS+HE segments), located at a distance of approximately 2.08 km from the site.

In accordance with Supplement No.1 of ASCE 7-16, when the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is less than $1.5F_a$ then the largest 84th percentile rotated response spectrum (Table 6, Column 4) shall be scaled by a single factor such that the maximum response spectral acceleration equals $1.5F_a$. For Site Classes A, B, C and D, F_a is determined using Table 11.4.1 with the value of S_s taken as 1.5; for Site Class E, F_a shall be taken as 1.0. When the largest spectral response acceleration of the probabilistic ground motion response of 21.2.1 is less than $1.2F_a$, the deterministic ground motion response spectrum does not need to be calculated.

As the largest probabilistic spectral response acceleration was determined to be 2.515 which is greater than $1.2F_a$, where F_a is taken as 1.000 from Table 11.4-1 in ASCE 7-16 Supplement No.1, the 84th percentile rotated response spectrum was calculated as part of the deterministic analyses. The maximum spectral acceleration from the 84th percentile rotated response spectrum was then compared to $1.5F_a$ to determine if a scale factor needed to be applied. The deterministic MCE spectrum are tabulated in Table 6, Column 5. The deterministic MCE_R is presented graphically on Figure 9 (blue line).

7.3.3 Site-Specific MCER

The site-specific MCE_R is defined by ASCE 7-16 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE_R's at each period. Spectral ordinates for the site-specific MCE_R are tabulated in Table 6, Column 7 and shown graphically on Figure 9 (dashed black line).

Table 6: Development of Site-Specific MCE_R Spectrum

Period (seconds)	CBC General Spectrum (g)	Risk Coefficient	Det. 84th Percentile Rotated	Deterministic MCE_R (g)	Probabilistic MCE_R (g)	Site-Specific MCE_R (g)
0.000	0.560	0.930	1.067	1.067	1.194	1.067
0.050	0.778	0.930	1.167	1.167	1.586	1.167
0.100	0.996	0.930	1.677	1.677	1.978	1.677
0.150	1.214	0.930	2.037	2.037	2.268	2.037
0.193	1.401	0.930	2.231	2.231	2.518	2.231
0.200	1.401	0.930	2.263	2.263	2.559	2.263
0.250	1.401	0.929	2.424	2.424	2.800	2.424
0.300	1.401	0.928	2.498	2.498	3.042	2.498
0.400	1.401	0.926	2.515	2.515	3.155	2.515
0.500	1.401	0.923	2.414	2.414	3.268	2.414
0.750	1.401	0.918	1.942	1.942	2.907	1.942
0.964	1.400	0.913	1.628	1.628	2.672	1.628
1.000	1.350	0.912	1.575	1.575	2.633	1.575
2.000	0.675	0.912	0.695	0.695	1.495	0.695
3.000	0.450	0.912	0.434	0.434	0.994	0.434
4.000	0.338	0.912	0.287	0.287	0.695	0.287
5.000	0.270	0.912	0.215	0.215	0.524	0.215

7.3.4 Design Response Spectrum

The Design Response Spectrum (DRS) is defined in ASCE 7-16 Section 21.3 as:

- two-thirds of the site-specific MCE_R, but
- not less than 80% of the general design response spectrum

Spectral accelerations corresponding to two-thirds of the MCE_R are tabulated in Table 7, Column 2. Ordinates corresponding to 80% of the general Site Class D response spectrum are tabulated below in Table 7, Column 3. Ordinates of the site-specific DRS are tabulated in Table 7, Column 4. Development of the site-specific DRS is presented graphically on Figure 10 (dashed black line).

Table 7: Development of Site-Specific Design Response Spectrum

Period (seconds)	2/3 Site-Specific MCE_R (g)	80% CBC General Spectrum (g)	Design Response Spectrum (g)
0.000	0.712	0.448	0.712
0.050	0.778	0.623	0.778
0.100	1.118	0.797	1.118
0.150	1.358	0.971	1.358
0.193	1.488	1.121	1.488
0.200	1.509	1.121	1.509
0.250	1.616	1.121	1.616
0.300	1.666	1.121	1.666
0.400	1.676	1.121	1.676
0.500	1.609	1.121	1.609
0.750	1.295	1.121	1.295
0.964	1.085	1.120	1.120
1.000	1.050	1.080	1.080
2.000	0.463	0.540	0.540
3.000	0.289	0.360	0.360
4.000	0.192	0.270	0.270
5.000	0.143	0.216	0.216

7.3.5 Design Acceleration Parameters

Design acceleration parameters (S_{DS} and S_{D1}) were determined in accordance with Section 21.4 of ASCE 7-16. S_{DS} is defined as the design spectral acceleration at 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 seconds, inclusive. S_{D1} is defined as the maximum value of the product, TS_a, for periods from 1 to 2 seconds for sites with $v_{s,30}$ > 1,200 ft/s ($v_{s,30}$ > 365.8 m/s) and for periods from 1 to 5 seconds for sites with $v_{s,30} \leq$ 1,200 ft/s ($v_{s,30} \leq$365.8 m/s).

Site-specific MCE_R spectral response acceleration parameters (S_{MS} and S_{M1}) are calculated as:

- 1.5 times the S_{DS} and S_{D1} values, respectively, but
- not less than 80% of the code-based values presented in Table 5

Recommended design acceleration parameters are summarized in Table 8.

When using the Equivalent Lateral Force Procedure, ASCE 7-16 Section 21.4 allows using the spectral acceleration at any period (T) in lieu of S_{D1}/T in Eq. 12.8-3 and $S_{D1}T_L/T^2$ in Eq. 12.8-4.

The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table 7, Column 4.

Table 8: Site-Specific Design Acceleration Parameters

Parameter	Value
S_{DS}	1.509
S_{D1}	1.080
S_{MS}	2.263
S_{M1}	1.620

7.3.6 Site-Specific MCEG Peak Ground Acceleration

We calculated the Site-Specific MCE_G Peak Ground Acceleration (PGA_M) per ASCE 7-16 Section 21.5. The Site-Specific PGA_M is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The 2% in 50-year probabilistic geometric mean PGA is 1.214g. The deterministic PGA is considered the greater of the largest 84th percentile deterministic geometric mean PGA (1.136 g) or one-half of the tabulated F_{PGA} value from ASCE 7-16 Table 11.8.1 with the value of PGA taken as 0.5g. For Site Class D, F_{PGA} is 1.10 and one-half of the F_{PGA} is 0.55g; therefore, the deterministic PGA is 0.97g. Additionally, the Site-Specific PGA_M may not be less than 80% of the mapped PGA_M determined from ASCE 7-16 Equation 11.8-1. The mapped PGA_M for the site is 0.97g; 80% of PGA_M is 0.78g.

Based on the above, the recommended Site-Specific PGA_M for the site is 0.97g.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed gymnasium may be supported on shallow foundations provided the estimated settlement (seismic and static) are tolerable and the recommendations in the “Earthwork” section and the sections below are followed.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings

Conventional shallow footings should bear on natural, undisturbed soil or engineered fill, and extend at least 18 inches below the lowest adjacent grade. 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. The deeper footing embedment is due to the presence of highly expansive soils and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the 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 9: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	220 kips
Exterior Isolated Column Footing	110 kips
Perimeter Strip Footing	4 to 6 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate the total static footing settlement will be on the order of about ½ inch, with about ¼ inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be up to ¾ inch between adjacent foundation elements; however, over-excavation of the upper 3 feet of the loose to medium dense dry sand is recommended and will reduce the estimated differential seismic settlement to less than ¼ inch resulting in a total estimated differential footing movement of about ½ inch between adjacent foundation elements or over a horizontal distance 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 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity unless the surrounding exterior grade is covered with pavement or flatwork.

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.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

Due to the moderate expansion potential of the surficial soil, the proposed slabs-on-grade should be at least 5 inches thick and be supported on at least 9 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

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-ON-GRADE 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
¾"	90 – 100
No. 4	0 – 10
No. 200	0 – 5

  The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- 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 6 inches of non-expansive fill 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 5. The design R-value was chosen based on engineering judgement considering the variable and expansive soil conditions.

Table 10: Asphalt Concrete Pavement Recommendations, Untreated Soils

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	8.5	11.0
4.5	2.5	9.5	12.0
5.0	3.0	11.0	14.0
5.5	3.0	12.0	15.0
6.0	3.5	13.5	17.0
6.5	4.0	14.5	18.5

[1]Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 5

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

10.2 PORTLAND CEMENT CONCRETE

The 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.

For design, we assumed a worst-case Average Daily Truck Traffic (ADTT) of 100 was used for the planned concrete street pavement. The following table presents minimum PCC pavements thicknesses for various traffic loading categories and the anticipated maximum Average Daily Truck Traffic (ADTT).

Table 11: PCC Pavement Recommendations

Traffic Category	Minimum PCC Thickness[1] (inches)	Class 2 Aggregate Base (inches)
Maximum ADTT = 0	4.0	6.0
Maximum ADTT = 25	6.5	6.0
Maximum ADTT = 100	7.5	6.0

[1]Subgrade design R-Value = 5

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.

10.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 8 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

10.3 PAVEMENT CUTOFF

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

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

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 12: Recommended Lateral Earth Pressures

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

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

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

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

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

Retaining walls may be supported on a continuous and or spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Berryessa Union School District specifically to support the design of the Piedmont Middle School Gymnasium project in San Jose, 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.

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

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

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

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

SECTION 13: REFERENCES

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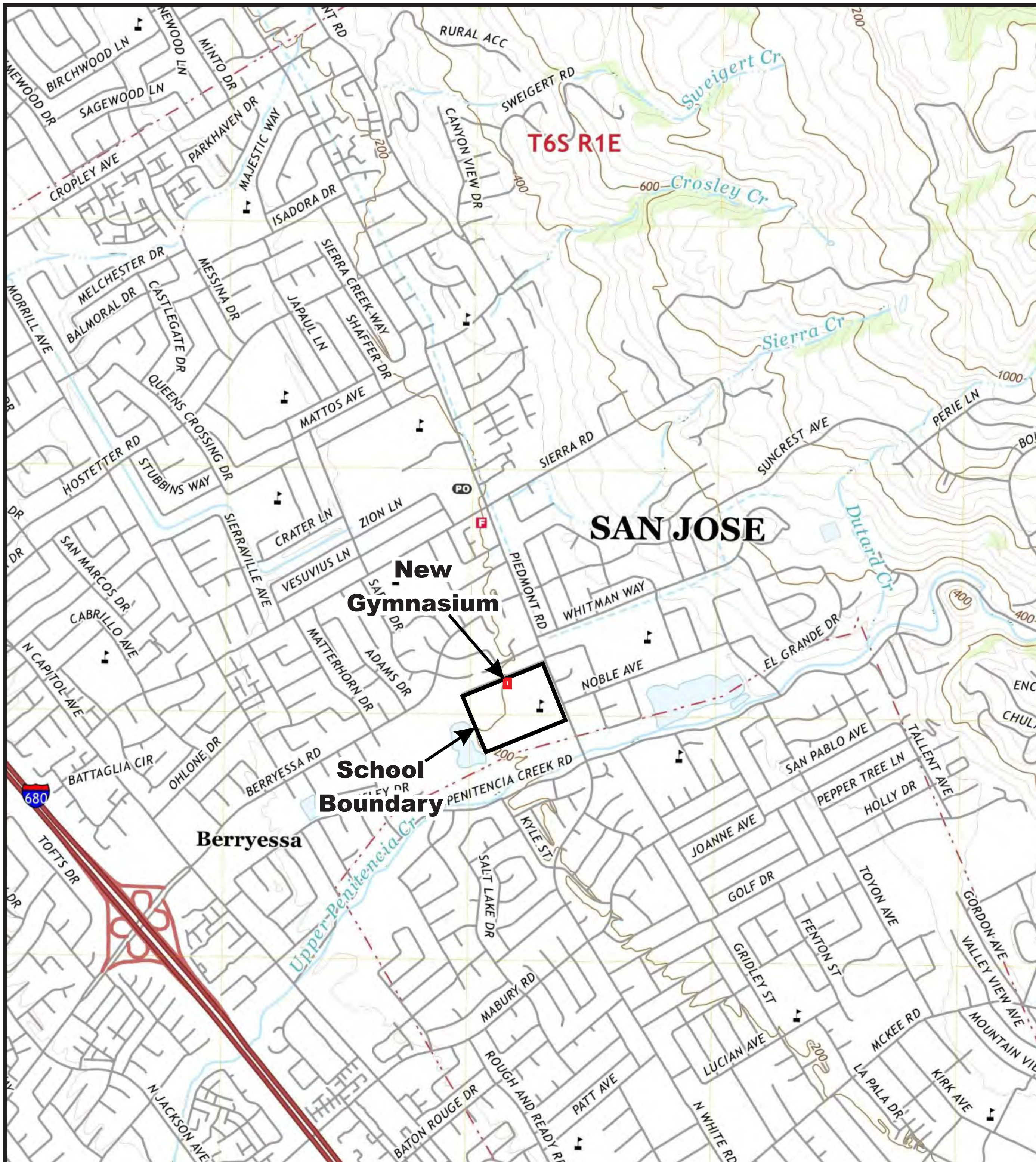
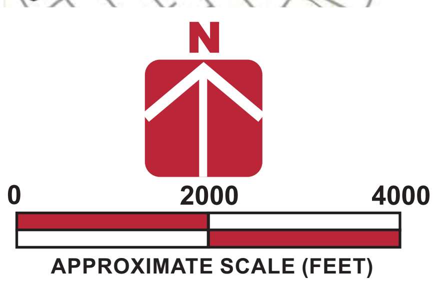
Aerial Photos

The following table shows aerial photos that were reviewed as part of the geologic hazards evaluation.

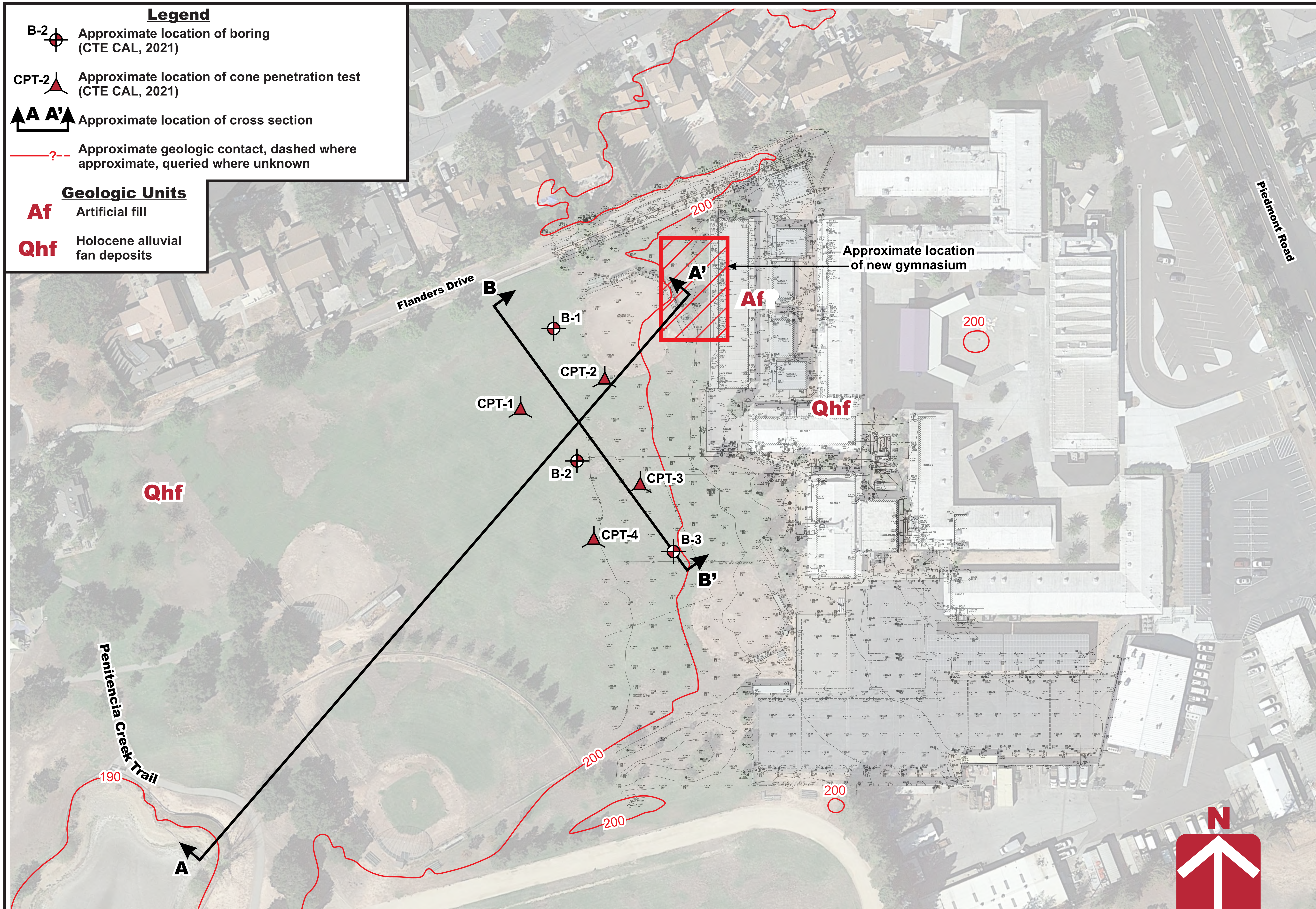
Flight vendor	Flight Line	Date Flown	Photo Frames	Theme	Scale
Google Earth	N/A	9/27/2021	N/A	Color	various
Google Earth	N/A	12/28/1999	N/A	B&W	various
USGS/Western Aerial Photos	GS_VEZR	10/28/1980	4-232, 3-31, 3-33	B&W	1:24,000
Cartwright Aerial Surveys	CAS_2310	5/6/1968	2-157, 2-158	B&W	1:12,000
Cartwright Aerial Surveys	CAS_65_130	5/17/1965	13-188, 13-32	B&W	1:12,000
UCSB Collection	CAS-SCL	7/16/1963	3-40	B&W	1:20,000
Fairchild Aerial Surveys	CIV_1956	6/12/1956	6R-149	B&W	1:20,000
Fairchild Aerial Surveys	CIV_1940	6/5/1940	343-71, 346-58	B&W	1:20,000
Fairchild Aerial Surveys	C_5750	7/31/1939	284-26	B&W	1:20,000





Base: USGS, Calaveras Reservoir 7.5'-Minute Quadrangle, California, dated 2021

CORNERSTONE EARTH GROUP	Vicinity Geologic Map	Project Number 1332-2-2
	Piedmont Middle School New Gymnasium 955 Piedmont Road San Jose, CA	Figure Number Figure 3
		Date August 2022 / Drawn By RRN



Base by Google Earth, dated 09/27/2021
Overlay by Carroll Engineering, Topographic Survey - Sheet 1, dated 04/28/2021,
Carroll Engineering, Topographic Survey - Sheet 2, dated 04/22/2021

Project Number 1332-2-2

Figure Number Figure 2

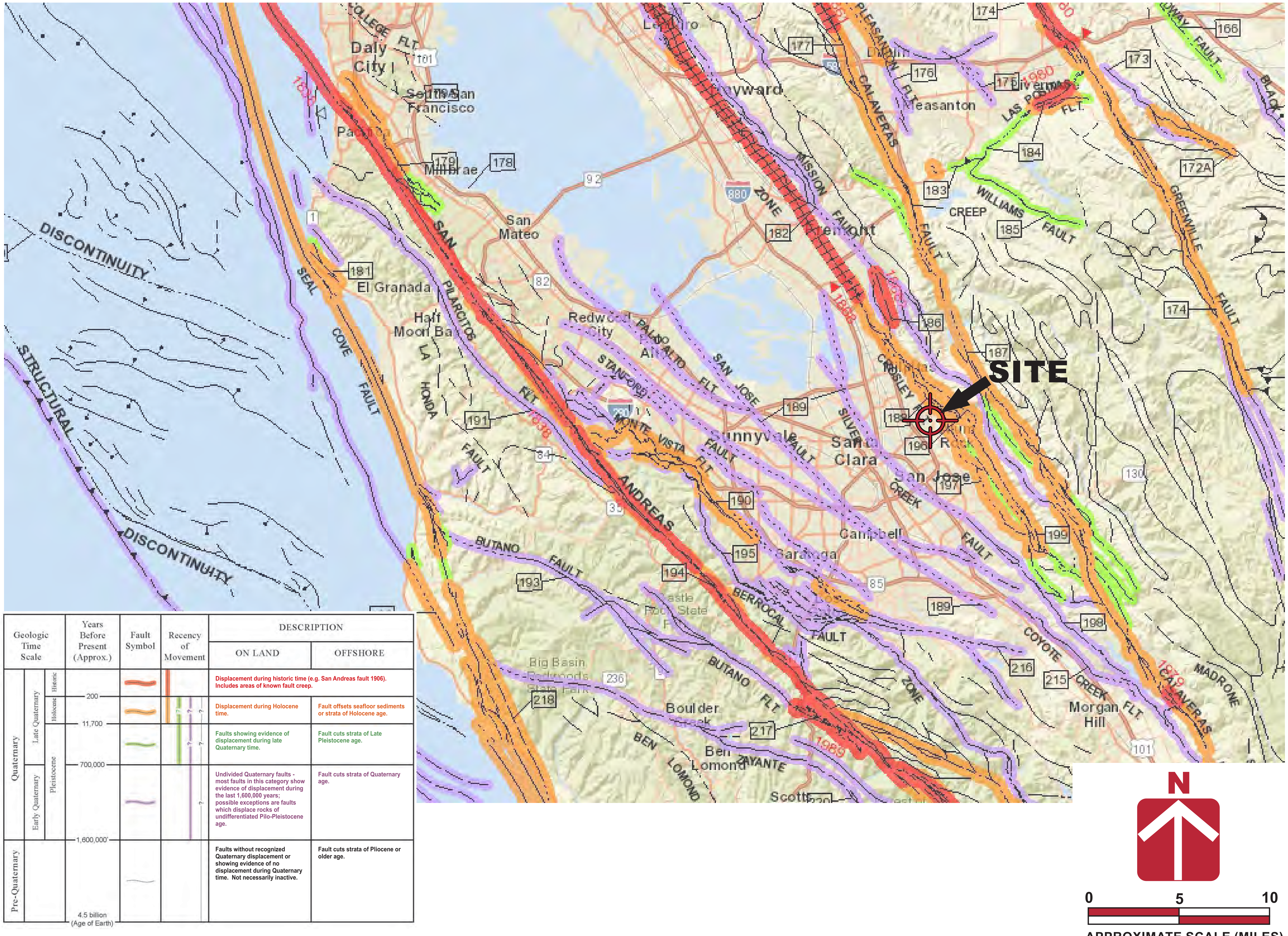
Date August 2022

Drawn By RRN

Site Plan and Geologic Map

Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

CORNERSTONE EARTH GROUP



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

Project Number 1332-2-2

Figure Number Figure 3

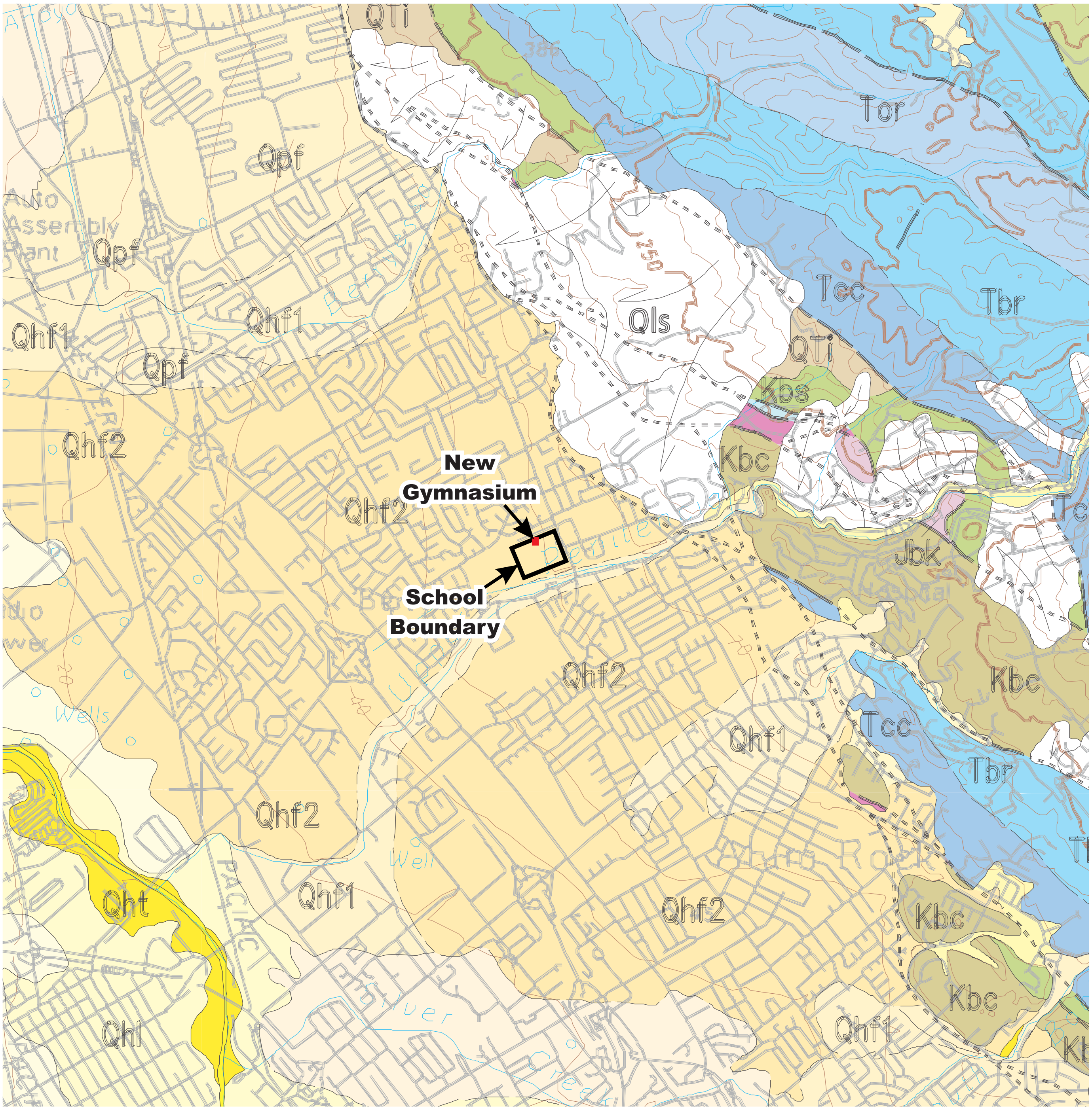
Date August 2022

Drawn By RRN

Regional Fault Map

Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

CORNERSTONE EARTH GROUP



Geologic Units

Qls	Landslide deposits (Holocene)
Qhl	Levee deposits (Holocene)
Qht	Steam terrace deposits (Holocene)
Qhf1	Younger alluvial fan deposits (Holocene)
Qhf2	Older alluvial fan deposits (Holocene)
Qhf	Alluvial fan deposits (Upper Pleistocene)
Kbc	Berryessa Formoration, conglomerate (Cretaceous)

Explanation

Contact - dashed where approximate, dotted where concealed

N

0 4000 8000

APPROXIMATE SCALE (FEET)

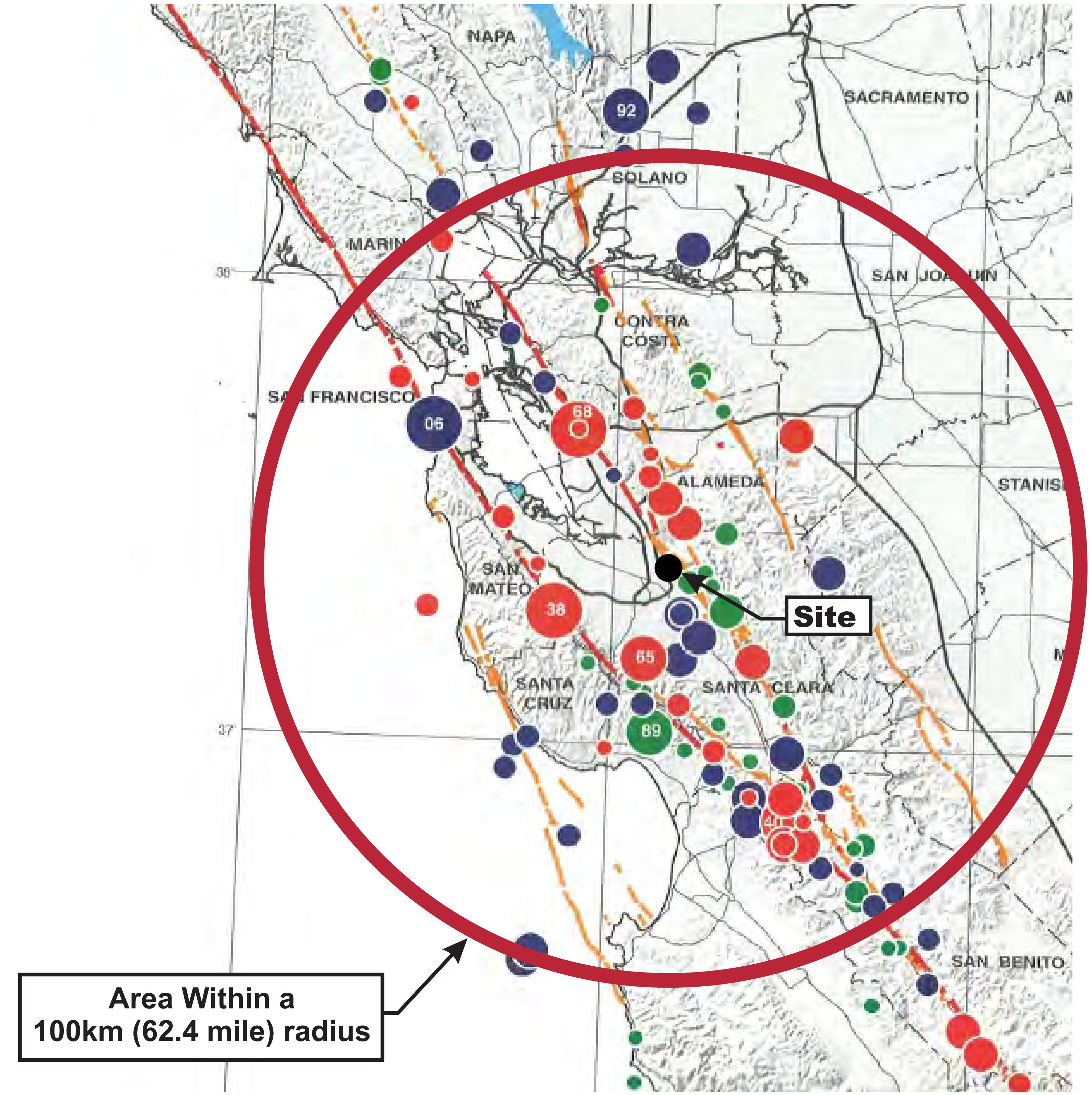
Base: USGS, Preliminary Geologic Map of the San Jose 30x60-Minute Quadrangle, California, by Wentworth, Blake, McLaughlin, and Graymer, 1999

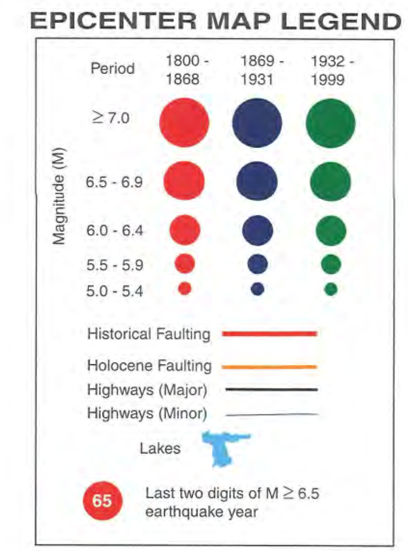
CORNERSTONE EARTH GROUP

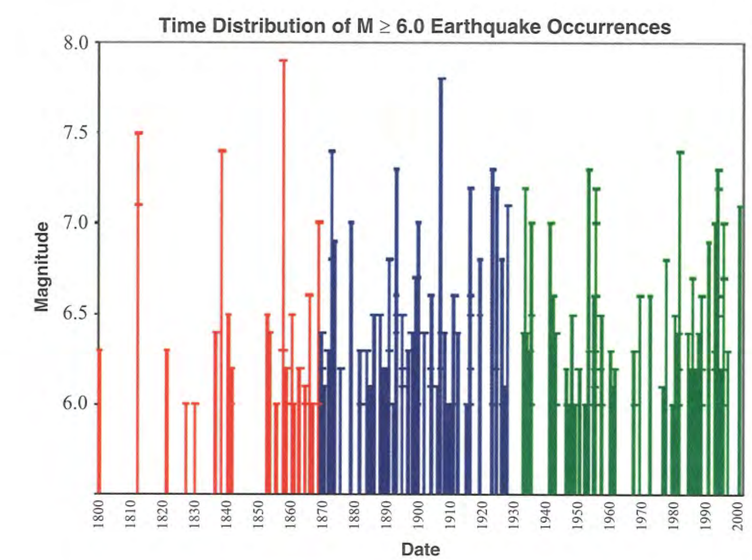
Vicinity Geologic Map

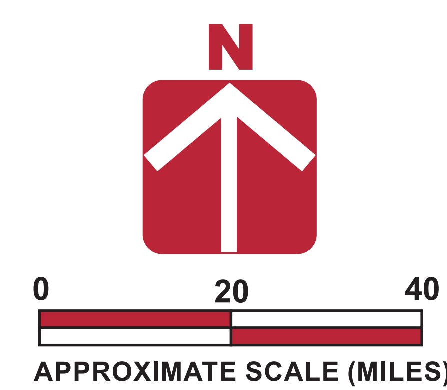
Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

Project Number	1332-2-2
Figure Number	Figure 4
Date	August 2022
Drawn By	RRN









From: T. Toppozada & Others (2000)

Historical Earthquakes Map

Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

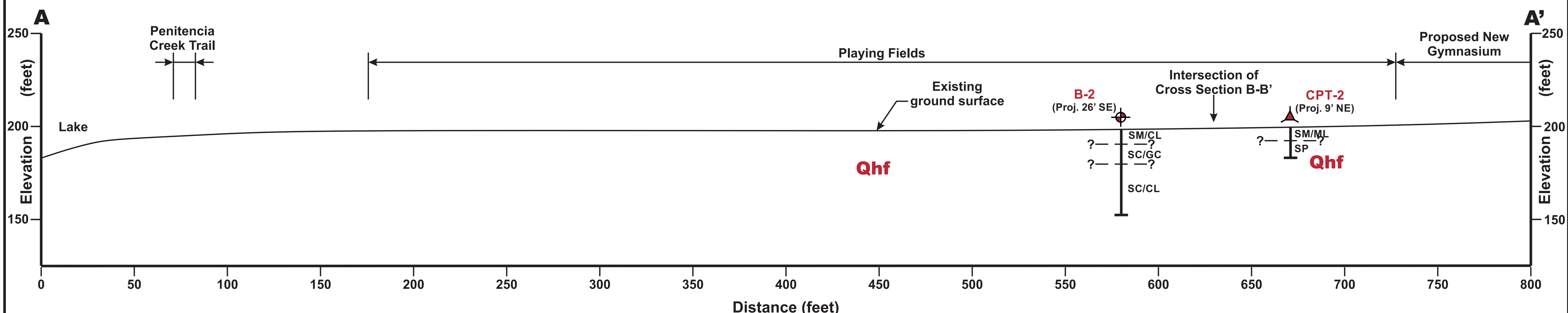
Project Number 1332-2-2

Figure Number Figure 5

Drawn By RRN

Date August 2022

CORNERSTONE EARTH GROUP



Section A-A'

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

Explanation

Geologic Units

Qhf Holocene alluvial fan deposits

Symbols

CL	Lean Clay
ML	Silt
SC	Clayey Sand
SM	Silty Sand
SP	Poorly Graded Sand
GC	Clayey Gravel
B-2	Approximate location of boring (CTE CAL, 2021)
CPT-2	Approximate location of cone penetration test (CTE CAL, 2021)

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The profile is based on Site Plan from CTE (2021).
3) See Figure 2 for location of cross section.

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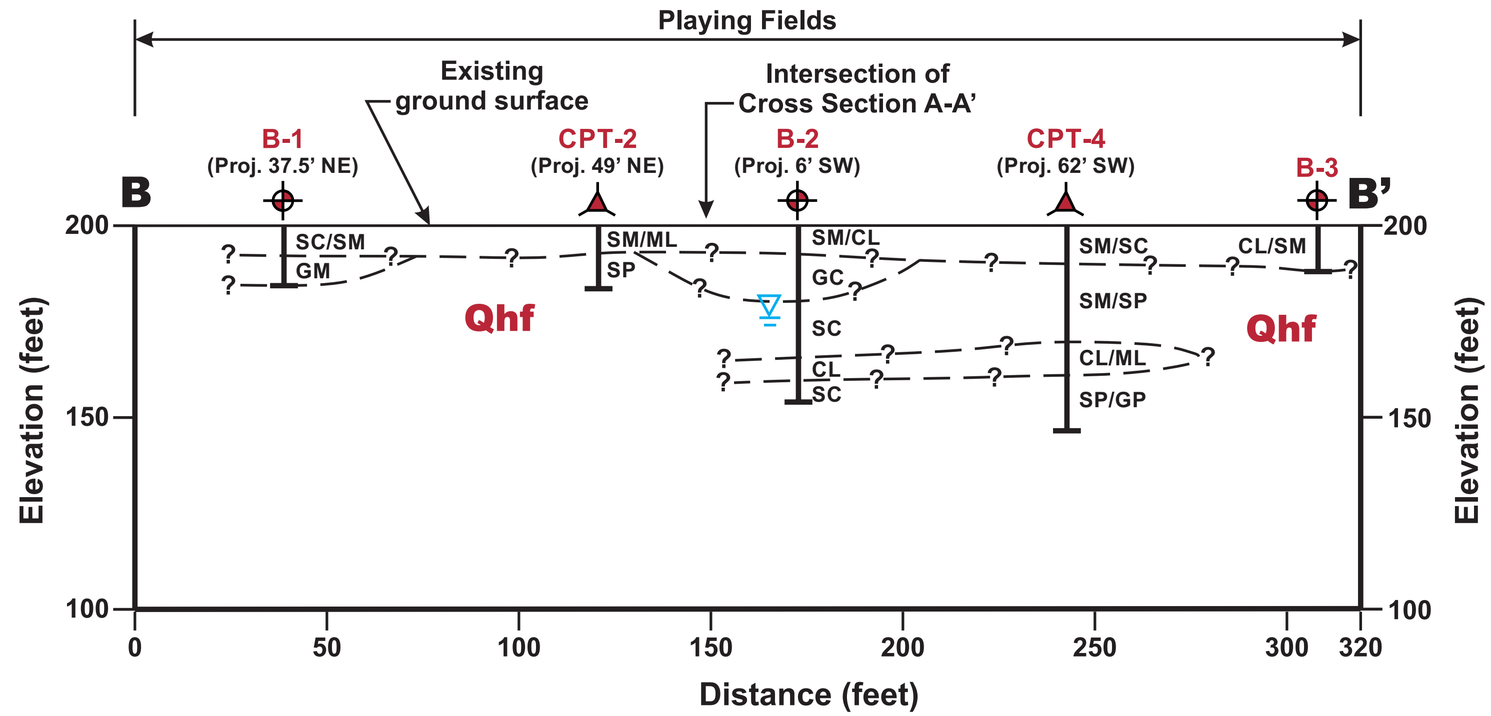
Geologic Cross Section A-A'

Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

Project Number	1332-2-2
Figure Number	Figure 6
Date	August 2022
Drawn By	RRN



Section B-B'

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

Explanation

Geologic Units

Qhf Holocene alluvial fan deposits

Symbols

CL	Lean Clay
ML	Silt
SC	Clayey Sand
SM	Silty Sand
SP	Poorly Graded Sand
GC	Clayey Gravel
GM	Silty Gravel
GP	Poorly Graded
	Approximate ground water depth at time of drilling; actual depth may vary
B-2	Approximate location of boring (CTE CAL, 2021)
CPT-2	Approximate location of cone penetration test (CTE CAL, 2021)

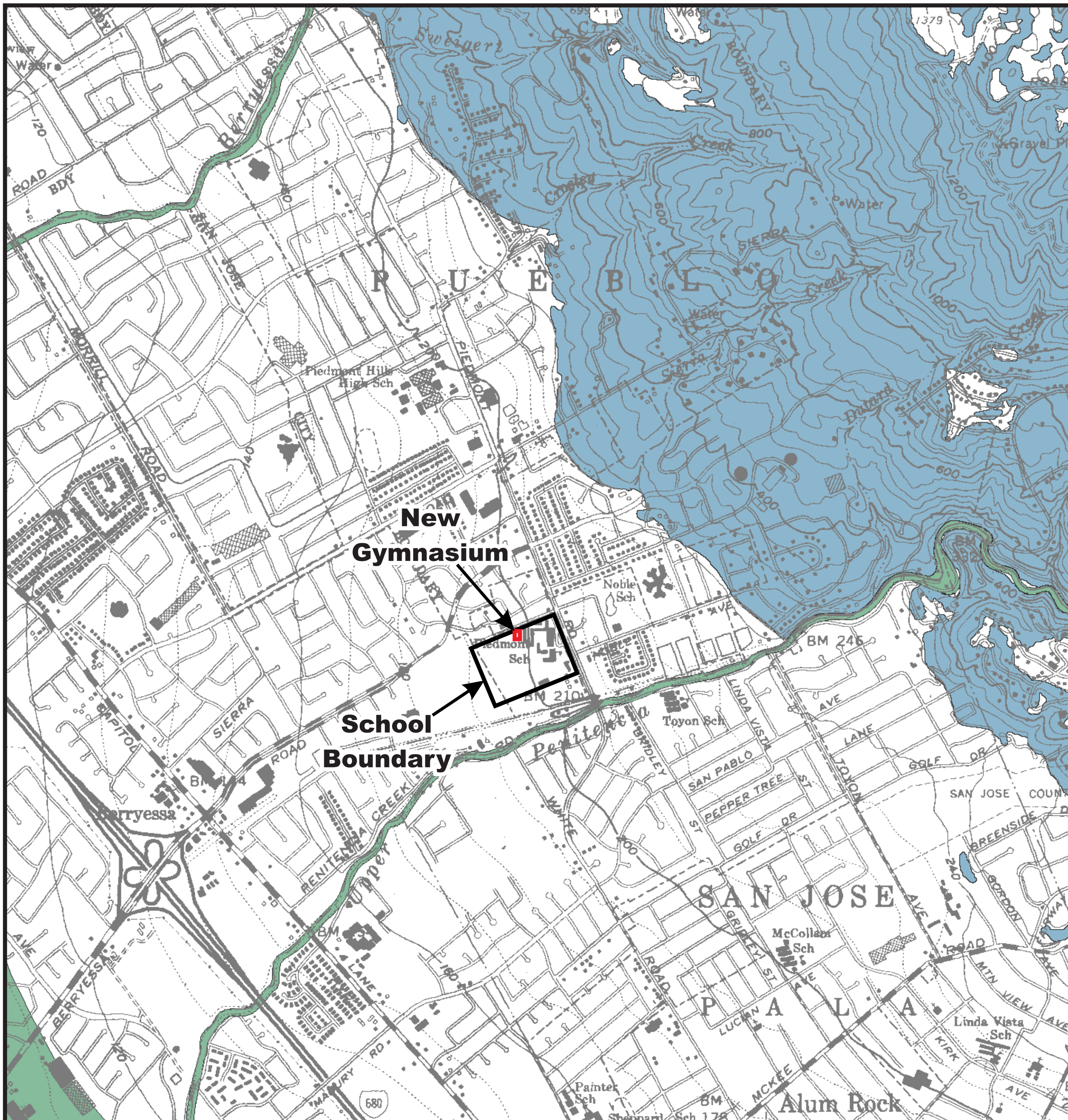
Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The profile is based on Site Plan from CTE (2021).
3) See Figure 2 for location of cross section.

CORNERSTONE EARTH GROUP

Geologic Cross Section B-B'

Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

Project Number	1332-2-2
Figure Number	Figure 7
Date	August 2022
Drawn By	RRN



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.

N

0 2000 4000

APPROXIMATE SCALE (FEET)

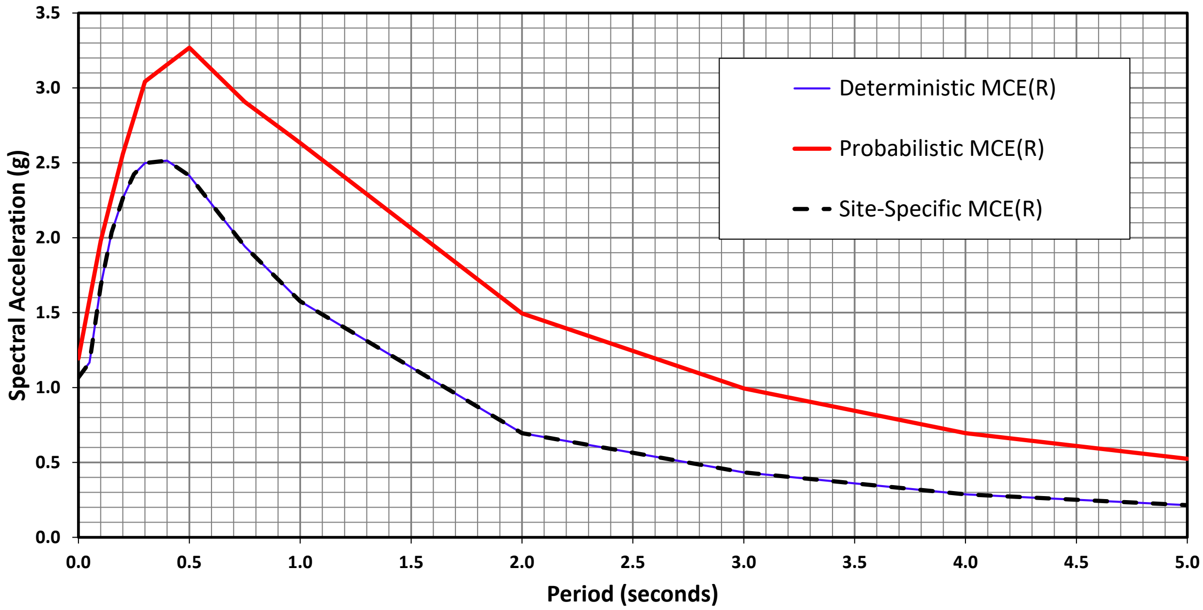
Base: USGS, State of California Seismic Hazard Zones, Calaveras Reservoir 7.5'-Minute Quadrangle, Santa Clara County, California, Released: October 17, 2001

CORNERSTONE EARTH GROUP

Seismic Hazard Map

Piedmont Middle School New Gymnasium
955 Piedmont Road
San Jose, CA

Project Number	1332-2-2
Figure Number	Figure 8
Date	August 2022
Drawn By	RRN



The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

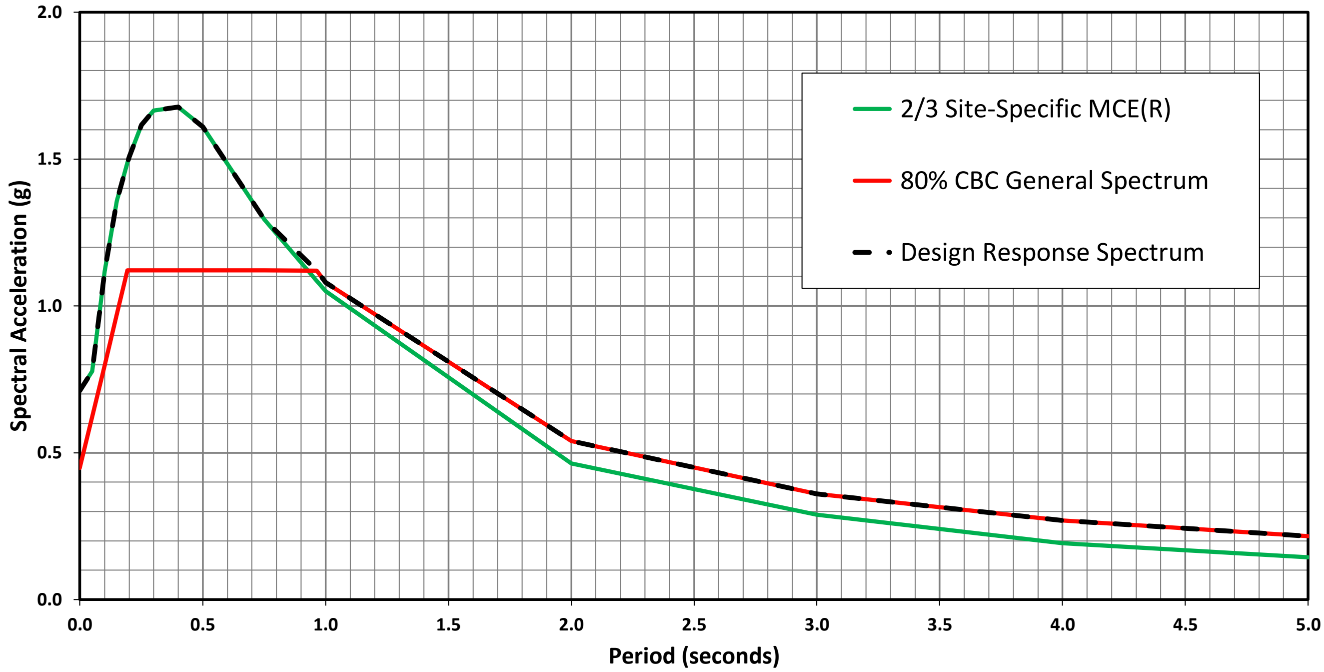
Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	1.067
0.05	1.167
0.10	1.677
0.15	2.037
0.19	2.231
0.20	2.263
0.25	2.424
0.30	2.498
0.40	2.515
0.50	2.414
0.75	1.942
0.96	1.628
1.00	1.575
2.00	0.695
3.00	0.434
4.00	0.287
5.00	0.215

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

MCE_R RESPONSE SPECTRA	FIGURE 9	
Piedmont MS New Gym 955 Piedmont Road San Jose, CA 95132	PROJECT NO. 1332-2-2	
	February 4, 2020	ELS



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.712
0.05	0.778
0.10	1.118
0.15	1.358
0.19	1.488
0.20	1.509
0.25	1.616
0.30	1.666
0.40	1.676
0.50	1.609
0.75	1.295
0.96	1.120
1.00	1.080
2.00	0.540
3.00	0.360
4.00	0.270
5.00	0.216

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	295
Site Latitude (degrees)	37.39319
Site Longitude (degrees)	-121.84613
Risk Category	III
Building Period (sec)	Unknown
Importance Factor, I_e	1.25
[1]Site Specific PGA_M (g)	0.97

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

Design Acceleration Parameters[1]	
S_{DS}	1.509
S_{D1}	1.080
S_{MS}	2.263
S_{M1}	1.620

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

DESIGN RESPONSE SPECTRA	FIGURE 10	
Piedmont MS New Gym 955 Piedmont Road San Jose, CA 95132	PROJECT NO. 1332-2-2	
	February 4, 2020	ELS

APPENDIX A: PREVIOUS BORINGS AND CPTS – PERFORMED BY CTE CAL, INC.

CTE CAL, INC.

SACRAMENTO
3628 Madison Ave., Ste. 22
Sacramento, CA 95660
Ph: (916) 331 - 6030

FREMONT
46716 Fremont Blvd.
Fremont, CA 94538
Ph: (510) 573 - 6362

MODESTO
4230 Kiernan Ave., Ste. 150
Modesto, CA 95356
Ph: (209) 543 - 1799

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS < 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL SAND MIXTURES, LITTLE OF NO FINES
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS < 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, SLIGHTLY PLASTIC CLAYEY SILTS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, SILTS OR LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILTS AND CLAYS
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" 4 10 40 200

CLEAR SQUARE SIEVE OPENING U.S. STANDARD SIEVE SIZE

PENETRATION RESISTANCE AND PROPERTIES BASED ON THE SPT (PECK ET AL.)

	SPT (N) Blows/ft	Relative Density		SPT (N) Blows/ft	Consistency
Sands	0-4	Very Loose	Clays	<2	Very Soft
	4-10	Loose		2-4	Soft
	10-30	Medium		4-8	Medium
	30-50	Dense		8-15	Stiff
	Over 50	Very Dense		15-30	Very Stiff
				Over 30	Hard

ADDITIONAL TESTS

(OTHER THAN TEST PIT AND BORING LOG COLUMN HEADINGS)

MAX- Maximum Dry Density	PM- Permeability	PP- Pocket Penetrometer
GS- Grain Size Distribution	SG- Specific Gravity	WA- Wash Analysis
SE- Sand Equivalent	HA- Hydrometer Analysis	DS- Direct Shear
EI- Expansion Index	AL- Atterberg Limits	UC- Unconfined Compression
CHM- Sulfate & Chloride, pH, Resistivity	RV- R-Value	MD- Moisture/Density
COR - Corrosivity	CN- Consolidation	M- Moisture
SD- Sample Disturbed	CP- Collapse Potential	SC- Swell Compression
REM- Remolded	HC- Hydrocollapse	OI- Organic Impurities

FIGURE: BL1

CTE CAL
3628 Madison Avenue, Suite 22 | Sacramento, CA 95660 | 916.331.6030 | Fax 916.331.6037

PROJECT: DRILLER: SHEET: of
CTE JOB NO: DRILL METHOD: DRILLING DATE:
LOGGED BY: SAMPLE METHOD: ELEVATION:

Depth (Feet)	Bulk Sample	Driven Type	Blows/Foot	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	BORING LEGEND DESCRIPTION	Laboratory Tests
0								Block or Chunk Sample	
								Bulk Sample	
5									
								Standard Penetration Test	
10								Modified Split-Barrel Drive Sampler (Cal Sampler)	
								Thin Walled Army Corp. of Engineers Sample	
15								Groundwater Table	
								Soil Grading Change- Minor Deviation	
								Soil Type or Classification Change	
20								? — ? — ? — ? — ? — ? — ? —	
								Formation Change [(Approximate boundaries queried (?)]	
25						"SM"		Quotes are placed around classifications where the soils exist in situ as bedrock	

FIGURE: BL2

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PROJECT:	New Gym Piedmont Middle School	DRILLER:	Exploration Geoservices	SHEET:	1 of 4
CTE JOB NO:	60-0938G	DRILL METHOD:	8" Hollow Auger	DRILLING DATE:	6/24/2021
LOGGED BY:	Walter Raymond	SAMPLE METHOD:	SPT (2.5" Mod Cal Liners)	ELEVATION:	EGS

BORING: B-1

Depth (Feet)	Bulk Sample	Driven Type	Blows/ 6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0								≈6" of topsoil with gass and vegetation	
3			7 10 10			SC		Brown/light brown, dry, weak cementation, medium, clayey SAND w/some subangular gravels and vegetation. ≈ 95% recovery.	
4.5 5			10 28 48			SM		Brown/light brown, dry, weak cementation, very dense, silty SAND w/some subangular gravels and cobbles. ≈ 80% recovery.	
8 10			50/5"			GM		Gray, dry, angular gravels with some fines. ≈ 10% recovery.	
15			50/3"					No recovery, only small rock shaving remnants. 0% recovery.	
20								Drilling terminated at 15 feet below egs No groundwater encountered Hole backfilled w/neat cement	

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PROJECT:	New Gym Piedmont Middle School	DRILLER:	Exploration Geoservices	SHEET:	2 of 4
CTE JOB NO:	60-0938G	DRILL METHOD:	8" Hollow Auger	DRILLING DATE:	6/24/2021
LOGGED BY:	Walter Raymond	SAMPLE METHOD:	SPT (1.5" Liners)	ELEVATION:	EGS

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/ 6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0								≈6" of topsoil with gass and vegetation	
3–4			5 4 8			SM		Brown, dry, loose, weak cementation, silty SAND w/vegetation. ≈ 85% recovery.	
5–7			6 13 21			CL		Brown/dark brown, dry, hard, moderate cementation, low plastic sandy CLAY w/vegetation. ≈ 95% recovery.	LL = 36% PL = 17% PI = 19%
10			23 36 50/5"			GC		Brown/dark brown/orange, dry, coarse, subangular clayey GRAVEL w/shavings. ≈ 50% recovery.	
15–19			31 35 50/6"			GC		Brown/dark brown/orange, dry, coarse, subangular clayey GRAVEL w/shavings. ≈ 66% recovery.	
20			15 7 9	101.7	8	SC		Brown, moist, medium, low plastic clayey SAND with gravels. ≈ 40% recovery.	FC = 48%
24								Groundwater encountered at ≈ 24 feet below egs	

Continued

PROJECT:	New Gym Piedmont Middle School	DRILLER:	Exploration Geoservices	SHEET:	3 of 4
CTE JOB NO:	60-0938G	DRILL METHOD:	8" Hollow Auger	DRILLING DATE:	6/24/2021
LOGGED BY:	Walter Raymond	SAMPLE METHOD:	SPT (1.5" Liners)	ELEVATION:	EGS

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/ 6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
25			16 12 16			SC		Brown, wet, medium, low plastic clayey SAND with subangular gravel. ≈ 95% recovery	
30			6 6 7			SC		Brown, wet, medium, low plastic clayey SAND with subangular gravel. ≈ 30% recovery	LL = 29% PL = 19% PI = 10% FC = 15.7%
33									
35			5 10 12	111.3	22	CL		Light brown, moist, medium plastic, very stiff sandy CLAY w/subangular gravels. ≈ 80% recovery	FC = 51.1%
39									
40			20 38 17	110.8	7	SC		Light brown, damp, moderate cementation, dense, low plastic clayey SAND w/subangular gravels. ≈ 60% recovery	LL = 39% PL = 22% PI = 17% FC = 18.9%
45			4 6 8			SC		Light brown, moist, low cementation, medium, low plastic clayey SAND. ≈ 66% recovery	FC = 45.6%

Drilling terminated at 45 feet below egs
Groundwater encountered at 24 feet below egs
Hole backfilled w/neat cement

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PROJECT:	New Gym Piedmont Middle School	DRILLER:	Exploration Geoservices	SHEET:	4 of 4
CTE JOB NO:	60-0938G	DRILL METHOD:	8" Hollow Auger	DRILLING DATE:	6/24/2021
LOGGED BY:	Walter Raymond	SAMPLE METHOD:	SPT (2.5" Mod Cal Liners)	ELEVATION:	EGS

BORING: B-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/ 6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0								≈6" of topsoil with gass and vegetation	
3–4			12 13 18			CL		Dark brown, dry, hard, moderate cementation, low plastic sandy CLAY w/few pebbles and vegetation. ≈ 95% recovery.	
5–8			48 46 50/5"			SM		Light brown, dry, very dense, moderate cementation, silty SAND with subangular gravels. ≈ 95% recovery.	LL = 42% PL = 23% PI = 20%
10			29 28 30			SC		Light brown/brown, moist, very dense, moderate cementation, low plastic clayey SAND with subangular gravels/cobbles. ≈ 95% recovery	

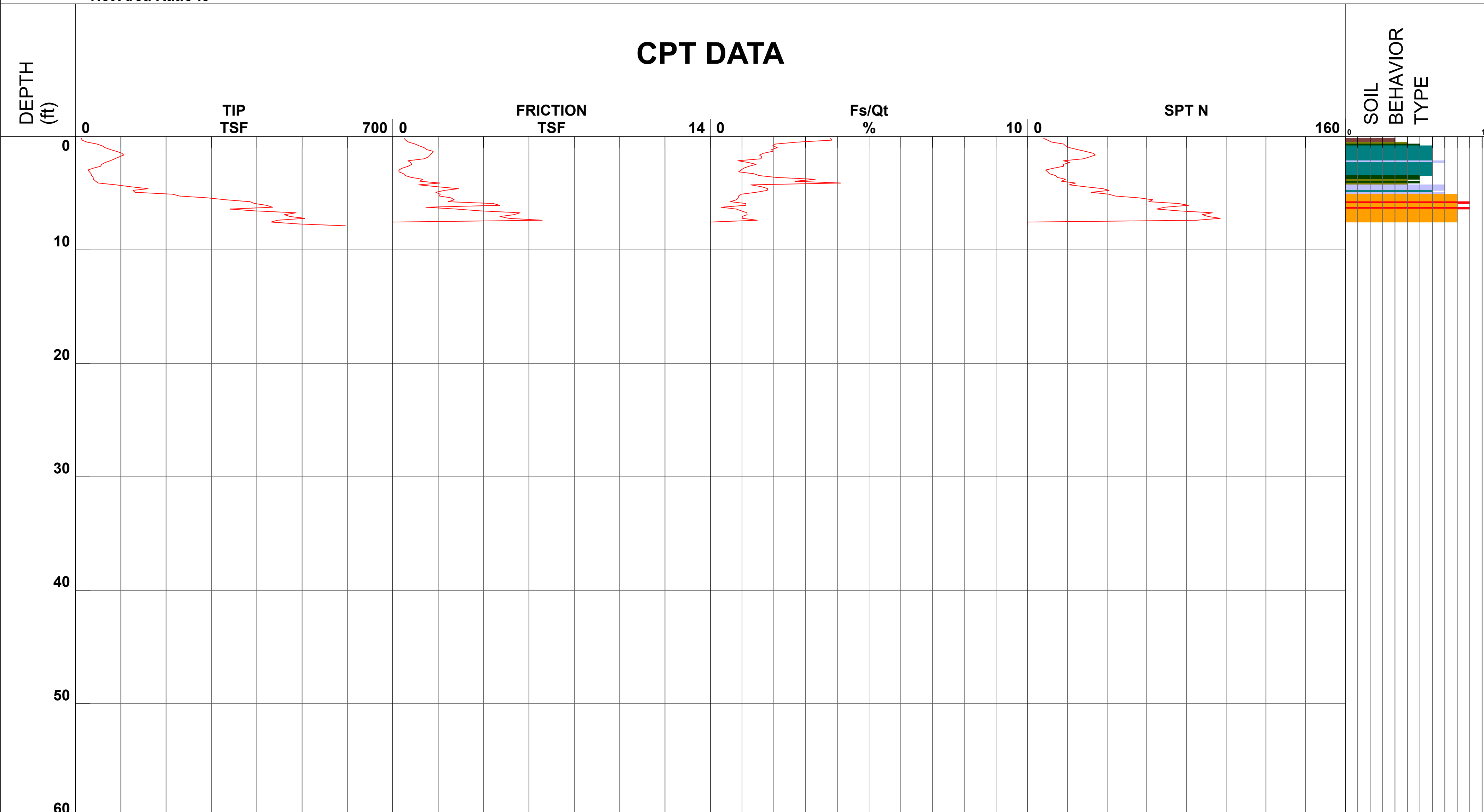
Drilling terminated at 10 feet below egs
No groundwater encountered
Hole backfilled w/neat cement

Depth scale: 0, 3, 4, 5, 8, 10, 15, 20

CTE CAL INC

Project	New Gym Piedmont Middle School	Operator	AJ-OO	Filename	SDF(776).cpt
Job Number	P21-224	Cone Number	DDG1589	GPS	
Hole Number	CPT-01	Date and Time	7/23/2021 3:57:08 PM	Maximum Depth	7.87 ft
EST GW Depth During Test			24.00 ft		

Net Area Ratio .8



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

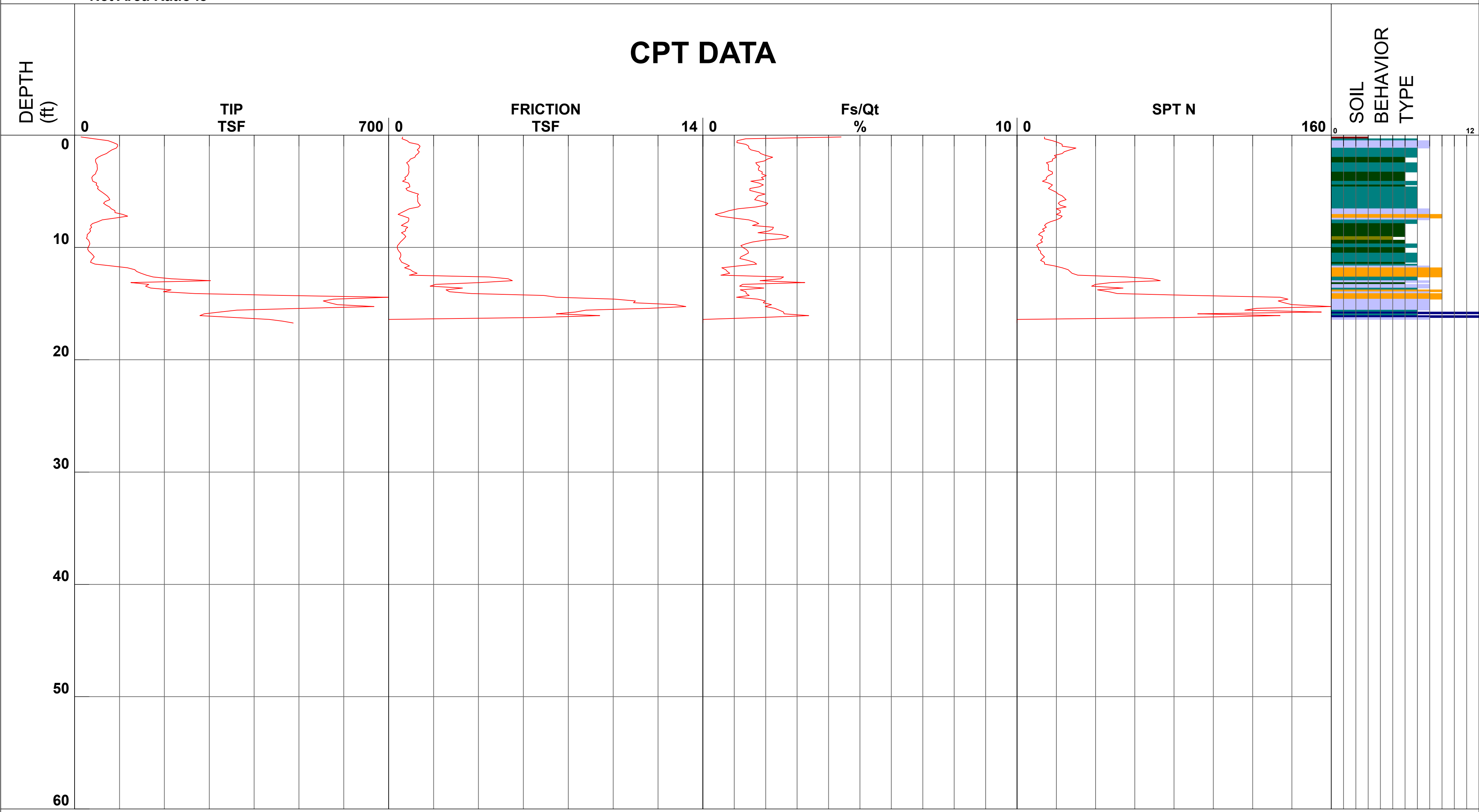
Cone Size 15cm squared

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

CTE CAL INC

Project	New Gym Piedmont Middle School	Operator	AJ-OO	Filename	SDF(773).cpt
Job Number	P21-224	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	7/23/2021 12:55:06 PM	Maximum Depth	16.73 ft
EST GW Depth During Test		24.00 ft			

Net Area Ratio .8



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

Cone Size 15cm squared

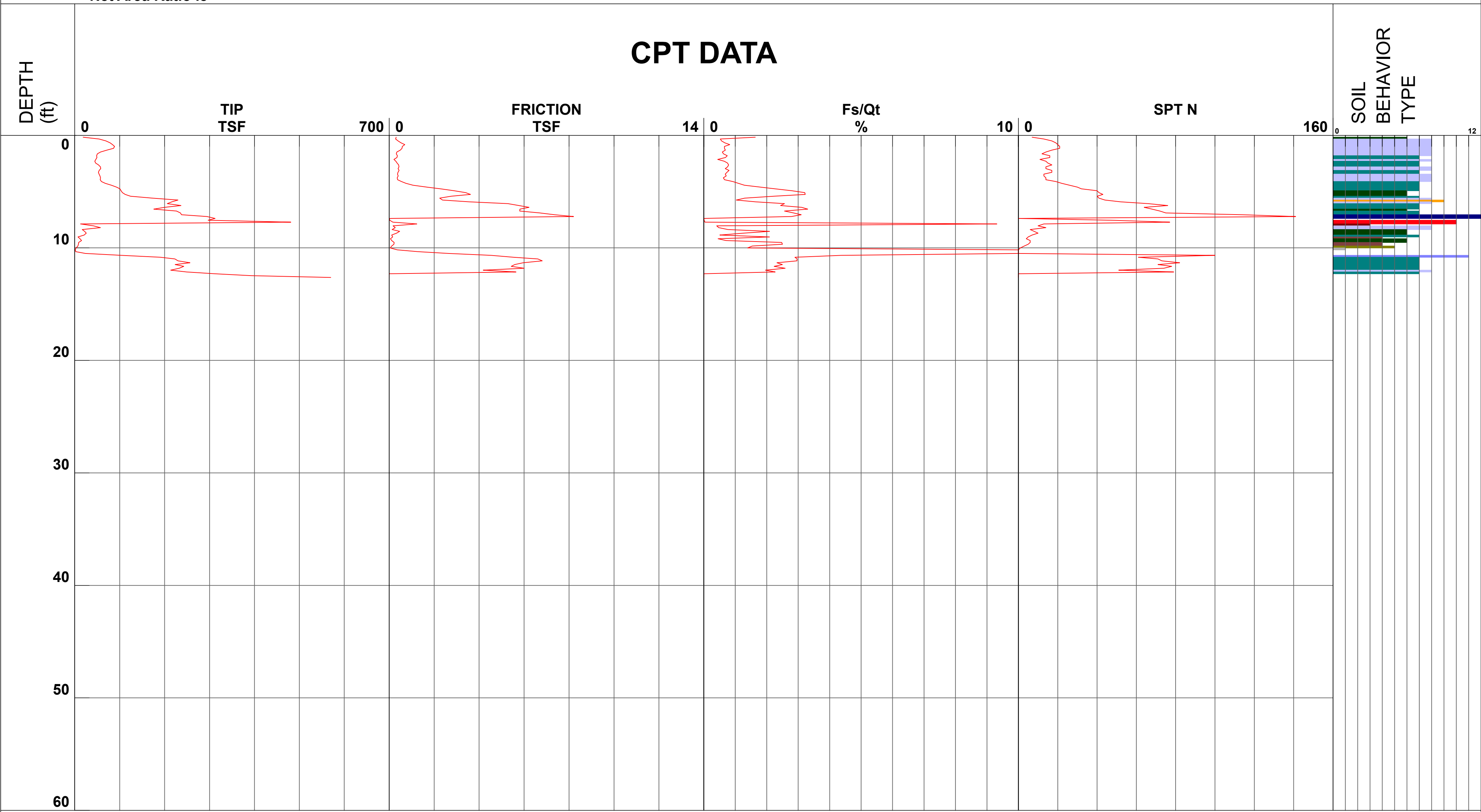
S*Soil behavior type and SPT based on data from UBC-1983

CTE CAL INC

Project	New Gym Piedmont Middle School	Operator	AJ-OO	Filename	SDF(774).cpt
Job Number	P21-224	Cone Number	DDG1589	GPS	
Hole Number	CPT-03	Date and Time	7/23/2021 1:57:03 PM	Maximum Depth	12.63 ft
EST GW Depth During Test			24.00 ft		

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

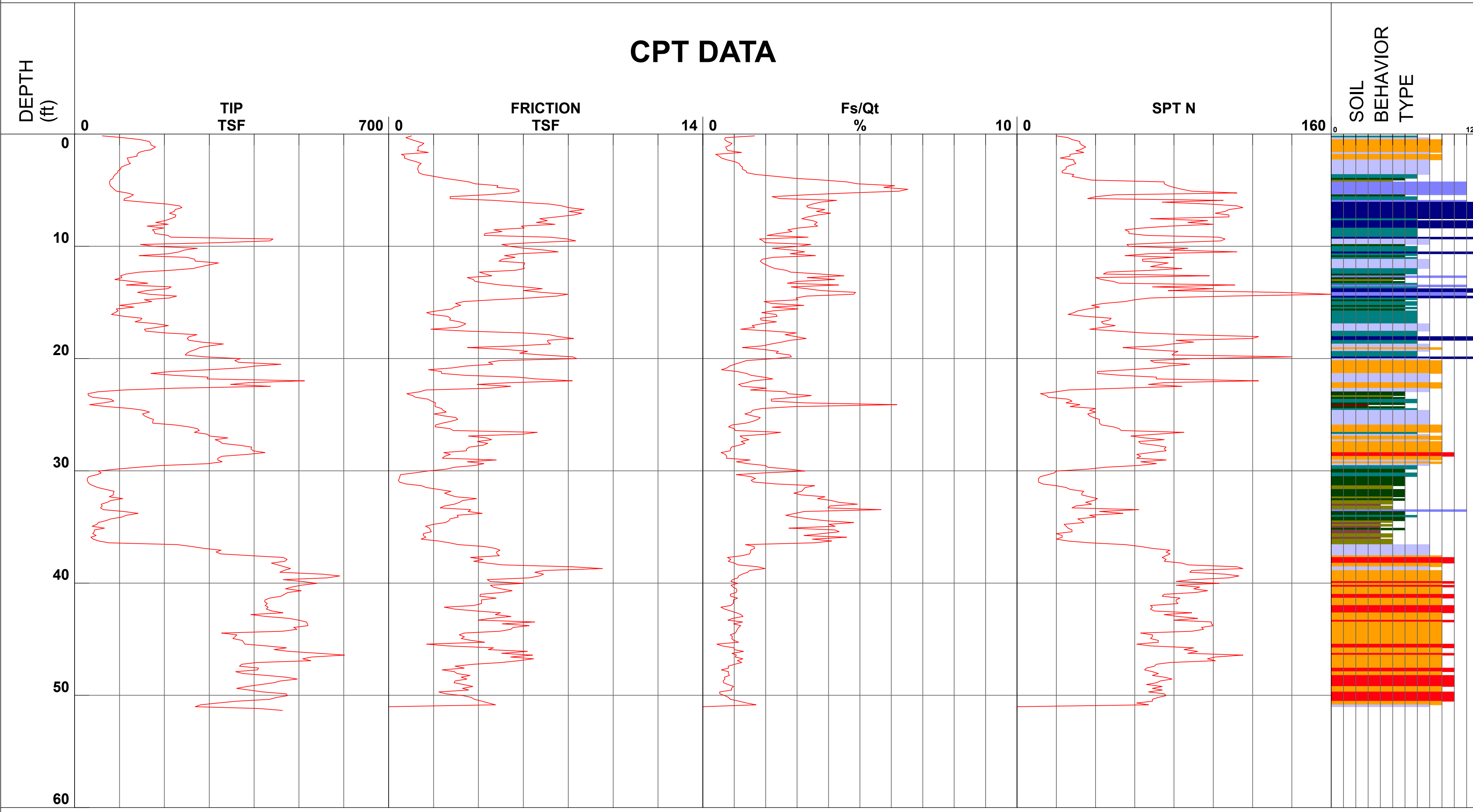
S*Soil behavior type and SPT based on data from UBC-1983

CTE CAL INC

Project	New Gym Piedmont Middle School	Operator	AJ-OO	Filename	SDF(775).cpt
Job Number	P21-224	Cone Number	DDG1589	GPS	
Hole Number	CPT-04	Date and Time	7/23/2021 2:53:59 PM	Maximum Depth	51.34 ft
EST GW Depth During Test		24.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

APPENDIX B: LABORATORY TESTS – PERFORMED BY CTE CAL, INC.

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CIVIL ENGINEERING | GEOTECHNICAL | ENVIRONMENTAL | CONSTRUCTION INSPECTION AND TESTING

ATTERBERG LIMITS
B-2 40'

ASTM D4318

Job Name: New Gym Piedmont MS **Date:** 06/10/21

Job Number: 60-0938G **Lab #:** Fremont

Sample Number:

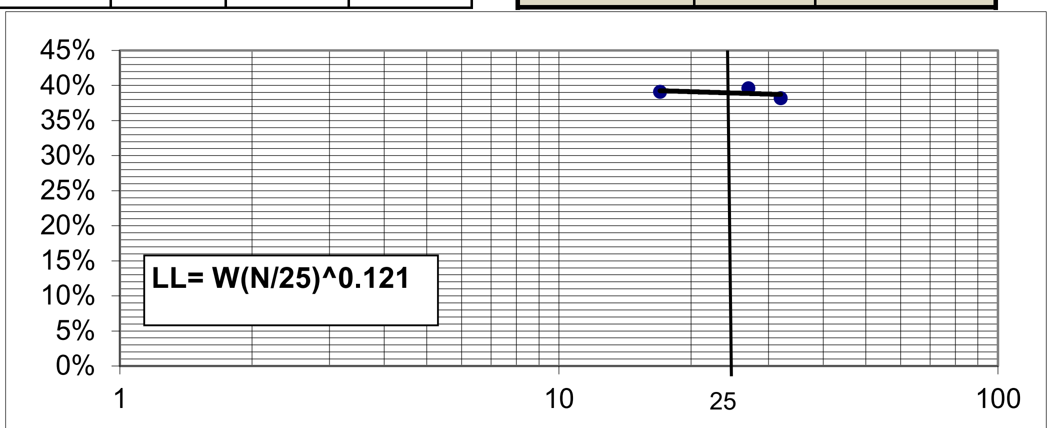
	LIQUID LIMITS		
WET SOIL	7.37	8.18	8.07
DRY SOIL	5.30	5.86	5.84
WATER	2.07	2.32	2.23
# BLOWS	17	27	32
% MOIST	**39.06%**	**39.59%**	**38.18%**

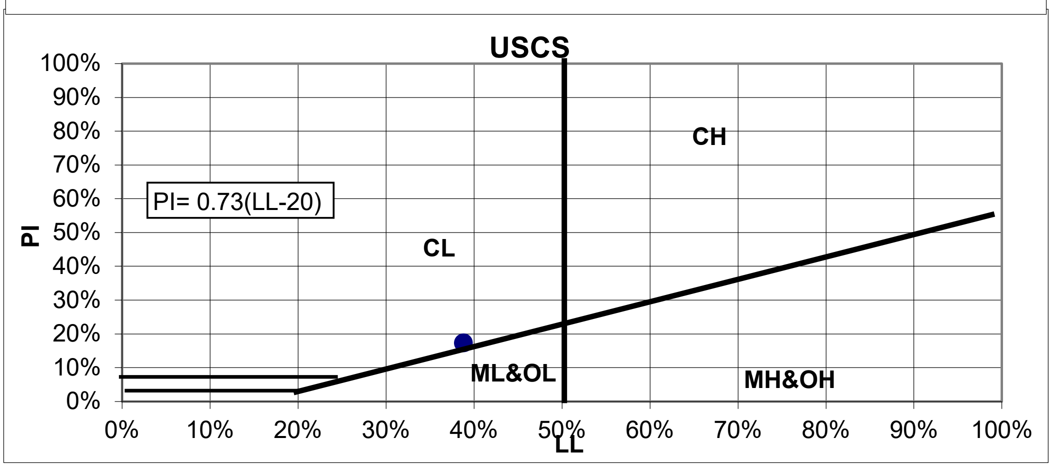
PLASTIC LIMIT	
7.04	5.5
5.82	4.5
1.22	1
20.96%	**22.22%**

Method used		Dry
	X	Moist
Plastic Limit	X	Hand rolled
		Mach.rolling device
Liquid Limit Apparatus	X	Manual
		Mechanical
Casagrande Grooving tool		Metal
	X	Plastic

NE POINT	**37.3%**	**40.0%**	**39.3%**

LL	**PL**	**PI**
39%	**22%**	**17%**





REVIEWED BY: Fella Damardji **DATE:** 6/14/2021

CTE CAL
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CIVIL ENGINEERING | GEOTECHNICAL | ENVIRONMENTAL | CONSTRUCTION INSPECTION AND TESTING

ATTERBERG LIMITS

ASTM D4318

Job Name: New Gym Piedmont MS **Date:** 06/25/21

Job Number: 60-0938G **Lab #:** Fremont

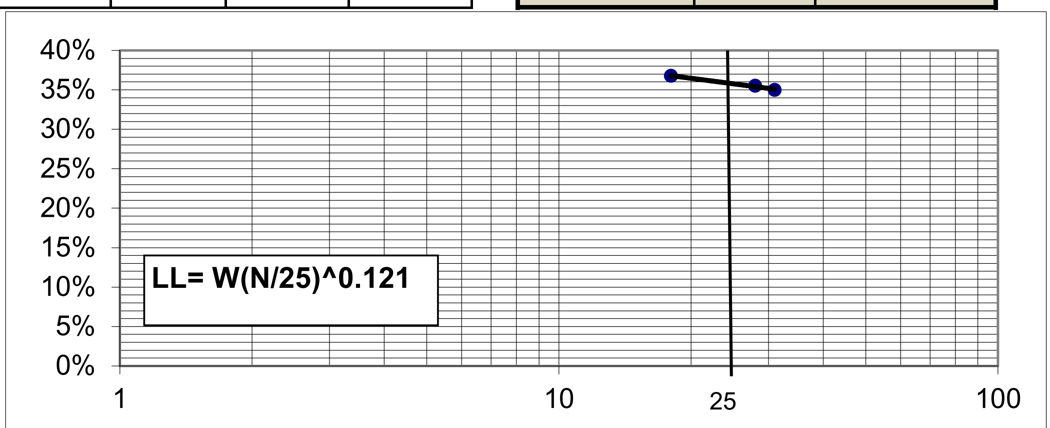
Sample Number: B2-5'

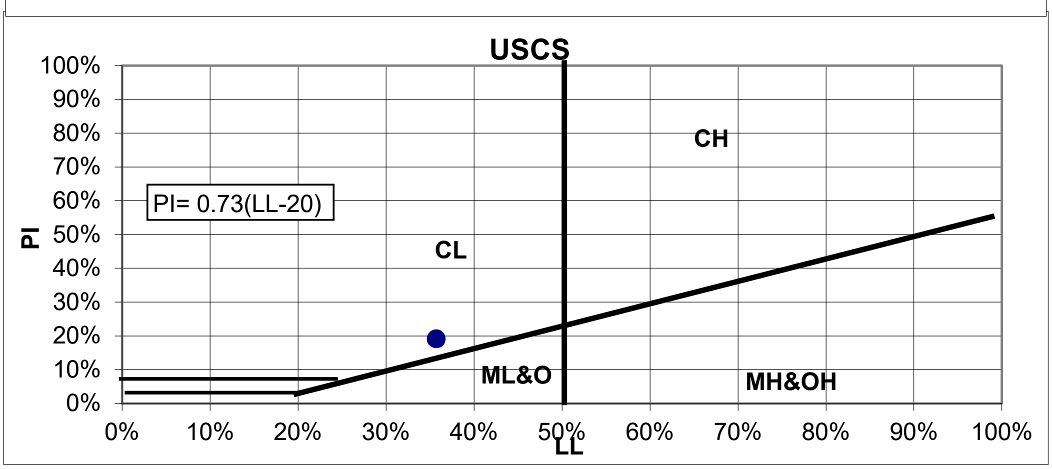
	LIQUID LIMITS			PLASTIC LIMIT	
WET SOIL	8.74	8.24	7.60	7.60	6.42
DRY SOIL	6.39	6.08	5.63	6.49	5.52
WATER	2.35	2.16	1.97	1.11	0.9
# BLOWS	18	28	31		
% MOIST	**36.78%**	**35.53%**	**34.99%**	**17.10%**	**16.30%**

Method used		Dry
	X	Moist
Plastic Limit	X	Hand rolled
		Mach.rolling device
Liquid Limit Apparatus	X	Manual
		Mechanical
Casagrande Grooving tool		Metal
	X	Plastic

NE POINT	**35.3%**	**36.0%**	**35.9%**

LL	PL	PI
36%	**17%**	**19%**





REVIEWED BY: Fella Damardji **DATE:** 6/30/2021

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ATTERBERG LIMITS

ASTM D4318

Job Name: New Gym Piedmont MS **Date:** 06/25/21

Job Number: 60-0938G **Lab #:** Fremont

Sample Number: B2-30'

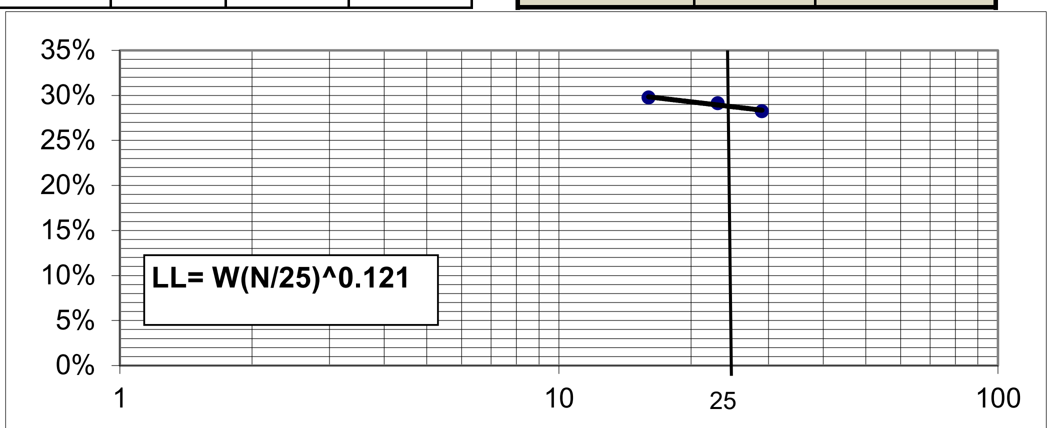
	LIQUID LIMITS		
WET SOIL	9.68	9.53	8.72
DRY SOIL	7.46	7.38	6.80
WATER	2.22	2.15	1.92
# BLOWS	16	23	29
% MOIST	**29.76%**	**29.13%**	**28.24%**

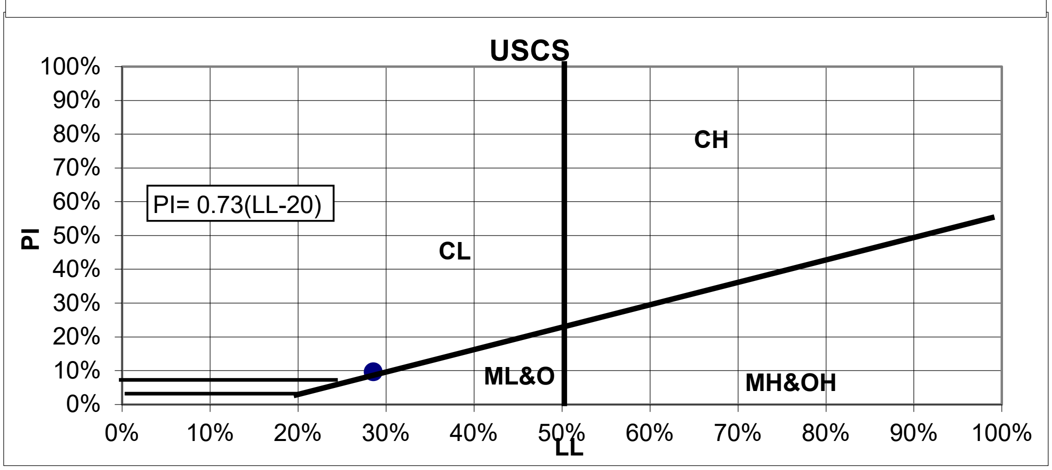
PLASTIC LIMIT	
7.02	6.68
5.90	5.61
1.12	1.07
18.98%	**19.07%**

Method used		Dry
	X	Moist
Plastic Limit	X	Hand rolled
		Mach.rolling device
Liquid Limit Apparatus	X	Manual
		Mechanical
Casagrande Grooving tool		Metal
	X	Plastic

NE POINT	**28.2%**	**28.8%**	**28.7%**

LL	PL	PI
29%	**19%**	**10%**





REVIEWED BY: Fella Damardji **DATE:** 6/30/2021

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ATTERBERG LIMITS
ASTM D4318

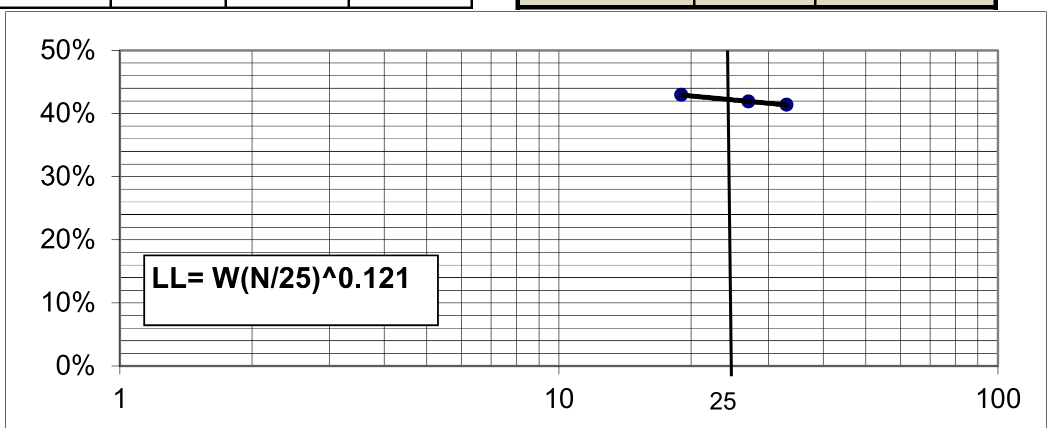
Job Name: New Gym Piedmont MS **Date:** 06/25/21
Job Number: 60-0938G **Lab #:** Fremont
Sample Number: B3- 5'

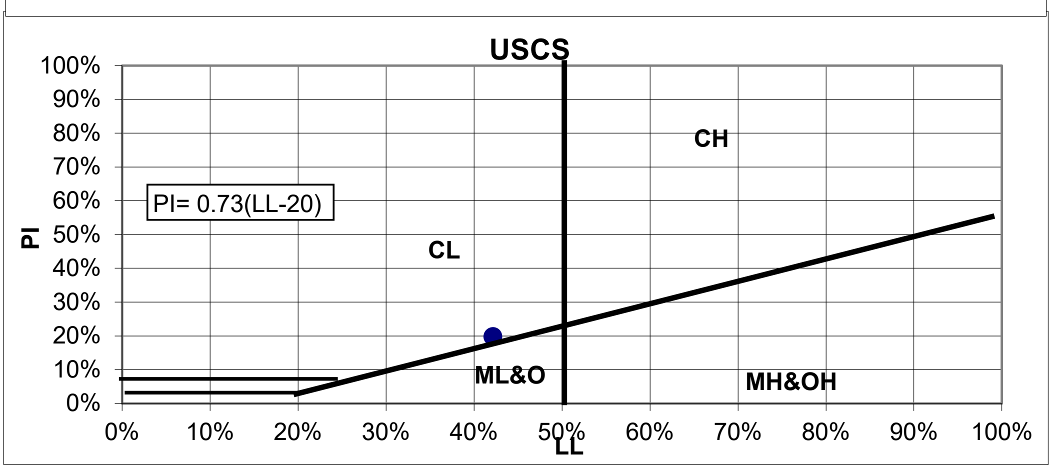
	LIQUID LIMITS		
WET SOIL	9.32	7.25	8.54
DRY SOIL	6.52	5.11	6.04
WATER	2.80	2.14	2.50
# BLOWS	19	27	33
% MOIST	**42.94%**	**41.88%**	**41.39%**
NE POINT	**41.5%**	**42.3%**	**42.8%**

PLASTIC LIMIT	
6.73	6
5.50	4.89
1.23	1.11
22.36%	**22.70%**

Method used		Dry
	X	Moist
Plastic Limit	X	Hand rolled
		Mach.rolling device
Liquid Limit Apparatus	X	Manual
		Mechanical
Casagrande Grooving tool		Metal
	X	Plastic

LL	PL	PI
42%	**23%**	**20%**





REVIEWED BY: Fella Damardji **DATE:** 6/29/2021

Material Finer than #200 Sieve

ASTM D-1140

Project Name: New Gym Piedmont MS

Date Received: 6/25/2021

Project #: 60-0938G

Sampled By: Ryan R

Sample Description: SPT Samples

Lab #: Fremont

Sample Location:	B2	Sample Location:	B2
Depth:	20 ft	Depth:	30 ft
Initial Dry Wt. + Tare (g):	220.4	Initial Dry Wt. + Tare (g):	139.4
Initial Tare:	86.7	Initial Tare:	86
Final Dry Wt. + Tare (g):	406.8	Final Dry Wt. + Tare (g):	387.3
Final Tare:	337.3	Final Tare:	342.3
Soil Loss (g):	64.2	Soil Loss (g):	8.4
Percent Finer than # 200 Sieve:	**48.0%**	Percent Finer than # 200 Sieve:	**15.7%**

Sample Location:	B2	Sample Location:	B2
Depth:	40 ft	Depth:	45 ft
Initial Dry Wt. + Tare (g):	378.6	Initial Dry Wt. + Tare (g):	432.7
Initial Tare:	87.7	Initial Tare:	87.2
Final Dry Wt. + Tare (g):	429.3	Final Dry Wt. + Tare (g):	536.1
Final Tare:	193.5	Final Tare:	348.1
Soil Loss (g):	55.1	Soil Loss (g):	157.5
Percent Finer than # 200 Sieve:	**18.9%**	Percent Finer than # 200 Sieve:	**45.6%**

Sample Location:	B2	Sample Location:	B2
Depth:	25 ft	Depth:	35 ft
Initial Dry Wt. + Tare (g):	721.6	Initial Dry Wt. + Tare (g):	565.6
Initial Tare:	342.1	Initial Tare:	202.5
Final Dry Wt. + Tare (g):	582.3	Final Dry Wt. + Tare (g):	380.2
Final Tare:	342.1	Final Tare:	202.5
Soil Loss (g):	139.3	Soil Loss (g):	185.4
Percent Finer than # 200 Sieve:	**36.7%**	Percent Finer than # 200 Sieve:	**51.1%**

Reviewed By: Fella Damardji
Laboratory Manager

Date: 7/1/2021

CTE CAL, INC. 46716 Fremont Blvd. | Fremont, CA 94538 | Ph: (510) 573-6362 | Fax: (510) 573-6684

Laboratory Determination of Moisture Content of Soil and Rock by Mass (ASTM D 2216) And Laboratory Test Method For Determination of Density of Soil Specimens (ASTM D7263)

Project Name: New Gym Piedmont MS Date: 6/25/2021

Project #: 60-0938G Sampled By: Ryan R

Moisture Content Method used: ■ A ☐ B
Drying Temperature: 225 °F

Sample Depth (ft): **B2- 20ft**

Volume (ft^3):	0.0051
Tare (g):	188.6
Moist Wt. + Tare (g):	442.1
Oven Dry Wt. + Tare (g):	423.8
Moisture Content:	**8%**
Wet Density (lb/ft^3)	**109.6**
Dry Density (lb/ft^3)	**101.7**

Moisture Content Method used: ■ A ☐ B
Drying Temperature: 225 °F

Sample Depth (ft): **B2-35ft**

Volume (ft^3):	0.0051
Tare (g):	200.5
Moist Wt. + Tare (g):	515.4
Oven Dry Wt. + Tare (g):	457.9
Moisture Content:	**22%**
Wet Density (lb/ft^3)	**136.1**
Dry Density (lb/ft^3)	**111.3**

Moisture Content Method used: ■ A ☐ B
Drying Temperature: 225 °F

Sample Depth (ft): **B2-40ft**

Volume (ft^3):	0.0051
Tare (g):	252.6
Moist Wt. + Tare (g):	526.8
Oven Dry Wt. + Tare (g):	508.9
Moisture Content:	**7%**
Wet Density (lb/ft^3)	**118.5**
Dry Density (lb/ft^3)	**110.8**

Laboratory Manager: Fella Damardji Date: 6/29/2021

Report of Aggregate Testing

CTE CAL, INC. 46716 Fremont Blvd. | Fremont, CA 94538 | Ph: (510) 573-6362 | Fax: (510) 573-6684

Project Name: New Gym Piedmont MS
Project No.: 60-0938G
Sample Location : B2- 45ft
Sampled By: Ryan R Date: 6/25/2021
Tested By : Fella Date: 6/28/2021

SM or SC

Sample Size: 20575 g	% Passing	Specifications	Tolerance	Notes
2.5"	**100**			
1.5"	**100**			
1 "	**100**			
3/4"	**100**			
1/2"	**100**			
3/8"	**98**			
#4	**97**			
#8	**95**			
#16	**93**			
#30	**91**			
#50	**87**			
#100	**65**			
#200	**46**			

All Sampling and Testing Done In Accordance With Applicable ASTM Standards

Reviewed By: Fella Damardji

Laboratory Technician

COOPER TESTING LABORATORY

Corrosivity Tests

CTL Job No:	1044-019	**Project No.:**	60-0938G	IC Ions to test for:	Both
Client:	CTE Cal	**Date:**	5/28/2021		
Project Name:	Piedmont Middle School	**By:**	JC		

Boring:						
Sample:	1					
Depth, ft:	2.0					
Soil Description:	Dark Grayish Brown Sandy CLAY w/ Gravel & organics					
EXTRACTION						
Extraction Flask No.						
Wt. of wet soil (g)	102.15					
Vol of water (ml)	300	300	300	300	300	300
% H_2O of Extracted Sample:						
Pan No.						
Pan wt. (g)	22.09					
Total wet wt. (g)	146.61					
Total dry wt (g)	134.53					
ORP / SULFIDE TESTS						
Beaker No.						
ORP, E_H (NHE) (Rmv)	232.4					
ORP Test Temp, ^OC	22.5					
Sulfide	Negative					
ASTM RESISTIVITY **- As Received**						
Small Dial Reading						
Large Dial Reading						
Temp. ^oC						
ASTM RESISTIVITY **- 100% Saturation**						
Bowl No.						
Small Dial Reading	1,000					
Large Dial Reading	1.97					
Temp. ^oC	22.3					
pH TEST						
pH measurement #1	6.85					
pH measurement #2	6.75					
pH measurement #3	6.74					
CHLORIDE AND SULFATE TESTING						
IC Ions to test for:	Both	Both	Both	Both	Both	Both
Vial No.						
CHLORIDE						
Meas. conc(mg **Cl^-**/L)	23.8					
SULFATE						
Meas. conc(mg **SO_4^{-2}**/L)	19.2					
Comments:						

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 1044-019 **Date:** 6/1/2021 **Tested By:** JC **Checked:** PJ

Client: CTE Cal **Project:** Piedmont Middle School **Proj. No:** 60-0938G

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test Temp °C	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
-	1	2.0	-	-	2,305	80	65	0.0065	6.8	232	23	Negative	10.7	Dark Grayish Brown Sandy CLAY w/ Gravel & organics

APPENDIX C: LIQUEFACTION ANALYSIS FIGURES AND CALCULATIONS

FIGURE C1

CPT NO. 4

PROJECT/CPT DATA

Project Title	Piedmont MS New Gymnasium
Project No.	1332-2-2
Project Manager	NSD

SEISMIC PARAMETERS

Controlling Fault	Calaveras	
Earthquake Magnitude (Mw)	7.18	
PGA (Amax)	0.97	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	24
Design Water Depth (feet)	50
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 50 FEET

0.23 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

0.00 (Inches)

TOTAL SEISMIC SETTLEMENT 0.2 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 L/H 40.0

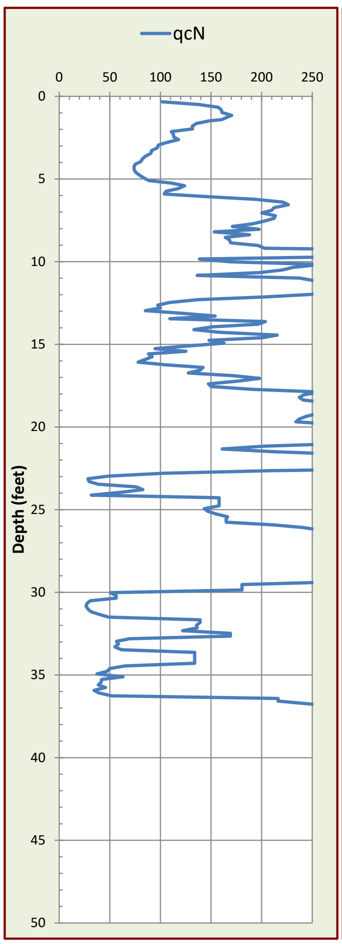
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

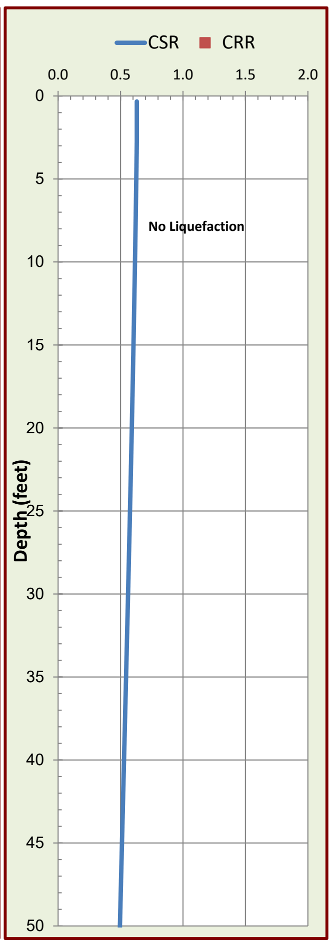
EXPECTED RANGE OF DISPLACEMENT

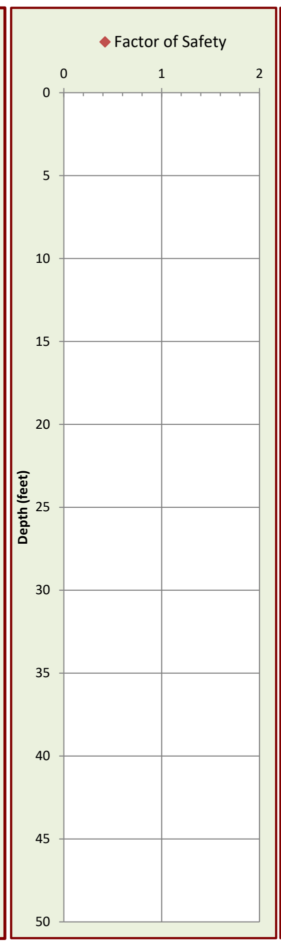
0.0 to 0.0 feet

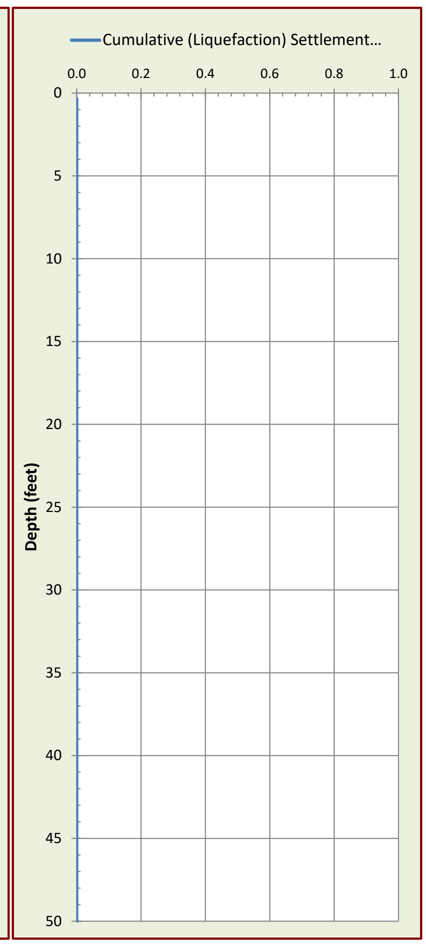
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

CPT No.	4

PGA (A_{max})	0.97

Total Settlement:	0.00	(Inches)

Depth (ft)	q_c (tsf)	f_s (tsf)	σ_{vc} (psf)	Insitu σ'_{vc} (psf)	Q	F (%)	I_c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q_{cN} near interfaces (soft layer)	Thin Layer Factor (K_H)	Interpreted q_{cN}	C_N	q_{c1N}	$q_{c1N\text{-}CS}$	Stress Reduction Coeff, r_d	CSR	K_σ for Sand	$CRR_{M=7.5, \sigma'vc = 1 atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε_v	Settlement (Inches)
0.330	107.600	0.800	39.6	39.6	743.288	0.744	1.24		Unsaturated	0.0			101.70	1.70	172.89	172.89	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.490	145.500	1.000	58.8	58.8	824.819	0.687	1.19		Unsaturated	0.0			137.52	1.70	233.79	233.79	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.660	166.200	1.300	79.2	79.2	811.778	0.782	1.25		Unsaturated	0.0			157.09	1.70	267.05	267.05	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.820	169.500	1.600	98.4	98.4	742.708	0.944	1.34		Unsaturated	0.0			160.21	1.70	272.35	272.35	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
0.980	170.200	1.500	117.6	117.6	682.147	0.882	1.33		Unsaturated	0.0			160.87	1.70	273.48	273.48	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.150	180.300	1.300	138.0	138.0	667.056	0.721	1.26		Unsaturated	0.0			170.42	1.70	289.71	289.71	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.410	169.600	1.300	169.2	169.2	566.606	0.767	1.32		Unsaturated	0.0			160.30	1.70	272.51	272.51	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.480	157.300	1.300	177.6	177.6	512.902	0.827	1.37		Unsaturated	0.0			148.68	1.70	252.75	252.75	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.640	143.400	1.800	196.8	196.8	444.131	1.256	1.55		Unsaturated	0.0			135.54	1.70	230.42	230.42	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.800	139.200	0.600	216.0	216.0	411.479	0.431	1.21		Unsaturated	0.0			131.57	1.70	223.67	223.67	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
1.970	139.300	0.700	236.4	236.4	393.578	0.503	1.27		Unsaturated	0.0			131.66	1.70	223.83	223.83	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.130	117.200	0.700	255.6	255.6	318.380	0.598	1.39		Unsaturated	0.0			110.78	1.70	188.32	188.32	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.300	119.300	0.900	276.0	276.0	311.857	0.755	1.47		Unsaturated	0.0			112.76	1.70	191.69	191.69	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.460	119.800	1.200	295.2	295.2	302.786	1.003	1.57		Unsaturated	0.0			113.23	1.70	192.50	192.50	1.00	0.631	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.620	124.700	1.400	314.4	314.4	305.386	1.124	1.61		Unsaturated	0.0			117.86	1.70	200.37	200.37	1.00	0.630	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.790	112.400	1.300	334.8	334.8	266.685	1.158	1.65		Unsaturated	0.0			106.24	1.70	180.60	180.60	1.00	0.630	1.100	n.a.	n.a.	n.a.	0.00	0.00
2.950	103.600	1.300	354.0	354.0	238.994	1.257	1.71		Unsaturated	0.0			97.92	1.70	166.47	166.47	1.00	0.630	1.100	n.a.	n.a.	n.a.	0.00	0.00
3.120	102.600	1.300	374.4	374.4	230.122	1.269	1.73		Unsaturated	1.1			96.98	1.70	164.86	164.86	1.00	0.630	1.100	n.a.	n.a.	n.a.	0.00	0.00
3.280	96.600	1.300	393.6	393.6	211.269	1.349	1.77		Unsaturated	4.6			91.30	1.70	155.22	155.31	1.00	0.629	1.100	n.a.	n.a.	n.a.	0.00	0.00
3.450	95.800	1.300	414.0	414.0	204.267	1.360	1.78		Unsaturated	5.6			90.55	1.70	153.93	154.38	1.00	0.629	1.100	n.a.	n.a.	n.a.	0.00	0.00
3.610	90.500	1.500	433.2	433.2	188.597	1.661	1.87		Unsaturated	12.7			85.54	1.70	145.42	163.86	1.00	0.629	1.100	n.a.	n.a.	n.a.	0.00	0.00
3.770	87.100	2.000	452.4	452.4	177.582	2.302	2.00		Unsaturated	22.9			82.33	1.70	139.86	189.67	1.00	0.628	1.100	n.a.	n.a.	n.a.	0.00	0.00
3.940	85.500	2.500	472.8	472.8	170.489	2.932	2.09		Unsaturated	30.4			80.81	1.65	133.39	196.36	1.00	0.628	1.100	n.a.	n.a.	n.a.	0.00	0.00
4.100	79.600	3.000	492.0	492.0	155.546	3.781	2.21		Unsaturated	39.5			75.24	1.64	123.06	194.20	1.00	0.628	1.100	n.a.	n.a.	n.a.	0.00	0.00
4.270	78.100	3.600	512.4	512.4	149.517	4.625	2.29		Unsaturated	46.0			73.82	1.61	119.09	194.23	1.00	0.627	1.100	n.a.	n.a.	n.a.	0.00	0.00
4.430	78.300	3.800	531.6	531.6	147.151	4.870	2.31		Unsaturated	47.8			74.01	1.59	117.99	193.96	0.99	0.627	1.100	n.a.	n.a.	n.a.	0.00	0.00
4.590	79.900	4.900	550.8	550.8	147.510	6.154	2.39		Unsaturated	54.5			75.52	1.56	118.16	197.71	0.99	0.627	1.100	n.a.	n.a.	n.a.	0.00	0.00
4.760	83.700	4.800	571.2	571.2	151.747	5.754	2.36		Unsaturated	52.0			79.11	1.54	121.61	200.89	0.99	0.627	1.100	n.a.	n.a.	n.a.	0.00	0.00
4.920	88.100	5.700	590.4	590.4	157.115	6.492	2.40		Unsaturated	54.9			83.27	1.51	125.33	207.02	0.99	0.626	1.100	n.a.	n.a.	n.a.	0.00	0.00
5.090	93.300	5.800	610.8	610.8	163.599	6.237	2.37		Unsaturated	53.0			88.19	1.48	130.19	212.27	0.99	0.626	1.100	n.a.	n.a.	n.a.	0.00	0.00
5.250	117.400	5.500	630.0	630.0	202.816	4.697	2.22		Unsaturated	40.7			110.96	1.41	156.46	236.76	0.99	0.626	1.100	n.a.	n.a.	n.a.	0.00	0.00
5.410	131.100	4.400	649.2	649.2	223.156	3.365	2.08		Unsaturated	29.1			123.91	1.39	172.26	240.83	0.99	0.625	1.100	n.a.	n.a.	n.a.	0.00	0.00
5.580	124.500	2.700	669.6	669.6	208.624	2.175	1.94		Unsaturated	17.9			117.67	1.44	169.57	209.15	0.99	0.625	1.100	n.a.	n.a.	n.a.	0.00	0.00
5.740	111.800	2.700	688.8	688.8	184.641	2.422	2.01		Unsaturated	23.4			105.67	1.44	151.83	204.97	0.99	0.625	1.100	n.a.	n.a.	n.a.	0.00	0.00
5.910	109.800	4.700	709.2	709.2	178.684	4.294	2.22		Unsaturated	40.4			103.78	1.39	144.37	221.49	0.99	0.624	1.100	n.a.	n.a.	n.a.	0.00	0.00
6.070	154.900	5.800	728.4	728.4	248.952	3.753	2.09		Unsaturated	30.3			146.41	1.32	193.98	269.14	0.99	0.624	1.100	n.a.	n.a.	n.a.	0.00	0.00
6.230	205.400	7.200	747.6	747.6	326.022	3.512	2.01		Unsaturated	23.6			194.14	1.32	255.46	326.13	0.99	0.624	1.100	n.a.	n.a.	n.a.	0.00	0.00
6.400	233.900	7.700	768.0	768.0	366.360	3.297	1.96		Unsaturated	19.8			221.08	1.31	288.85	350.60	0.99	0.623	1.100	n.a.	n.a.	n.a.	0.00	0.00
6.560	239.500	8.000	787.2	787.2	370.528	3.346	1.96		Unsaturated	20.1			226.37	1.30	293.84	357.39	0.99	0.623	1.100	n.a.	n.a.	n.a.	0.00	0.00
6.730	224.600	8.700	807.6	807.6	343.006	3.881	2.04		Unsaturated	25.9			212.29	1.29	273.71	354.17	0.99	0.623	1.100	n.a.	n.a.	n.a.	0.00	0.00
6.890	222.100	8.000	826.8	826.8	335.206	3.609	2.01		Unsaturated	24.0			209.92	1.28	268.99	343.08	0.99	0.622	1.100	n.a.	n.a.	n.a.	0.00	0.00
7.050	211.800	8.600	846.0	846.0	315.969	4.069	2.07		Unsaturated	28.7			200.19	1.27	254.97	339.01	0.99	0.622	1.100	n.a.	n.a.	n.a.	0.00	0.00
7.220	225.900	8.200	866.4	866.4	333.039	3.637	2.02		Unsaturated	24.4			213.52	1.27	270.23	345.60	0.99	0.622	1.100	n.a.	n.a.	n.a.	0.00	0.00
7.380	224.700	7.500	885.6	885.6	327.642	3.344	1.99		Unsaturated	22.1			212.38	1.26	267.25	334.59	0.99	0.621	1.100	n.a.	n.a.	n.a.	0.00	0.00
7.550	214.100	6.700	906.0	906.0	308.606	3.136	1.98		Unsaturated	21.1			202.36	1.25	253.12	315.04	0.98	0.621	1.100	n.a.	n.a.	n.a.	0.00	0.00
7.710	202.100	7.100	925.2	925.2	288.221	3.521	2.04		Unsaturated	25.8			191.02	1.24	237.61	311.38	0.98	0.621	1.100	n.a.	n.a.	n.a.	0.00	0.00
7.870	181.000	6.600	944.4	944.4	255.410	3.656	2.08		Unsaturated	29.1			171.08	1.24	211.65	288.01	0.98	0.620	1.100	n.a.	n.a.	n.a.	0.00	0.00
8.040	209.000	7.400	964.8	964.8	291.875	3.549	2.04		Unsaturated	25.8			197.54	1.23	243.02	317.82	0.98	0.620	1.100	n.a.	n.a.	n.a.	0.00	0.00
8.200	162.100	5.900	984.0	984.0	223.995	3.651	2.10		Unsaturated	31.4			153.21	1.22	187.51	263.23	0.98	0.620	1.100	n.a.	n.a.	n.a.	0.00	0.00
8.370	199.000	6.000	1004.4	1004.4	272.317	3.023	1.99		Unsaturated	22.2			188.09	1.22	228.95	290.99	0.98	0.619	1.100	n.a.	n.a.	n.a.	0.00	0.00
8.530	173.600	4.700	1023.6	1023.6	235.220	2.715	1.99		Unsaturated	21.8			164.08	1.21	198.73	254.90	0.98	0.619	1.100	n.a.	n.a.	n.a.	0.00	0.00
8.690	178.100	5.000	1042.8	1042.8	239.090	2.816	1.99		Unsaturated	22.6			168.34	1.21	202.89	261.88	0.98	0.619	1.100	n.a.	n.a.	n.a.	0.00	0.00
8.860	179.200	4.300	1063.2	1063.2	238.239	2.407	1.94		Unsaturated	18.1			169.38	1.20	204.00	248.31	0.98	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
9.020	208.000	4.300	1082.4	1082.4	274.164	2.073	1.85		Unsaturated	11.1			196.60	1.20	235.12	251.22	0.98	0.618	1.100	n.a.	n.a.	n.a.	0.00	0.00
9.190	214.800	7.200	1102.8	1102.8	280.506	3.361	2.02		Unsaturated	24.9			203.02	1.19	241.11	312.98	0.98	0.617	1.100	n.a.	n.a.	n.a.	0.00	0.00
9.350	442.500	8.000	1122.0	1122.0	573.638	1.810	1.64		Unsaturated	0.0			418.24	1.18	494.44	494.44	0.98	0.617	1.100	n.a.	n.a.	n.a.	0.00	0.00
9.510	435.700	8.300	1141.2	1141.2	560.028	1.907	1.67		Unsaturated	0.0			411.81	1.18	484.67	484.67	0.98	0.617	1.100	n.a.	n.a.	n.a.	0.00	0.00
9.680	322.400	6.400	1161.6	1161.6	410.540	1.989	1.74		Unsaturated	2.5			304.73	1.17	356.96	356.96	0.98	0.616	1.100	n.a.	n.a.	n.a.	0.00	0.00
9.840	146.600	5.000	1180.8	1180.8	184.742	3.424	2.13		Unsaturated	33.2			138.56	1.18	163.48	236.84	0.98	0.616	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.010	178.800	5.400	1201.2	1201.2	223.548	3.030	2.04		Unsaturated	26.0			169.00	1.16	196.23	262.99	0.98	0.616	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.170	274.000	6.100	1220.4	1220.4	340.254	2.231	1.83		Unsaturated	9.2			258.98	1.16	299.45	309.27	0.98	0.615	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.340	244.600	6.900	1240.8	1240.8	301.144	2.828	1.94		Unsaturated	18.5			231.19	1.15	266.15	319.29	0.98	0.615	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.500	233.300	7.600	1260.0	1260.0	284.988	3.266	2.01		Unsaturated	23.7			220.51	1.15	252.83	323.38	0.97	0.615	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.660	209.700	5.800	1279.2	1279.2	254.141	2.774	1.97		Unsaturated	21.0			198.20	1.14	226.35	283.89	0.97	0.614	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.830	144.200	5.200	1299.6	1299.6	173.130	3.622	2.16		Unsaturated	36.1			136.29	1.15	156.89	232.49	0.97	0.614	1.100	n.a.	n.a.	n.a.	0.00	0.00
10.990	251.200	5.600	1318.8	1318.8	299.958	2.235	1.86		Unsaturated	11.6			237.43	1.13	268.97	288.80	0.97	0.613	1.100	n.a.	n.a.	n.a.	0.00	0.00
11.160	267.900	5.100	1339.2	1339.2	317.494	1.908	1.79		Unsaturated	5.9			253.21	1.13	285.69	286.57	0.97	0.613	1.100	n.a.	n.a.	n.a.	0.00	0.00

CORNERSTONE EARTH GROUP

CPT No.	4

PGA (A_{max})	0.97

Total Settlement:	0.00	(Inches)

Depth (ft)	q_c (tsf)	f_s (tsf)	σ_{vc} (psf)	Insitu σ'_{vc} (psf)	Q	F (%)	I_c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q_{cN} near interfaces (soft layer)	Thin Layer Factor (K_H)	Interpreted q_{cN}	C_N	q_{c1N}	$q_{c1N\text{-}CS}$	Stress Reduction Coeff, r_d	CSR	K_σ for Sand	$CRR_{M=7.5,\ \sigma'vc = 1\ atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε_v	Settlement (Inches)
11.320	269.000	4.900	1358.4	1358.4	316.528	1.826	1.77		Unsaturated	4.6			254.25	1.12	285.79	285.93	0.97	0.613	1.100	n.a.	n.a.	n.a.	0.00	0.00
11.480	320.900	6.100	1377.6	1377.6	375.100	1.905	1.75		Unsaturated	2.8			303.31	1.12	339.67	339.67	0.97	0.612	1.100	n.a.	n.a.	n.a.	0.00	0.00
11.810	282.000	6.000	1417.2	1417.2	324.873	2.133	1.82		Unsaturated	8.7			266.54	1.11	296.27	304.04	0.97	0.611	1.100	n.a.	n.a.	n.a.	0.00	0.00
11.980	263.700	6.000	1437.6	1437.6	301.563	2.282	1.86		Unsaturated	12.1			249.24	1.11	276.00	298.72	0.97	0.611	1.100	n.a.	n.a.	n.a.	0.00	0.00
12.140	213.000	5.600	1456.8	1456.8	241.804	2.638	1.97		Unsaturated	20.5			201.32	1.10	222.16	277.31	0.97	0.611	1.100	n.a.	n.a.	n.a.	0.00	0.00
12.300	145.100	4.000	1476.0	1476.0	163.374	2.771	2.08		Unsaturated	29.7			137.15	1.12	153.08	218.97	0.97	0.610	1.100	n.a.	n.a.	n.a.	0.00	0.00
12.470	115.300	4.300	1496.4	1496.4	128.751	3.754	2.25		Unsaturated	43.1			108.98	1.12	122.37	196.35	0.97	0.610	1.087	n.a.	n.a.	n.a.	0.00	0.00
12.630	103.000	4.600	1515.6	1515.6	114.185	4.499	2.35		Unsaturated	50.7			97.35	1.12	109.43	184.77	0.97	0.610	1.074	n.a.	n.a.	n.a.	0.00	0.00
12.800	105.800	3.500	1536.0	1536.0	116.519	3.332	2.24		Unsaturated	42.0			100.00	1.12	111.99	182.55	0.97	0.609	1.070	n.a.	n.a.	n.a.	0.00	0.00
12.960	90.100	3.800	1555.2	1555.2	98.478	4.254	2.37		Unsaturated	52.3			85.16	1.12	95.58	168.01	0.97	0.609	1.058	n.a.	n.a.	n.a.	0.00	0.00
13.120	128.000	3.800	1574.4	1574.4	139.394	2.987	2.15		Unsaturated	35.2			120.98	1.10	133.24	202.39	0.96	0.608	1.080	n.a.	n.a.	n.a.	0.00	0.00
13.290	163.000	4.400	1594.8	1594.8	176.594	2.713	2.06		Unsaturated	27.5			154.06	1.09	167.20	231.88	0.96	0.608	1.085	n.a.	n.a.	n.a.	0.00	0.00
13.450	115.600	5.000	1614.0	1614.0	124.233	4.356	2.31		Unsaturated	48.0			109.26	1.09	119.63	196.18	0.96	0.608	1.068	n.a.	n.a.	n.a.	0.00	0.00
13.620	215.600	6.000	1634.4	1634.4	230.990	2.794	2.00		Unsaturated	23.0			203.78	1.07	218.15	280.81	0.96	0.607	1.077	n.a.	n.a.	n.a.	0.00	0.00
13.780	207.900	6.800	1653.6	1653.6	221.402	3.284	2.07		Unsaturated	28.5			196.50	1.07	209.71	284.56	0.96	0.607	1.074	n.a.	n.a.	n.a.	0.00	0.00
13.940	160.100	6.000	1672.8	1672.8	169.304	3.767	2.18		Unsaturated	37.7			151.32	1.07	161.60	240.09	0.96	0.606	1.071	n.a.	n.a.	n.a.	0.00	0.00
14.110	140.800	6.800	1693.2	1693.2	147.877	4.859	2.31		Unsaturated	47.6			133.08	1.07	142.23	224.42	0.96	0.606	1.067	n.a.	n.a.	n.a.	0.00	0.00
14.270	167.200	8.000	1712.4	1712.4	174.774	4.809	2.26		Unsaturated	44.1			158.03	1.06	167.14	253.13	0.96	0.605	1.063	n.a.	n.a.	n.a.	0.00	0.00
14.440	227.800	7.500	1732.8	1732.8	237.026	3.305	2.06		Unsaturated	27.4			215.31	1.05	226.97	302.74	0.96	0.605	1.060	n.a.	n.a.	n.a.	0.00	0.00
14.600	212.100	6.300	1752.0	1752.0	219.406	2.983	2.04		Unsaturated	25.9			200.47	1.05	210.71	279.84	0.96	0.605	1.057	n.a.	n.a.	n.a.	0.00	0.00
14.760	156.600	4.700	1771.2	1771.2	160.867	3.018	2.12		Unsaturated	32.4			148.02	1.05	156.01	226.66	0.96	0.604	1.053	n.a.	n.a.	n.a.	0.00	0.00
14.930	172.300	3.300	1791.6	1791.6	176.065	1.925	1.94		Unsaturated	18.2			162.85	1.05	171.58	212.32	0.96	0.604	1.050	n.a.	n.a.	n.a.	0.00	0.00
15.090	136.400	3.000	1810.8	1810.8	138.439	2.214	2.05		Unsaturated	27.3			128.92	1.05	135.88	194.12	0.96	0.603	1.038	n.a.	n.a.	n.a.	0.00	0.00
15.260	100.200	3.200	1831.2	1831.2	100.875	3.223	2.27		Unsaturated	44.3			94.71	1.06	99.96	169.13	0.96	0.603	1.028	n.a.	n.a.	n.a.	0.00	0.00
15.420	132.500	2.900	1850.4	1850.4	132.988	2.204	2.06		Unsaturated	28.0			125.24	1.05	131.14	189.86	0.96	0.603	1.031	n.a.	n.a.	n.a.	0.00	0.00
15.580	93.200	2.800	1869.6	1869.6	92.776	3.035	2.27		Unsaturated	44.6			88.09	1.05	92.42	159.89	0.96	0.602	1.022	n.a.	n.a.	n.a.	0.00	0.00
15.750	97.200	2.200	1890.0	1890.0	96.264	2.286	2.17		Unsaturated	36.5			91.87	1.05	96.01	158.07	0.95	0.602	1.019	n.a.	n.a.	n.a.	0.00	0.00
15.910	90.200	1.700	1909.2	1909.2	88.804	1.905	2.14		Unsaturated	33.9			85.26	1.04	88.91	146.82	0.95	0.601	1.016	n.a.	n.a.	n.a.	0.00	0.00
16.080	82.500	1.900	1929.6	1929.6	80.702	2.330	2.23		Unsaturated	41.2			77.98	1.04	81.01	143.38	0.95	0.601	1.014	n.a.	n.a.	n.a.	0.00	0.00
16.240	111.200	2.600	1948.8	1948.8	108.560	2.359	2.14		Unsaturated	34.5			105.10	1.03	108.36	171.15	0.95	0.600	1.016	n.a.	n.a.	n.a.	0.00	0.00
16.400	150.400	2.800	1968.0	1968.0	146.439	1.874	1.98		Unsaturated	21.6			142.16	1.02	145.69	193.21	0.95	0.600	1.018	n.a.	n.a.	n.a.	0.00	0.00
16.570	147.000	2.700	1988.4	1988.4	142.361	1.849	1.99		Unsaturated	21.9			138.94	1.02	141.94	189.68	0.95	0.600	1.015	n.a.	n.a.	n.a.	0.00	0.00
16.730	134.900	3.200	2007.6	2007.6	129.928	2.390	2.10		Unsaturated	30.7			127.50	1.02	129.80	192.44	0.95	0.599	1.013	n.a.	n.a.	n.a.	0.00	0.00
16.900	182.700	3.400	2028.0	2028.0	175.412	1.871	1.93		Unsaturated	17.5			172.68	1.01	174.99	213.73	0.95	0.599	1.013	n.a.	n.a.	n.a.	0.00	0.00
17.060	209.500	3.300	2047.2	2047.2	200.331	1.583	1.84		Unsaturated	10.1			198.02	1.01	200.09	210.87	0.95	0.598	1.010	n.a.	n.a.	n.a.	0.00	0.00
17.230	185.600	3.100	2067.6	2067.6	176.478	1.680	1.89		Unsaturated	14.5			175.43	1.01	176.75	204.17	0.95	0.598	1.006	n.a.	n.a.	n.a.	0.00	0.00
17.390	156.000	1.900	2086.8	2086.8	147.483	1.226	1.85		Unsaturated	10.6			147.45	1.01	148.25	159.39	0.95	0.597	1.002	n.a.	n.a.	n.a.	0.00	0.00
17.550	158.800	3.500	2106.0	2106.0	149.453	2.219	2.03		Unsaturated	25.6			150.09	1.00	150.32	208.00	0.95	0.597	1.001	n.a.	n.a.	n.a.	0.00	0.00
17.720	201.400	6.000	2126.4	2126.4	188.891	2.995	2.07		Unsaturated	28.9			190.36	1.00	190.11	261.99	0.95	0.596	0.999	n.a.	n.a.	n.a.	0.00	0.00
17.880	272.900	7.200	2145.6	2145.6	255.147	2.649	1.96		Unsaturated	19.6			257.94	1.00	257.00	313.51	0.95	0.596	0.996	n.a.	n.a.	n.a.	0.00	0.00
18.050	255.900	7.600	2166.0	2166.0	238.052	2.983	2.02		Unsaturated	24.3			241.87	0.99	240.39	310.59	0.94	0.596	0.993	n.a.	n.a.	n.a.	0.00	0.00
18.210	250.900	8.200	2185.2	2185.2	232.344	3.283	2.06		Unsaturated	27.6			237.15	0.99	235.14	312.86	0.94	0.595	0.990	n.a.	n.a.	n.a.	0.00	0.00
18.370	255.100	7.100	2204.4	2204.4	235.211	2.795	2.00		Unsaturated	22.7			241.12	0.99	238.53	303.42	0.94	0.595	0.988	n.a.	n.a.	n.a.	0.00	0.00
18.540	282.700	7.100	2224.8	2224.8	259.561	2.521	1.94		Unsaturated	17.8			267.20	0.99	263.69	313.67	0.94	0.594	0.985	n.a.	n.a.	n.a.	0.00	0.00
18.700	332.000	7.200	2244.0	2244.0	303.689	2.176	1.84		Unsaturated	10.5			313.80	0.98	308.97	325.25	0.94	0.594	0.982	n.a.	n.a.	n.a.	0.00	0.00
18.870	302.800	5.800	2264.4	2264.4	275.629	1.923	1.82		Unsaturated	8.8			286.20	0.98	281.13	288.99	0.94	0.593	0.980	n.a.	n.a.	n.a.	0.00	0.00
19.030	280.000	3.500	2283.6	2283.6	253.715	1.255	1.70		Unsaturated	0.0			264.65	0.98	259.38	259.38	0.94	0.593	0.977	n.a.	n.a.	n.a.	0.00	0.00
19.190	270.300	5.200	2302.8	2302.8	243.858	1.932	1.85		Unsaturated	11.4			255.48	0.98	249.84	268.00	0.94	0.592	0.975	n.a.	n.a.	n.a.	0.00	0.00
19.360	258.100	6.200	2323.2	2323.2	231.770	2.413	1.95		Unsaturated	18.7			243.95	0.98	238.01	288.79	0.94	0.592	0.972	n.a.	n.a.	n.a.	0.00	0.00
19.520	250.900	5.800	2342.4	2342.4	224.342	2.323	1.94		Unsaturated	18.3			237.15	0.97	230.87	279.03	0.94	0.592	0.970	n.a.	n.a.	n.a.	0.00	0.00
19.690	247.500	6.800	2362.8	2362.8	220.321	2.761	2.01		Unsaturated	23.6			233.93	0.97	227.22	293.04	0.94	0.591	0.967	n.a.	n.a.	n.a.	0.00	0.00
19.850	291.800	8.200	2382.0	2382.0	258.887	2.822	1.98		Unsaturated	21.1			275.80	0.97	267.32	331.31	0.94	0.591	0.964	n.a.	n.a.	n.a.	0.00	0.00
20.010	370.200	8.400	2401.2	2401.2	327.404	2.276	1.84		Unsaturated	10.5			349.91	0.97	338.43	355.46	0.94	0.590	0.962	n.a.	n.a.	n.a.	0.00	0.00
20.180	357.600	4.900	2421.6	2421.6	314.880	1.375	1.67		Unsaturated	0.0			338.00	0.97	326.18	326.18	0.94	0.590	0.960	n.a.	n.a.	n.a.	0.00	0.00
20.340	366.400	4.400	2440.8	2440.8	321.375	1.205	1.62		Unsaturated	0.0			346.31	0.96	333.51	333.51	0.93	0.589	0.957	n.a.	n.a.	n.a.	0.00	0.00
20.510	460.300	4.600	2461.2	2461.2	402.325	1.002	1.50		Unsaturated	0.0			435.07	0.96	418.06	418.06	0.93	0.589	0.955	n.a.	n.a.	n.a.	0.00	0.00
20.670	403.000	3.400	2480.4	2480.4	350.734	0.846	1.47		Unsaturated	0.0			380.91	0.96	365.27	365.27	0.93	0.588	0.952	n.a.	n.a.	n.a.	0.00	0.00
20.830	368.600	2.800	2499.6	2499.6	319.461	0.762	1.47		Unsaturated	0.0			348.39	0.96	333.41	333.41	0.93	0.588	0.950	n.a.	n.a.	n.a.	0.00	0.00
21.000	299.200	1.800	2520.0	2520.0	258.048	0.604	1.46		Unsaturated	0.0			282.80	0.95	270.06	270.06	0.93	0.587	0.948	n.a.	n.a.	n.a.	0.00	0.00
21.160	214.200	2.300	2539.2	2539.2	183.722	1.080	1.74		Unsaturated	2.1			202.46	0.94	190.19	190.19	0.93	0.587	0.957	n.a.	n.a.	n.a.	0.00	0.00
21.330	170.400	2.400	2559.6	2559.6	145.339	1.419	1.90		Unsaturated	14.6			161.06	0.93	150.32	176.23	0.93	0.586	0.961	n.a.	n.a.	n.a.	0.00	0.00
21.490	222.600	3.400	2578.8	2578.8	189.481	1.536	1.84		Unsaturated	10.5			210.40	0.94	197.63	210.03	0.93	0.586	0.941	n.a.	n.a.	n.a.	0.00	0.00
21.650	297.500	5.700	2598.0	2598.0	252.662	1.924	1.84		Unsaturated	10.6			281.19	0.95	266.37	281.29	0.93	0.586	0.938	n.a.	n.a.	n.a.	0.00	0.00
21.820	295.500	6.600	2618.4	2618.4	249.967	2.243	1.90		Unsaturated	15.2			279.30	0.95	264.04	301.65	0.93	0.585	0.936	n.a.	n.a.	n.a.	0.00	0.00
21.980	512.400	8.200	2637.6	2637.6	432.671	1.604	1.65		Unsaturated	0.0			484.31	0.94	456.96	456.96	0.93	0.585	0.934	n.a.	n.a.	n.a.	0.00	0.00
22.150	389.600	5.400	2658.0	2658.0	327.438	1.391	1.66		Unsaturated	0.0			368.24	0.94	346.74	346.74	0.93	0.584	0.932	n.a.	n.a.	n.a.	0.00	0.00
22.310	347.700	3.900	2677.2	2677.2	291.046	1.126	1.62		Unsaturated	0.0			328.64	0.94	308.86	308.86	0.93	0.584	0.929	n.a.	n.a.	n.a.	0.00	0.00

CPT No.	4

PGA (A_{max})	0.97

Total Settlement:	0.00	(Inches)

Depth (ft)	q_c (tsf)	f_s (tsf)	σ_{vc} (psf)	Insitu σ'_{vc} (psf)	Q	F (%)	I_c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q_{cN} near interfaces (soft layer)	Thin Layer Factor (K_H)	Interpreted q_{cN}	C_N	q_{c1N}	$q_{c1N\text{-}CS}$	Stress Reduction Coeff, r_d	CSR	K_σ for Sand	$CRR_{M=7.5, \sigma'vc = 1 atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε_v	Settlement (Inches)
22.470	436.900	5.400	2696.4	2696.4	364.687	1.240	1.60		Unsaturated	0.0			412.95	0.94	387.37	387.37	0.92	0.583	0.927	n.a.	n.a.	n.a.	0.00	0.00
22.640	222.700	4.500	2716.8	2716.8	184.632	2.033	1.95		Unsaturated	18.6			210.49	0.93	196.37	241.65	0.92	0.583	0.925	n.a.	n.a.	n.a.	0.00	0.00
22.800	108.600	1.600	2736.0	2736.0	89.133	1.492	2.06		Unsaturated	28.0			102.65	0.90	92.28	143.43	0.92	0.582	0.961	n.a.	n.a.	n.a.	0.00	0.00
22.970	52.200	1.400	2756.4	2756.4	42.087	2.755	2.48		Unsaturated	61.6			49.34	0.88	43.37	104.68	0.92	0.582	0.971	n.a.	n.a.	n.a.	0.00	0.00
23.130	30.100	0.800	2775.6	2775.6	20.689	2.786	2.72		Unsaturated	80.8			28.45	0.86	24.60	84.10	0.92	0.581	0.975	n.a.	n.a.	n.a.	0.00	0.00
23.300	31.200	1.100	2796.0	2796.0	21.318	3.691	2.79		Unsaturated	86.1			29.49	0.86	25.43	85.85	0.92	0.581	0.973	n.a.	n.a.	n.a.	0.00	0.00
23.460	40.700	1.200	2815.2	2815.2	27.914	3.054	2.65		Unsaturated	74.7			38.47	0.86	33.25	94.40	0.92	0.580	0.971	n.a.	n.a.	n.a.	0.00	0.00
23.620	80.500	1.700	2834.4	2834.4	64.584	2.150	2.27		Unsaturated	44.8			76.09	0.88	66.87	127.97	0.92	0.580	0.961	n.a.	n.a.	n.a.	0.00	0.00
23.790	87.500	1.900	2854.8	2854.8	70.041	2.207	2.26		Unsaturated	43.4			82.70	0.88	72.71	134.45	0.92	0.579	0.958	n.a.	n.a.	n.a.	0.00	0.00
23.950	64.400	2.100	2874.0	2874.0	51.064	3.335	2.48		Unsaturated	61.3			60.87	0.87	52.82	116.71	0.92	0.579	0.963	n.a.	n.a.	n.a.	0.00	0.00
24.120	33.500	2.100	2895.0	2887.5	22.201	6.552	2.94		Unsaturated	98.4			31.66	0.85	26.91	89.05	0.92	0.578	0.970	n.a.	n.a.	n.a.	0.00	0.00
24.280	70.900	2.100	2915.0	2897.5	56.090	3.024	2.42		Unsaturated	56.6	157.94		157.94	0.91	144.21	231.86	0.92	0.578	0.906	n.a.	n.a.	n.a.	0.00	0.00
24.440	124.500	2.300	2935.0	2907.5	99.204	1.869	2.10		Unsaturated	30.7	157.94		157.94	0.90	142.72	208.08	0.92	0.577	0.908	n.a.	n.a.	n.a.	0.00	0.00
24.610	153.000	2.400	2956.3	2918.2	121.953	1.584	1.98		Unsaturated	21.6	157.94		157.94	0.89	141.34	188.26	0.91	0.577	0.926	n.a.	n.a.	n.a.	0.00	0.00
24.770	167.100	2.600	2976.3	2928.2	133.065	1.570	1.95		Unsaturated	19.3			157.94	0.89	140.71	181.15	0.91	0.576	0.930	n.a.	n.a.	n.a.	0.00	0.00
24.940	151.600	2.000	2997.5	2938.8	120.384	1.332	1.93		Unsaturated	17.7			143.29	0.88	126.18	160.08	0.91	0.576	0.943	n.a.	n.a.	n.a.	0.00	0.00
25.100	155.400	2.400	3017.5	2948.9	123.214	1.560	1.97		Unsaturated	21.0			146.88	0.89	130.06	173.70	0.91	0.575	0.934	n.a.	n.a.	n.a.	0.00	0.00
25.260	163.900	3.000	3037.5	2958.9	129.791	1.848	2.01		Unsaturated	24.0			154.91	0.89	138.10	190.25	0.91	0.575	0.921	n.a.	n.a.	n.a.	0.00	0.00
25.430	176.000	3.100	3058.8	2969.5	139.204	1.777	1.98		Unsaturated	21.4			166.35	0.89	148.50	195.87	0.91	0.574	0.915	n.a.	n.a.	n.a.	0.00	0.00
25.590	174.400	2.800	3078.8	2979.5	137.687	1.620	1.95		Unsaturated	19.3			164.84	0.89	146.40	187.51	0.91	0.574	0.922	n.a.	n.a.	n.a.	0.00	0.00
25.760	174.600	2.400	3100.0	2990.2	137.593	1.387	1.90		Unsaturated	15.4			165.03	0.88	145.38	173.59	0.91	0.573	0.931	n.a.	n.a.	n.a.	0.00	0.00
25.920	223.200	2.100	3120.0	3000.2	175.932	0.947	1.71		Unsaturated	0.0			210.96	0.89	186.86	186.86	0.91	0.573	0.921	n.a.	n.a.	n.a.	0.00	0.00
26.080	254.700	2.100	3140.0	3010.2	200.594	0.830	1.63		Unsaturated	0.0			240.74	0.90	215.90	215.90	0.91	0.572	0.894	n.a.	n.a.	n.a.	0.00	0.00
26.250	273.700	2.700	3161.3	3020.9	215.262	0.992	1.67		Unsaturated	0.0			258.70	0.90	233.54	233.54	0.91	0.572	0.893	n.a.	n.a.	n.a.	0.00	0.00
26.410	278.400	2.900	3181.3	3030.9	218.610	1.048	1.68		Unsaturated	0.0			263.14	0.90	237.75	237.75	0.91	0.571	0.892	n.a.	n.a.	n.a.	0.00	0.00
26.580	267.200	6.600	3202.5	3041.5	209.389	2.485	1.98		Unsaturated	21.6			252.55	0.91	229.50	289.48	0.91	0.571	0.891	n.a.	n.a.	n.a.	0.00	0.00
26.740	295.900	5.900	3222.5	3051.5	231.626	2.005	1.88		Unsaturated	13.5			279.68	0.91	253.93	282.50	0.90	0.570	0.890	n.a.	n.a.	n.a.	0.00	0.00
26.900	300.700	3.600	3242.5	3061.5	235.011	1.204	1.70		Unsaturated	0.0			284.22	0.91	257.82	257.82	0.90	0.570	0.889	n.a.	n.a.	n.a.	0.00	0.00
27.070	341.600	4.400	3263.8	3072.2	266.678	1.294	1.69		Unsaturated	0.0			322.87	0.91	292.62	292.62	0.90	0.569	0.888	n.a.	n.a.	n.a.	0.00	0.00
27.230	313.400	4.600	3283.8	3082.2	244.152	1.476	1.76		Unsaturated	3.9			296.22	0.91	268.24	268.26	0.90	0.569	0.887	n.a.	n.a.	n.a.	0.00	0.00
27.400	324.100	3.900	3305.0	3092.8	252.088	1.209	1.68		Unsaturated	0.0			306.33	0.90	277.14	277.14	0.90	0.568	0.886	n.a.	n.a.	n.a.	0.00	0.00
27.560	329.300	4.400	3325.0	3102.9	255.732	1.343	1.72		Unsaturated	0.3			311.25	0.90	281.35	281.35	0.90	0.568	0.885	n.a.	n.a.	n.a.	0.00	0.00
27.720	368.600	4.200	3345.0	3112.9	285.938	1.145	1.63		Unsaturated	0.0			348.39	0.90	314.66	314.66	0.90	0.567	0.884	n.a.	n.a.	n.a.	0.00	0.00
27.890	394.900	3.600	3366.3	3123.5	305.902	0.916	1.54		Unsaturated	0.0			373.25	0.90	336.81	336.81	0.90	0.567	0.883	n.a.	n.a.	n.a.	0.00	0.00
28.050	393.700	3.500	3386.3	3133.5	304.473	0.893	1.53		Unsaturated	0.0			372.12	0.90	335.50	335.50	0.90	0.566	0.882	n.a.	n.a.	n.a.	0.00	0.00
28.220	398.300	3.500	3407.5	3144.2	307.516	0.883	1.52		Unsaturated	0.0			376.47	0.90	339.12	339.12	0.90	0.566	0.881	n.a.	n.a.	n.a.	0.00	0.00
28.380	424.700	2.500	3427.5	3154.2	327.457	0.591	1.38		Unsaturated	0.0			401.42	0.90	361.29	361.29	0.90	0.565	0.880	n.a.	n.a.	n.a.	0.00	0.00
28.540	385.300	2.700	3447.5	3164.2	296.477	0.704	1.46		Unsaturated	0.0			364.18	0.90	327.50	327.50	0.90	0.565	0.879	n.a.	n.a.	n.a.	0.00	0.00
28.710	328.800	2.500	3468.8	3174.8	252.375	0.764	1.54		Unsaturated	0.0			310.78	0.90	279.23	279.23	0.90	0.564	0.878	n.a.	n.a.	n.a.	0.00	0.00
28.870	319.700	2.400	3488.8	3184.9	244.959	0.755	1.54		Unsaturated	0.0			302.17	0.90	271.27	271.27	0.89	0.564	0.877	n.a.	n.a.	n.a.	0.00	0.00
29.040	319.100	4.800	3510.0	3195.5	244.081	1.513	1.77		Unsaturated	4.6			301.61	0.90	270.53	270.64	0.89	0.563	0.876	n.a.	n.a.	n.a.	0.00	0.00
29.200	329.400	3.500	3530.0	3205.5	251.601	1.068	1.64		Unsaturated	0.0			311.34	0.90	279.03	279.03	0.89	0.563	0.875	n.a.	n.a.	n.a.	0.00	0.00
29.360	298.000	4.200	3550.0	3215.5	227.126	1.418	1.77		Unsaturated	4.3			281.66	0.89	251.99	252.07	0.89	0.562	0.874	n.a.	n.a.	n.a.	0.00	0.00
29.530	191.100	3.800	3571.3	3226.2	144.914	2.007	2.01		Unsaturated	23.6			180.62	0.88	158.31	212.93	0.89	0.562	0.873	n.a.	n.a.	n.a.	0.00	0.00
29.690	142.200	2.900	3591.3	3236.2	107.309	2.065	2.10		Unsaturated	31.3	180.62		180.62	0.88	159.51	229.29	0.89	0.561	0.873	n.a.	n.a.	n.a.	0.00	0.00
29.860	92.900	2.600	3612.5	3246.8	69.507	2.854	2.34		Unsaturated	49.9	180.62		180.62	0.89	160.98	249.55	0.89	0.561	0.872	n.a.	n.a.	n.a.	0.00	0.00
30.020	53.500	1.700	3632.5	3256.9	31.738	3.289	2.63		Unsaturated	73.0			50.57	0.81	40.95	104.07	0.89	0.560	0.953	n.a.	n.a.	n.a.	0.00	0.00
30.190	59.500	1.200	3653.8	3267.5	43.867	2.081	2.39		Unsaturated	54.1			56.24	0.81	45.51	105.14	0.89	0.560	0.952	n.a.	n.a.	n.a.	0.00	0.00
30.350	50.500	0.500	3673.8	3277.5	36.957	1.027	2.27		Unsaturated	44.3	56.24		56.24	0.80	45.25	100.59	0.89	0.559	0.953	n.a.	n.a.	n.a.	0.00	0.00
30.510	32.500	0.500	3693.8	3287.5	18.648	1.631	2.62		Unsaturated	73.0			30.72	0.79	24.21	82.41	0.89	0.559	0.959	n.a.	n.a.	n.a.	0.00	0.00
30.680	28.900	0.500	3715.0	3298.2	16.398	1.849	2.70		Unsaturated	79.1			27.32	0.78	21.43	79.74	0.89	0.558	0.960	n.a.	n.a.	n.a.	0.00	0.00
30.840	28.100	0.400	3735.0	3308.2	15.859	1.525	2.67		Unsaturated	76.5			26.56	0.78	20.77	78.50	0.88	0.558	0.960	n.a.	n.a.	n.a.	0.00	0.00
31.010	30.000	0.500	3756.3	3318.8	16.947	1.778	2.68		Unsaturated	77.4			28.36	0.78	22.19	80.48	0.88	0.557	0.959	n.a.	n.a.	n.a.	0.00	0.00
31.170	33.100	0.800	3776.3	3328.8	18.752	2.563	2.73		Unsaturated	81.8			31.29	0.78	24.54	84.15	0.88	0.557	0.957	n.a.	n.a.	n.a.	0.00	0.00
31.330	41.600	1.500	3796.3	3338.9	23.782	3.778	2.76		Unsaturated	83.7			39.32	0.79	31.10	92.94	0.88	0.556	0.954	n.a.	n.a.	n.a.	0.00	0.00
31.500	51.700	1.700	3817.5	3349.5	29.731	3.414	2.66		Unsaturated	75.6			48.87	0.80	38.95	101.93	0.88	0.556	0.950	n.a.	n.a.	n.a.	0.00	0.00
31.660	69.400	2.200	3837.5	3359.5	50.619	3.260	2.47		Unsaturated	60.9	83.84	1.66	139.17	0.86	119.68	202.30	0.88	0.555	0.875	n.a.	n.a.	n.a.	0.00	0.00
31.830	88.700	2.800	3858.8	3370.2	64.986	3.227	2.39		Unsaturated	54.6		1.66	139.17	0.86	119.33	199.24	0.88	0.554	0.879	n.a.	n.a.	n.a.	0.00	0.00
31.990	86.500	2.500	3878.8	3380.2	63.237	2.956	2.38		Unsaturated	53.1		1.66	135.72	0.85	115.88	194.14	0.88	0.554	0.885	n.a.	n.a.	n.a.	0.00	0.00
32.150	86.900	2.700	3898.8	3390.2	63.435	3.178	2.40		Unsaturated	54.8		1.66	136.35	0.85	116.41	195.62	0.88	0.553	0.882	n.a.	n.a.	n.a.	0.00	0.00
32.320	77.700	3.000	3920.0	3400.8	56.468	3.961	2.50		Unsaturated	63.1		1.66	121.91	0.85	103.02	181.67	0.88	0.553	0.898	n.a.	n.a.	n.a.	0.00	0.00
32.480	107.800	3.900	3940.0	3410.8	78.786	3.685	2.38		Unsaturated	53.5		1.66	169.14	0.87	147.54	234.56	0.88	0.552	0.857	n.a.	n.a.	n.a.	0.00	0.00
32.650	81.100	3.300	3961.3	3421.5	58.809	4.171	2.51		Unsaturated	63.5	101.89	1.66	169.14	0.87	147.80	239.30	0.88	0.552	0.856	n.a.	n.a.	n.a.	0.00	0.00
32.810	73.500	3.200	3981.3	3431.5	41.678	4.475	2.63		Unsaturated	73.5			69.47	0.80	55.89	123.49	0.87	0.551	0.938	n.a.	n.a.	n.a.	0.00	0.00
32.970	59.900	2.900	4001.3	3441.5	33.648	5.009	2.73		Unsaturated	81.5			56.62	0.79	44.93	110.62	0.87	0.551	0.944	n.a.	n.a.	n.a.	0.00	0.00
33.140	61.900	2.500	4022.5	3452.2	34.696	4.174	2.67		Unsaturated	76.3			58.51	0.79	46.41	111.73	0.87	0.550	0.943	n.a.	n.a.	n.a.	0.00	0.00
33.300	58.100	2.300	4042.5	3462.2	32.395	4.101	2.68		Unsaturated	77.7			54.91	0.79	43.33	107.96	0.87	0.550	0.944	n.a.	n.a.	n.a.	0.00	0.00

CORNERSTONE EARTH GROUP

CPT No.	4

PGA (A_{max})	0.97

Total Settlement:	0.00	(Inches)

Depth (ft)	q_c (tsf)	f_s (tsf)	σ_{vc} (psf)	Insitu σ'_{vc} (psf)	Q	F (%)	I_c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q_{cN} near interfaces (soft layer)	Thin Layer Factor (K_H)	Interpreted q_{cN}	C_N	q_{c1N}	$q_{c1N\text{-}CS}$	Stress Reduction Coeff, r_d	CSR	K_σ for Sand	$CRR_{M=7.5, \sigma'vc = 1 atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε_v	Settlement (Inches)
33.470	64.900	3.700	4063.8	3472.8	36.206	5.885	2.76		Unsaturated	83.7			61.34	0.79	48.72	115.88	0.87	0.549	0.940	n.a.	n.a.	n.a.	0.00	0.00
33.630	109.800	3.500	4083.8	3482.8	79.388	3.248	2.34		Unsaturated	50.0	133.84		133.84	0.84	112.69	188.55	0.87	0.549	0.885	n.a.	n.a.	n.a.	0.00	0.00
33.790	141.600	4.200	4103.8	3492.9	102.661	3.010	2.24		Unsaturated	42.1			133.84	0.84	112.12	182.79	0.87	0.548	0.891	n.a.	n.a.	n.a.	0.00	0.00
33.960	115.800	3.000	4125.0	3503.5	83.546	2.638	2.26		Unsaturated	43.5	133.84		133.84	0.84	112.08	183.75	0.87	0.548	0.889	n.a.	n.a.	n.a.	0.00	0.00
34.120	105.600	3.100	4145.0	3513.5	75.938	2.994	2.32		Unsaturated	49.0	133.84		133.84	0.84	112.27	187.44	0.87	0.547	0.884	n.a.	n.a.	n.a.	0.00	0.00
34.290	81.400	2.700	4166.3	3524.2	58.091	3.404	2.45		Unsaturated	58.6	133.84		133.84	0.84	112.53	192.28	0.87	0.547	0.877	n.a.	n.a.	n.a.	0.00	0.00
34.450	69.000	2.700	4186.3	3534.2	37.863	4.035	2.63		Unsaturated	73.3			65.22	0.79	51.48	117.76	0.87	0.546	0.937	n.a.	n.a.	n.a.	0.00	0.00
34.610	52.800	2.500	4206.3	3544.2	28.608	4.931	2.78		Unsaturated	85.2			49.91	0.78	38.73	103.07	0.87	0.546	0.944	n.a.	n.a.	n.a.	0.00	0.00
34.780	50.000	2.000	4227.5	3554.8	26.942	4.177	2.75		Unsaturated	82.8			47.26	0.77	36.49	99.83	0.86	0.545	0.945	n.a.	n.a.	n.a.	0.00	0.00
34.940	39.500	1.600	4247.5	3564.8	20.969	4.281	2.84		Unsaturated	89.9			37.33	0.76	28.46	90.23	0.86	0.544	0.949	n.a.	n.a.	n.a.	0.00	0.00
35.110	66.700	1.800	4268.8	3575.5	46.947	2.788	2.45		Unsaturated	59.1			63.04	0.78	49.17	111.40	0.86	0.544	0.939	n.a.	n.a.	n.a.	0.00	0.00
35.270	44.300	1.900	4288.8	3585.5	23.514	4.507	2.81		Unsaturated	88.1			41.87	0.76	31.99	94.64	0.86	0.543	0.946	n.a.	n.a.	n.a.	0.00	0.00
35.430	44.000	1.900	4308.8	3595.5	23.277	4.540	2.82		Unsaturated	88.5			41.59	0.76	31.72	94.33	0.86	0.543	0.946	n.a.	n.a.	n.a.	0.00	0.00
35.600	40.800	1.500	4330.0	3606.2	21.427	3.882	2.80		Unsaturated	87.1			38.56	0.76	29.24	90.94	0.86	0.542	0.947	n.a.	n.a.	n.a.	0.00	0.00
35.760	48.400	1.500	4350.0	3616.2	25.566	3.245	2.69		Unsaturated	78.4			45.75	0.76	34.91	97.14	0.86	0.542	0.944	n.a.	n.a.	n.a.	0.00	0.00
35.930	36.400	1.700	4371.3	3626.8	18.867	4.969	2.91		Unsaturated	96.1			34.40	0.75	25.90	87.51	0.86	0.541	0.948	n.a.	n.a.	n.a.	0.00	0.00
36.090	41.600	1.400	4391.3	3636.8	21.670	3.553	2.77		Unsaturated	84.8			39.32	0.76	29.70	91.25	0.86	0.541	0.946	n.a.	n.a.	n.a.	0.00	0.00
36.260	54.500	2.200	4412.5	3647.5	28.674	4.207	2.73		Unsaturated	81.4			51.51	0.77	39.44	103.46	0.86	0.540	0.941	n.a.	n.a.	n.a.	0.00	0.00
36.420	76.800	2.700	4432.5	3657.5	53.620	3.620	2.49		Unsaturated	62.1	216.26		216.26	0.87	187.19	289.32	0.86	0.540	0.836	n.a.	n.a.	n.a.	0.00	0.00
36.580	228.800	3.100	4452.5	3667.5	162.666	1.368	1.85		Unsaturated	11.0			216.26	0.83	179.51	193.21	0.86	0.539	0.866	n.a.	n.a.	n.a.	0.00	0.00
36.750	261.500	4.300	4473.8	3678.2	185.865	1.659	1.87		Unsaturated	13.0			247.16	0.85	210.98	234.45	0.85	0.539	0.834	n.a.	n.a.	n.a.	0.00	0.00
36.910	288.700	4.700	4493.8	3688.2	205.079	1.641	1.84		Unsaturated	10.5			272.87	0.86	234.87	248.54	0.85	0.538	0.833	n.a.	n.a.	n.a.	0.00	0.00
37.080	326.800	5.000	4515.0	3698.8	232.013	1.541	1.79		Unsaturated	6.1			308.88	0.86	266.57	267.70	0.85	0.537	0.832	n.a.	n.a.	n.a.	0.00	0.00
37.240	315.800	4.800	4535.0	3708.8	223.839	1.531	1.80		Unsaturated	6.7			298.49	0.86	257.41	259.39	0.85	0.537	0.832	n.a.	n.a.	n.a.	0.00	0.00
37.400	325.700	4.800	4555.0	3718.8	230.589	1.484	1.78		Unsaturated	5.3			307.84	0.86	265.29	265.67	0.85	0.536	0.831	n.a.	n.a.	n.a.	0.00	0.00
37.570	394.300	4.900	4576.3	3729.5	279.092	1.250	1.67		Unsaturated	0.0			372.68	0.86	320.93	320.93	0.85	0.536	0.830	n.a.	n.a.	n.a.	0.00	0.00
37.730	465.000	3.600	4596.3	3739.5	328.978	0.778	1.46		Unsaturated	0.0			439.51	0.86	378.20	378.20	0.85	0.535	0.829	n.a.	n.a.	n.a.	0.00	0.00
37.900	474.300	4.200	4617.5	3750.1	335.106	0.890	1.50		Unsaturated	0.0			448.30	0.86	385.48	385.48	0.85	0.535	0.828	n.a.	n.a.	n.a.	0.00	0.00
38.060	469.500	3.800	4637.5	3760.2	331.249	0.813	1.48		Unsaturated	0.0			443.76	0.86	381.31	381.31	0.85	0.534	0.828	n.a.	n.a.	n.a.	0.00	0.00
38.220	439.000	4.400	4657.5	3770.2	309.205	1.008	1.57		Unsaturated	0.0			414.93	0.86	356.29	356.29	0.85	0.534	0.827	n.a.	n.a.	n.a.	0.00	0.00
38.390	453.000	5.100	4678.8	3780.8	318.661	1.132	1.60		Unsaturated	0.0			428.17	0.86	367.38	367.38	0.85	0.533	0.826	n.a.	n.a.	n.a.	0.00	0.00
38.550	467.700	8.200	4698.8	3790.8	328.613	1.762	1.75		Unsaturated	2.9			442.06	0.86	379.03	379.03	0.84	0.533	0.825	n.a.	n.a.	n.a.	0.00	0.00
38.720	481.700	9.500	4720.0	3801.5	338.018	1.982	1.79		Unsaturated	5.8			455.29	0.86	390.09	391.10	0.84	0.532	0.824	n.a.	n.a.	n.a.	0.00	0.00
38.880	466.500	6.900	4740.0	3811.5	326.862	1.487	1.69		Unsaturated	0.0			440.93	0.86	377.52	377.52	0.84	0.532	0.823	n.a.	n.a.	n.a.	0.00	0.00
39.040	458.200	6.500	4760.0	3821.5	320.589	1.426	1.68		Unsaturated	0.0			433.08	0.86	370.54	370.54	0.84	0.531	0.823	n.a.	n.a.	n.a.	0.00	0.00
39.210	552.300	6.900	4781.3	3832.1	386.227	1.255	1.59		Unsaturated	0.0			522.02	0.85	446.32	446.32	0.84	0.531	0.822	n.a.	n.a.	n.a.	0.00	0.00
39.370	591.200	6.700	4801.3	3842.2	413.002	1.138	1.54		Unsaturated	0.0			558.79	0.85	477.42	477.42	0.84	0.530	0.821	n.a.	n.a.	n.a.	0.00	0.00
39.540	563.900	6.300	4822.5	3852.8	393.301	1.122	1.54		Unsaturated	0.0			532.99	0.85	455.04	455.04	0.84	0.529	0.820	n.a.	n.a.	n.a.	0.00	0.00
39.700	465.200	4.400	4842.5	3862.8	323.738	0.951	1.54		Unsaturated	0.0			439.70	0.85	375.14	375.14	0.84	0.529	0.819	n.a.	n.a.	n.a.	0.00	0.00
39.860	506.400	4.500	4862.5	3872.8	352.096	0.893	1.49		Unsaturated	0.0			478.64	0.85	408.09	408.09	0.84	0.528	0.819	n.a.	n.a.	n.a.	0.00	0.00
40.030	540.200	6.000	4883.8	3883.5	375.188	1.116	1.55		Unsaturated	0.0			510.59	0.85	435.01	435.01	0.84	0.528	0.818	n.a.	n.a.	n.a.	0.00	0.00
40.190	505.500	4.500	4903.8	3893.5	350.519	0.895	1.49		Unsaturated	0.0			477.79	0.85	406.79	406.79	0.84	0.527	0.817	n.a.	n.a.	n.a.	0.00	0.00
40.360	484.600	4.700	4925.0	3904.1	335.491	0.975	1.53		Unsaturated	0.0			458.03	0.85	389.69	389.69	0.84	0.527	0.816	n.a.	n.a.	n.a.	0.00	0.00
40.520	470.200	5.000	4945.0	3914.2	325.047	1.069	1.57		Unsaturated	0.0			444.42	0.85	377.85	377.85	0.83	0.526	0.815	n.a.	n.a.	n.a.	0.00	0.00
40.680	505.000	5.500	4965.0	3924.2	348.778	1.094	1.56		Unsaturated	0.0			477.32	0.85	405.55	405.55	0.83	0.526	0.815	n.a.	n.a.	n.a.	0.00	0.00
40.850	475.400	5.000	4986.3	3934.8	327.783	1.057	1.57		Unsaturated	0.0			449.34	0.85	381.50	381.50	0.83	0.525	0.814	n.a.	n.a.	n.a.	0.00	0.00
41.010	468.400	4.100	5006.3	3944.8	322.514	0.880	1.51		Unsaturated	0.0			442.72	0.85	375.63	375.63	0.83	0.525	0.813	n.a.	n.a.	n.a.	0.00	0.00
41.180	460.700	4.100	5027.5	3955.5	316.749	0.895	1.52		Unsaturated	0.0			435.44	0.85	369.20	369.20	0.83	0.524	0.812	n.a.	n.a.	n.a.	0.00	0.00
41.340	434.700	4.800	5047.5	3965.5	298.390	1.111	1.61		Unsaturated	0.0			410.87	0.85	348.13	348.13	0.83	0.524	0.812	n.a.	n.a.	n.a.	0.00	0.00
41.500	424.400	4.100	5067.5	3975.5	290.905	0.972	1.57		Unsaturated	0.0			401.13	0.85	339.65	339.65	0.83	0.523	0.811	n.a.	n.a.	n.a.	0.00	0.00
41.670	423.700	4.100	5088.8	3986.1	290.027	0.974	1.57		Unsaturated	0.0			400.47	0.85	338.85	338.85	0.83	0.522	0.810	n.a.	n.a.	n.a.	0.00	0.00
41.830	429.800	4.100	5108.8	3996.2	293.852	0.960	1.56		Unsaturated	0.0			406.24	0.85	343.50	343.50	0.83	0.522	0.809	n.a.	n.a.	n.a.	0.00	0.00
42.000	425.000	3.800	5130.0	4006.8	290.157	0.900	1.55		Unsaturated	0.0			401.70	0.84	339.43	339.43	0.83	0.521	0.808	n.a.	n.a.	n.a.	0.00	0.00
42.160	431.600	2.500	5150.0	4016.8	294.316	0.583	1.40		Unsaturated	0.0			407.94	0.84	344.47	344.47	0.83	0.521	0.808	n.a.	n.a.	n.a.	0.00	0.00
42.320	427.400	2.900	5170.0	4026.8	291.065	0.683	1.46		Unsaturated	0.0			403.97	0.84	340.90	340.90	0.83	0.520	0.807	n.a.	n.a.	n.a.	0.00	0.00
42.490	440.000	3.900	5191.3	4037.5	299.296	0.892	1.54		Unsaturated	0.0			415.88	0.84	350.70	350.70	0.82	0.520	0.806	n.a.	n.a.	n.a.	0.00	0.00
42.650	464.700	5.000	5211.3	4047.5	315.798	1.082	1.59		Unsaturated	0.0			439.22	0.84	370.15	370.15	0.82	0.519	0.805	n.a.	n.a.	n.a.	0.00	0.00
42.820	392.800	4.800	5232.5	4058.1	266.304	1.230	1.68		Unsaturated	0.0			371.27	0.84	312.66	312.66	0.82	0.519	0.805	n.a.	n.a.	n.a.	0.00	0.00
42.980	426.800	5.400	5252.5	4068.1	289.146	1.273	1.67		Unsaturated	0.0			403.40	0.84	339.50	339.50	0.82	0.518	0.804	n.a.	n.a.	n.a.	0.00	0.00
43.150	478.000	4.500	5273.8	4078.8	323.617	0.947	1.53		Unsaturated	0.0			451.80	0.84	379.97	379.97	0.82	0.518	0.803	n.a.	n.a.	n.a.	0.00	0.00
43.310	499.800	4.000	5293.8	4088.8	338.037	0.805	1.47		Unsaturated	0.0			472.40	0.84	397.04	397.04	0.82	0.517	0.802	n.a.	n.a.	n.a.	0.00	0.00
43.470	516.700	6.500	5313.8	4098.8	349.094	1.264	1.61		Unsaturated	0.0			488.37	0.84	410.20	410.20	0.82	0.516	0.802	n.a.	n.a.	n.a.	0.00	0.00
43.640	519.500	5.100	5335.0	4109.5	350.533	0.987	1.53		Unsaturated	0.0			491.02	0.84	412.14	412.14	0.82	0.516	0.801	n.a.	n.a.	n.a.	0.00	0.00
43.800	520.200	6.300	5355.0	4119.5	350.574	1.217	1.60		Unsaturated	0.0			491.68	0.84	412.43	412.43	0.82	0.515	0.800	n.a.	n.a.	n.a.	0.00	0.00
43.970	488.600	5.500	5376.3	4130.1	328.737	1.132	1.59		Unsaturated	0.0			461.81	0.84	387.12	387.12	0.82	0.515	0.799	n.a.	n.a.	n.a.	0.00	0.00
44.130	495.600	5.400	5396.3	4140.1	333.062	1.096	1.58		Unsaturated	0.0			468.43	0.84	392.41	392.41	0.82	0.514	0.799	n.a.	n.a.	n.a.	0.00	0.00
44.290	463.600	4.900	5416.3	4150.2	311.056	1.063	1.58		Unsaturated	0.0			438.19	0.84	366.84	366.84	0.81	0.514	0.798	n.a.	n.a.	n.a.	0.00	0.00

CORNERSTONE EARTH GROUP

CPT No.	4

PGA (A_{max})	0.97

Total Settlement:	0.00	(Inches)

Depth (ft)	q_c (tsf)	f_s (tsf)	σ_{vc} (psf)	Insitu σ'_{vc} (psf)	Q	F (%)	I_c	Layer "Plastic" PI > 7	Flag Soil Type	Fines (%)	q_{cN} near interfaces (soft layer)	Thin Layer Factor (K_H)	Interpreted q_{cN}	C_N	q_{c1N}	q_{c1N-CS}	Stress Reduction Coeff, r_d	CSR	K_σ for Sand	$CRR_{M=7.5, \sigma'vc = 1 atm}$	CRR	Factor of Safety (CRR/CSR)	Vertical Strain ε_v	Settlement (Inches)
44.460	328.000	3.400	5437.5	4160.8	219.252	1.045	1.68		Unsaturated	0.0			310.02	0.84	259.37	259.37	0.81	0.513	0.797	n.a.	n.a.	n.a.	0.00	0.00
44.620	361.800	3.100	5457.5	4170.8	241.737	0.863	1.59		Unsaturated	0.0			341.97	0.84	285.91	285.91	0.81	0.513	0.796	n.a.	n.a.	n.a.	0.00	0.00
44.790	358.000	3.400	5478.8	4181.5	238.867	0.957	1.62		Unsaturated	0.0			338.37	0.84	282.72	282.72	0.81	0.512	0.796	n.a.	n.a.	n.a.	0.00	0.00
44.950	353.000	3.200	5498.8	4191.5	235.217	0.914	1.61		Unsaturated	0.0			333.65	0.83	278.59	278.59	0.81	0.512	0.795	n.a.	n.a.	n.a.	0.00	0.00
45.110	373.700	3.700	5518.8	4201.5	248.814	0.997	1.62		Unsaturated	0.0			353.21	0.83	294.75	294.75	0.81	0.511	0.794	n.a.	n.a.	n.a.	0.00	0.00
45.280	376.800	4.300	5540.0	4212.1	250.569	1.150	1.67		Unsaturated	0.0			356.14	0.83	296.99	296.99	0.81	0.511	0.793	n.a.	n.a.	n.a.	0.00	0.00
45.440	379.600	1.700	5560.0	4222.1	252.139	0.451	1.38		Unsaturated	0.0			358.79	0.83	299.01	299.01	0.81	0.510	0.793	n.a.	n.a.	n.a.	0.00	0.00
45.610	427.500	2.600	5581.3	4232.8	283.825	0.612	1.43		Unsaturated	0.0			404.06	0.83	336.52	336.52	0.81	0.509	0.792	n.a.	n.a.	n.a.	0.00	0.00
45.770	472.300	4.700	5601.3	4242.8	313.387	1.001	1.56		Unsaturated	0.0			446.41	0.83	371.55	371.55	0.81	0.509	0.791	n.a.	n.a.	n.a.	0.00	0.00
45.930	444.600	4.400	5621.3	4252.8	294.543	0.996	1.58		Unsaturated	0.0			420.23	0.83	349.54	349.54	0.81	0.508	0.791	n.a.	n.a.	n.a.	0.00	0.00
46.100	478.500	6.200	5642.5	4263.5	316.741	1.303	1.65		Unsaturated	0.0			452.27	0.83	375.95	375.95	0.81	0.508	0.790	n.a.	n.a.	n.a.	0.00	0.00
46.260	544.500	5.000	5662.5	4273.5	360.259	0.923	1.50		Unsaturated	0.0			514.65	0.83	427.54	427.54	0.80	0.507	0.789	n.a.	n.a.	n.a.	0.00	0.00
46.430	601.800	6.400	5683.8	4284.1	397.867	1.069	1.52		Unsaturated	0.0			568.81	0.83	472.22	472.22	0.80	0.507	0.788	n.a.	n.a.	n.a.	0.00	0.00
46.590	529.100	5.400	5703.8	4294.1	349.160	1.026	1.54		Unsaturated	0.0			500.09	0.83	414.92	414.92	0.80	0.506	0.788	n.a.	n.a.	n.a.	0.00	0.00
46.750	508.300	6.500	5723.8	4304.2	334.963	1.286	1.63		Unsaturated	0.0			480.43	0.83	398.36	398.36	0.80	0.506	0.787	n.a.	n.a.	n.a.	0.00	0.00
46.920	526.600	5.800	5745.0	4314.8	346.655	1.107	1.57		Unsaturated	0.0			497.73	0.83	412.44	412.44	0.80	0.505	0.786	n.a.	n.a.	n.a.	0.00	0.00
47.080	407.000	5.000	5765.0	4324.8	267.175	1.237	1.68		Unsaturated	0.0			384.69	0.83	318.57	318.57	0.80	0.505	0.786	n.a.	n.a.	n.a.	0.00	0.00
47.250	372.900	3.800	5786.3	4335.5	244.323	1.027	1.64		Unsaturated	0.0			352.46	0.83	291.69	291.69	0.80	0.504	0.785	n.a.	n.a.	n.a.	0.00	0.00
47.410	367.500	3.000	5806.3	4345.5	240.473	0.823	1.57		Unsaturated	0.0			347.35	0.83	287.29	287.29	0.80	0.504	0.784	n.a.	n.a.	n.a.	0.00	0.00
47.570	410.300	3.400	5826.3	4355.5	268.387	0.835	1.55		Unsaturated	0.0			387.81	0.83	320.55	320.55	0.80	0.503	0.783	n.a.	n.a.	n.a.	0.00	0.00
47.740	407.500	2.400	5847.5	4366.1	266.210	0.593	1.44		Unsaturated	0.0			385.16	0.83	318.16	318.16	0.80	0.502	0.783	n.a.	n.a.	n.a.	0.00	0.00
47.900	359.000	2.900	5867.5	4376.1	234.022	0.814	1.58		Unsaturated	0.0			339.32	0.83	280.13	280.13	0.80	0.502	0.782	n.a.	n.a.	n.a.	0.00	0.00
48.070	375.900	3.000	5888.8	4386.8	244.825	0.804	1.56		Unsaturated	0.0			355.29	0.83	293.12	293.12	0.80	0.501	0.781	n.a.	n.a.	n.a.	0.00	0.00
48.230	429.300	3.600	5908.8	4396.8	279.554	0.844	1.54		Unsaturated	0.0			405.77	0.82	334.56	334.56	0.79	0.501	0.781	n.a.	n.a.	n.a.	0.00	0.00
48.390	470.400	3.200	5928.8	4406.8	306.148	0.685	1.44		Unsaturated	0.0			444.61	0.82	366.37	366.37	0.79	0.500	0.780	n.a.	n.a.	n.a.	0.00	0.00
48.560	496.700	3.500	5950.0	4417.5	322.977	0.709	1.44		Unsaturated	0.0			469.47	0.82	386.61	386.61	0.79	0.500	0.779	n.a.	n.a.	n.a.	0.00	0.00
48.720	458.800	3.000	5970.0	4427.5	297.840	0.658	1.44		Unsaturated	0.0			433.65	0.82	356.90	356.90	0.79	0.499	0.779	n.a.	n.a.	n.a.	0.00	0.00
48.890	444.000	2.900	5991.3	4438.1	287.817	0.658	1.45		Unsaturated	0.0			419.66	0.82	345.17	345.17	0.79	0.499	0.778	n.a.	n.a.	n.a.	0.00	0.00
49.050	415.000	3.300	6011.3	4448.1	268.581	0.801	1.53		Unsaturated	0.0			392.25	0.82	322.43	322.43	0.79	0.498	0.777	n.a.	n.a.	n.a.	0.00	0.00
49.220	384.900	3.700	6032.5	4458.8	248.654	0.969	1.62		Unsaturated	0.0			363.80	0.82	298.86	298.86	0.79	0.498	0.776	n.a.	n.a.	n.a.	0.00	0.00
49.380	361.600	3.300	6052.5	4468.8	233.215	0.920	1.62		Unsaturated	0.0			341.78	0.82	280.60	280.60	0.79	0.497	0.776	n.a.	n.a.	n.a.	0.00	0.00
49.540	388.500	3.600	6072.5	4478.8	250.423	0.934	1.60		Unsaturated	0.0			367.20	0.82	301.30	301.30	0.79	0.497	0.775	n.a.	n.a.	n.a.	0.00	0.00
49.710	419.700	2.200	6093.8	4489.4	270.365	0.528	1.40		Unsaturated	0.0			396.69	0.82	325.29	325.29	0.79	0.496	0.774	n.a.	n.a.	n.a.	0.00	0.00
49.870	471.700	2.500	6113.8	4499.5	303.762	0.533	1.37		Unsaturated	0.0			445.84	0.82	365.38	365.38	0.79	0.496	0.774	n.a.	n.a.	n.a.	0.00	0.00
50.040	474.600	3.300	6135.0	4510.1	305.274	0.700	1.45		Sand	0.0			448.58	0.82	367.39	367.39	0.79	0.495	0.773	########	########	#########	0.00	0.00
50.200	450.900	3.700	6155.0	4520.1	289.603	0.826	1.52		Sand	0.0			426.18	0.82	348.84	348.84	0.78	0.496	0.772	########	########	#########	0.00	0.00
50.360	434.900	3.800	6175.0	4530.1	278.941	0.880	1.55		Sand	0.0			411.06	0.82	336.27	336.27	0.78	0.496	0.772	########	########	#########	0.00	0.00
50.530	360.800	4.200	6196.3	4540.8	230.796	1.174	1.70		Sand	0.0			341.02	0.82	278.80	278.80	0.78	0.496	0.771	8090.130	13721.210	27657.16	0.00	0.00
50.690	319.500	4.500	6216.3	4550.8	203.917	1.422	1.80		Sand	6.8			301.98	0.81	245.34	247.39	0.78	0.496	0.770	96.153	162.940	328.24	0.00	0.00
50.860	281.100	4.800	6237.5	4561.4	178.952	1.727	1.90		Sand	14.9			265.69	0.81	215.88	248.24	0.78	0.497	0.770	106.005	179.471	361.32	0.00	0.00

APPENDIX D: DRY SAND SETTLEMENT CALCULATIONS

Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils

(Pradel, 1998)

a_{max} 0.97

M_w 7.18

Piedmont Gymnasium																
Pradell (1998) Procedure																
Boring	Depth (ft)	Thickness (ft)	σ_v (psf)	N'_{60cs} (bpf)	τ_{av} (psf)	P (psf)	G_{max} (psf)	a	b	γ (%)	ε_{15} (%)	N_c (cycles)	$2*\varepsilon_{Nc}$ (%)	ΔS (in)	Depth Factor	Total (in)
B-1	3	4.5	360	29.6	227	240	957,361	0.129	22,839	0.62	0.39	12.31	0.72	0.39	0.96	0.37
B-2	3	4.0	360	18.8	227	240	823,685	0.129	22,839	1.71	1.84	12.31	3.37	1.62	0.97	1.56