CORNERSTONE EARTH GROUP

Type of Services	**Preliminary Geotechnical Feasibility Study**
Project Name	**District Office Building and Adult Education Classroom - Westmoor Park Site**
Location	**123 Edgemont Drive Daly City, California**
Client	**Jefferson Union High School District**
Client Address	**699 Serramonte Boulevard Daly City, California**
Project Number	**250-4-9**
Date	**May 20, 2020**

DRAFT

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SECTION 1: INTRODUCTION

This geotechnical feasibility evaluation was prepared for the sole use of Jefferson Union High School District for the Westmoor Park Site project in Daly City, California. The purpose of this preliminary geotechnical feasibility study was to evaluate the existing subsurface conditions and develop an opinion regarding potential geotechnical concerns that could impact the proposed development. The preliminary geotechnical recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design. For our use, we were provided with the following documents:

- A conceptual site plan titled “Site Plan – Concept 10, JUHSD District Office, Daly City, CA 94015,” prepared by Forge, dated February 26, 2020.
- A conceptual plan sheet titled “Section & 3D View – Concept 10, JUHSD District Office, Daly City, CA 94015,” prepared by Forge, dated February 26, 2020.

1.1 PROJECT DESCRIPTION

The project will consist of a new district office building, parking lot, and potential new adult education classroom building. The proposed development will be located at existing Westmoor Park site located directly south of the Westmoor High School Campus. The planned office building will be an at-grade, two-story structure. The potential classroom building will also be a two-story, at-grade structure. The planned district office building will have a footprint of approximately 17,000 to 18,000 square feet. The square footage of the potential classroom building is not known at this time. For planning purposes, we have scaled the conceptual layout provided and have estimated the footprint to be approximately 13,000 square feet. As part of the potential classroom building, the additional development would also include a new basketball court. The new parking lot will be at-grade and is currently planned for the northwest corner of the site. Appurtenant utilities, landscaping and other improvements necessary for site development are also planned.

Structural loads are not currently known for the proposed structure(s); however, structural loads are expected to be typical of similar type structures. Grading for the proposed development is expected to be minor. The existing park appears to have been constructed on previously graded terraces similar to the adjacent Westmoor High School campus.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated March 26, 2020 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this feasibility report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on May 1, 2020 with truck-mounted hollow-stem auger drilling equipment and one boring drilled on May 1, 2020 with hand-auger equipment. The borings were drilled to depths ranging from about 5½ to 29½ feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGIC SETTING

2.1.1 Regional Geologic Setting

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

Movement on the many splays 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: the North American plate to the east and the Pacific plate to the west. The San Andreas fault system 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 this system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

The Westmoor Park site is located on a modified ridge top in an area of otherwise undulating terrain. The actual site exists on a flat, partially graded terrace with low to moderately steeply inclined downslopes bordering on the east, south, and west. A southerly facing downslope exists on the north. The site is underlain by the Plio-Pleistocene Merced Formation (QTm). The Merced Formation has been folded into a series of anticlinal and synclinal folds by regional tectonic activity along the San Andreas Fault system.

2.2 REGIONAL SEISMICITY

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

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site (CDMG, 1998).

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	0.9	1.5
San Gregorio	4.7	7.5

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

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The project site is located at Westmoor Park at 123 Edgemont Drive in Daly City. The site is located directly south of the Westmoor High School campus. The site is currently occupied by an existing building and at-grade parking lot in the northwest corner, tennis courts in the northeast corner, a baseball diamond in the center of the site, and grass fields and concrete sidewalks throughout the remainder of the park. The site is relatively level, with elevations on the order of 422 feet to 426 feet, except in the southwest corner where elevations drop to about 415 to 418 feet and along the southwest, south and southeast perimeters of the site where elevations drop toward the adjacent streets and properties to about 400 to 418 feet based on Google Earth®. Our review of pre-development aerial photos indicates the site was created by cutting the top of the natural ridge into a flat surface and placing fills off the margins of the new terrace surface. The northwest, north, and northeast perimeters of the site were likely created by a combination of minor cutting and filling. Some minor surficial sheet fill exists within the north-central and southern portions of the property. The inclination of the cut and fill slopes are about 2:1 (horizontal to vertical). The existing park appears to have been constructed on previously graded terraces similar to the adjacent Westmoor High School campus.

Surface pavements were not encountered during our preliminary investigation; however, they are present at the site. Based on visual observations, the existing pavements are in poor condition with surface erosion and cracking observed.

3.2 SUBSURFACE CONDITIONS

Below the ground surface, our explorations generally encountered undocumented fill consisting of medium dense silty sand to depths of approximately 2½ to 5 feet. Beneath the fills, our borings generally encountered medium dense to very dense poorly graded sand with silt of the Merced Formation to the maximum depth explored of about 29½ feet. We note that likely thicker fill is present at the site around the perimeters of the terrace. We noted a subtle landform within the north-central portion of the property that may represent a surficial sheet fill about 18 inches thick. This fill may extend locally deeper into the ground surface in the area of HA-1 where we encountered approximately 3 feet of fill.

3.2.1 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 3 percent below to about 3 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was not encountered in any of our borings during drilling; however, the borings were not left open but were immediately backfilled when the boring was completed. Historic high groundwater levels are not currently mapped by California Geological Survey (CGS). We also reviewed relatively recent groundwater level data from three wells in the California Department of Water Resources Water Data Library within one mile of the campus (Stations 376669N1224483W001, 376852N1224702W001, and 376954N1224798W001), which shows groundwater levels ranging from as high as approximately elevation -24 feet to -64 feet mean sea level (msl) and as low as approximately elevation -80 feet to -94 feet msl. The earliest data available from these stations is from November of 2011. Data recorded in the last two years shows groundwater levels ranging from approximately elevation -45 feet to -73 feet msl. Therefore, we anticipate groundwater is at a depth of greater than 50 feet below the existing ground surface.

We did not encounter groundwater in any of our explorations conducted in 2015 on the adjacent Westmoor High School campus; however, we encountered groundwater in borings EB-18, EB-19, and EB-20 on the adjacent Westmoor High School campus in 2016 at depths of approximately 11½ to 18 feet. We believe that this was perched groundwater associated with the presence of claystone (i.e. fat clays). In addition, those surface elevations were higher than the subject site. The maximum depth of 35 feet achieved at those locations would not be equivalent to the ground surface of the subject site.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

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

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration $(PGA)_M$ was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2019 edition of the California Building Code. For our liquefaction screening analysis we used a PGA_M of 1.214g. We have assumed a site-specific analysis will not be required for this project; therefore, this is a code-based value of PGA_M. If a site-specific analysis is performed, this value may change.

4.3 LIQUEFACTION POTENTIAL

The site has not been evaluated for liquefaction by the State of California (CGS, 2000), but is within a zone mapped as having a very low liquefaction potential by the Association of Bay Area Governments (ABAG, 2020). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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

As discussed in the “Subsurface” section above, we primarily encountered medium dense to very dense granular soils. In addition, the design groundwater level is anticipated to be below a depth of 50 feet. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction, and is in general agreement with local mapping for the site by ABAG.

4.4 LATERAL SPREADING

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

Since groundwater is anticipated to be 50 feet or greater below the ground surface and the potential for liquefaction is low, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose to medium dense unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the medium dense native poorly graded sands based on the work by Pradel (1998). For this feasibility study we assume the medium dense silty sands will be removed during removal of undocumented fill and have not considered them in our analysis. Our analyses indicate that the native sands could experience up to about ½-inch or less of movement after strong seismic shaking.

4.6 LANDSLIDING

Leighton (1976) characterized the slope stability and earthquake stability of the Merced Formation as fair to good and good, respectively. However, this is a general characterization and not based on site-specific information. There are no landslides shown at or immediately adjacent to the site on the various published geologic maps reviewed for this evaluation (Bonilla 1960, 1998; California Geological Survey, 2000, 2016).

Sloping portions of the site occur around the perimeter (north, east, and west sides) of the northwest area, along the north and east sides of the east area, and around the perimeter of the baseball field terrace. The southerly facing slopes along the north perimeter are likely compound slopes (cut into Merced Formation within the basal portion, and fill in the upper portion. The slopes around the west, east, and south sides of the baseball field are considered to be fill slopes and the southerly-facing slopes in the eastern portion of the site are thought to be fill slopes. The fills at and adjacent to the site are considered to be underlain by the Merced Formation. These slopes are generally moderately inclined. Our reconnaissance and review of historic aerial photos suggested no evidence of past landsliding at the site. Therefore, the potential for landsliding occurring at the site is judged to be low.

4.7 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 0.6 miles inland from the Pacific Ocean coastline, about 4.6 miles from the San Francisco Bay shoreline, and is approximately 415 to 426 feet above mean sea level based on Google Earth®. The site is not mapped within a Tsunami Inundation area by CGS (City and County of San Francisco, 2009). Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.8 FLOODING

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

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. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical and geologic hazards investigation should be performed once site development plans are prepared indicating where proposed structures are planned. The design-level investigation findings will be used to confirm the preliminary recommendations and develop detailed recommendations for design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Presence of undocumented fill
- Excavation in granular soils
- Stability of slopes

5.1.1 Presence of Undocumented Fill

As discussed, up to 5 feet of fill was encountered in our explorations. On a preliminary basis, we recommend all existing fill materials be removed from within the building footprint and replaced as engineered fill. Due to the existing site development, it is likely that localized areas of undocumented fill may be encountered elsewhere on the site. The distribution, thickness, and density of the undocumented fill should be further evaluated during the design-level geotechnical investigation.

5.1.2 Excavation in Granular Soils

The soil encountered during our subsurface exploration consists of fine to medium sand ranging from loose to medium dense. The sandy soil is likely to not stand vertical when excavated (i.e. foundation, utility trench, and drilled pier excavations). The contractor will need to address this issue. We recommend that consideration be given to installing Stay-Form®, or similar, on all excavations including shallow footings/grade beams, drilled pier foundations, and trenches to reduce the potential for sidewall collapse.

5.1.3 Stability of Slopes

While the majority of the site is a relatively flat terrace, this terrace was created by shaving off undulating ground on a natural ridgetop and placing fill around the margins of the terrace in order to create a slightly large parcel of useable ground. A quantitative slope stability analysis was not performed as part of our preliminary investigation. It is our judgment that the existing slopes are relatively stable based on their apparently acceptable performance over the years. We recommend the slope to the south of the site, adjacent to the existing fire station, be evaluated during our design-level investigation to verify the proposed site improvements will not affect the stability of the existing slope.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this feasibility study were based on limited site development information and limited exploration and our experience in the area with similar projects. As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that we be retained to 1) perform a design-level geotechnical investigation, once detailed site development plans are available; 2) 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; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: EARTHWORK

6.1 ANTICIPATED EARTHWORK MEASURES

Standard earthwork practices for demolition and site preparation will be required. As discussed above, there are undocumented fills and medium dense sandy soils present at the site. The undocumented fill within the building pads should be removed and replaced with engineered fill prior to building construction.

On a preliminary basis, we recommend that the site be stripped of all surface vegetation; site stripping should extend from 3 to 6 inches below the surface. Additionally, any existing improvements and/or abandoned underground utilities should be removed entirely from within the new building areas, and the resulting excavations be backfilled with engineered fill. Any native soils that are disturbed during demolition of existing improvements should also be removed and replaced as engineered fill. Furthermore, we recommend all fills encountered during site grading be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the structures; footprints or to a lateral distance equal to fill depth below the perimeter footings, whichever is greater. For planning purposes, an over-excavation depth of 5 feet in the office building and 3 feet in the classroom building should be anticipated for the undocumented fill removals. Additional characterization of the lateral distribution of the fill should be performed as part of the design-level geotechnical investigation.

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted. Due to the sandy soils likely to be encountered at the subgrade elevation, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction. Depending on the time of year, subgrade soils may be overly wet and unstable, and require stabilization. There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill including scarification and drying, removal and replacement, and chemical treatment. Additional subgrade stabilization recommendations will be provided in the design-level investigation.

On-site soils below the stripping depth appear to be suitable for use as fill at the site. Imported fill material for use as general fill should be predominantly granular with a Plasticity Index of 15 or less. All fill as well as scarified surface soils in those areas to receive fill or slabs-on-grade should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 2 percent above optimum. The upper 6 inches of subgrade in pavement areas and all aggregate base materials should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition). Utility trench backfill should be compacted to at least 90 percent relative compaction (ASTM D-1557, latest edition) by mechanical means only.

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. General backfill over shading materials may consist of on-site native materials provided they are of suitable material, and are moisture conditioned and compacted.

Imported fill material for use as general fill should be predominantly granular with a Plasticity index of 15 or less. All fill as well as scarified soils in those areas to receive fill or slabs-on-grade should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 2 percent above optimum. Areas of fill placed behind retaining walls where surface improvements are planned should be compacted to 95 percent. The upper 6 inches of subgrade in pavement areas and all aggregate base materials should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition). Utility trench backfill should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition) by mechanical means only.

Surface water runoff should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 1 to 2 percent towards suitable discharge facilities; landscape areas should slope at least 2 to 3 percent away from buildings. Bio-treatment basins should be kept at least 10 feet away from building and, where possible, at least 3 feet from pavements and flatwork. Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Site slopes for water discharge should confirm to the building code. 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. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site.

Upslope sources of water should be evaluated. If upslope irrigation of is present or planned, additional surface and subsurface drainage, or construction of drained buttress fills may be needed to protect site improvements. We should be consulted if this issue will affect the project.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, the proposed structures may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed. Additional exploration and analysis should be performed in a design-level investigation to further evaluate static settlements.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2019 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, on a preliminary basis, the site is underlain by shallow undocumented fill underlain by the Merced formation with typical SPT “N” values greater than 50 blows per foot. Therefore, on a preliminary basis, we have classified the site as Soil Classification C. We recommend additional explorations be performed during our design-level investigation to confirm the site classification. The mapped spectral acceleration parameters S_S and S_1 were calculated using the SEAOC/OSHPD Seismic Design Maps on-line calculator (https://seismicmaps.org/), based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters. For our preliminary report we have not performed a site-specific analysis. If an exception is not taken by the structural engineer, we should be notified and a site-specific analysis and revised seismic design coefficients will be performed as part of the design-level investigation. The SEAOC/OSHPD Seismic Design parameter outputs are provided in Appendix C of this report.

Table 2: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.67997°
Site Longitude	-122.483191°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.366g
1-second Period Mapped Spectral Acceleration[1], S_1	0.99g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.839g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.386g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.892g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.924g

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

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

On a preliminary basis, the planned structures may be supported on conventional shallow footings. On a preliminary basis, footings should bear on natural, undisturbed soil or engineered fill, be at least 15 inches wide, and extend at least 12 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.

On a preliminary basis, footings should be designed for 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.

7.3.2 Footing Settlement

On a preliminary basis, we estimate that the total static footing settlement will be on the order of less than ½-inch between footings. In addition, preliminary estimated dry sand settlements will be on the order of ½-inch or less. On a preliminary basis, total differential settlement is anticipated to be on the order of ½-inch or less.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed slabs-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. 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.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

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

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

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

8.3 EXTERIOR FLATWORK

8.3.1 Pedestrian Concrete Flatwork

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

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 10. The preliminary design R-value was chosen based on engineering judgment considering the variable surface conditions. We recommend additional laboratory testing be performed during the design-level investigation to confirm this value and revise as necessary.

Table 3: Asphalt Concrete Pavement Recommendations, Design R-value = 10

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.0	9.5
4.5	2.5	8.5	11.0
5.0	3.0	9.0	12.0
5.5	3.0	11.0	14.0
6.0	3.5	11.5	15.0
6.5	4.0	13.0	17.0

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

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

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 4: PCC Pavement Recommendations, Design R-value = 10

Allowable ADTT	Minimum PCC Thickness (inches)
4	5
57	5½
480	6

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

9.2.1 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 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Jefferson Union High School District specifically to support the design of the Westmoor Park Site project in Daly City, California. The opinions, conclusions, and preliminary 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.

Preliminary recommendations in this report are based upon the soil and ground water conditions encountered during our limited subsurface exploration. Preparation of a design-level investigation is anticipated to provide additional information and refine the preliminary recommendations presented herein. If variations or unsuitable conditions are encountered during the construction phase, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Jefferson Union High School District may have provided Cornerstone with plans, reports and other documents prepared by others. Jefferson Union High 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 11: REFERENCES

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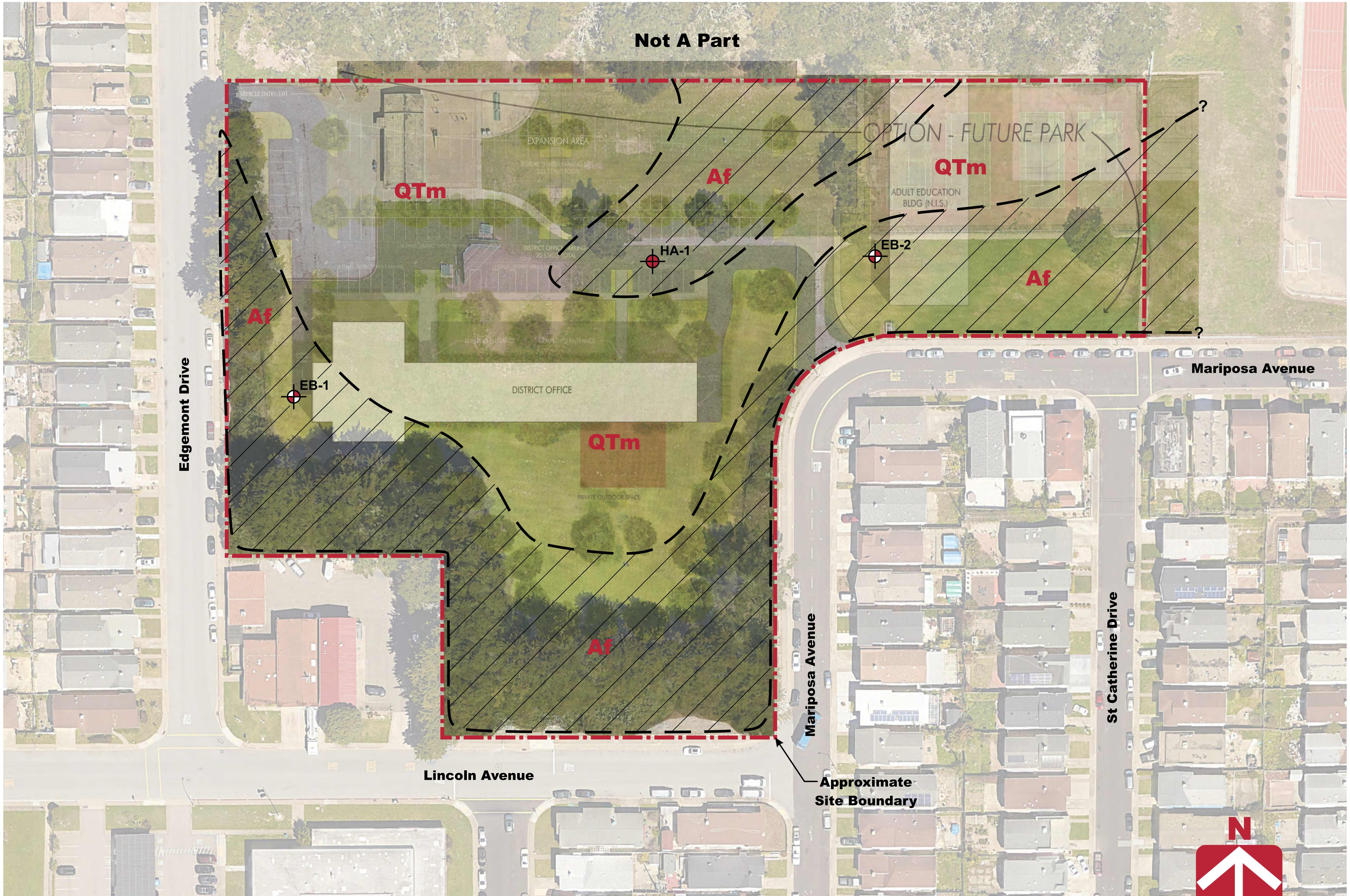
Vicinity Map

Westmoor Park
123 Edgemont Drive
Daly City, CA

Project Number	250-4-9
Figure Number	Figure 1
Date	May 2020
Drawn By	RRN



Geologic Units

Af Artificial fill

QTm Plio-Pleistocene Merced Formation

Legend

Approximate location of exploratory boring (EB)

Approximate location of hand auger boring (HA)

0 80 160

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 03/26/2018
Overlay by Forge, Site Plan - Concept 10 - A08.2, dated 0/26/2020

Project Number 250-4-9

Figure Number Figure 2

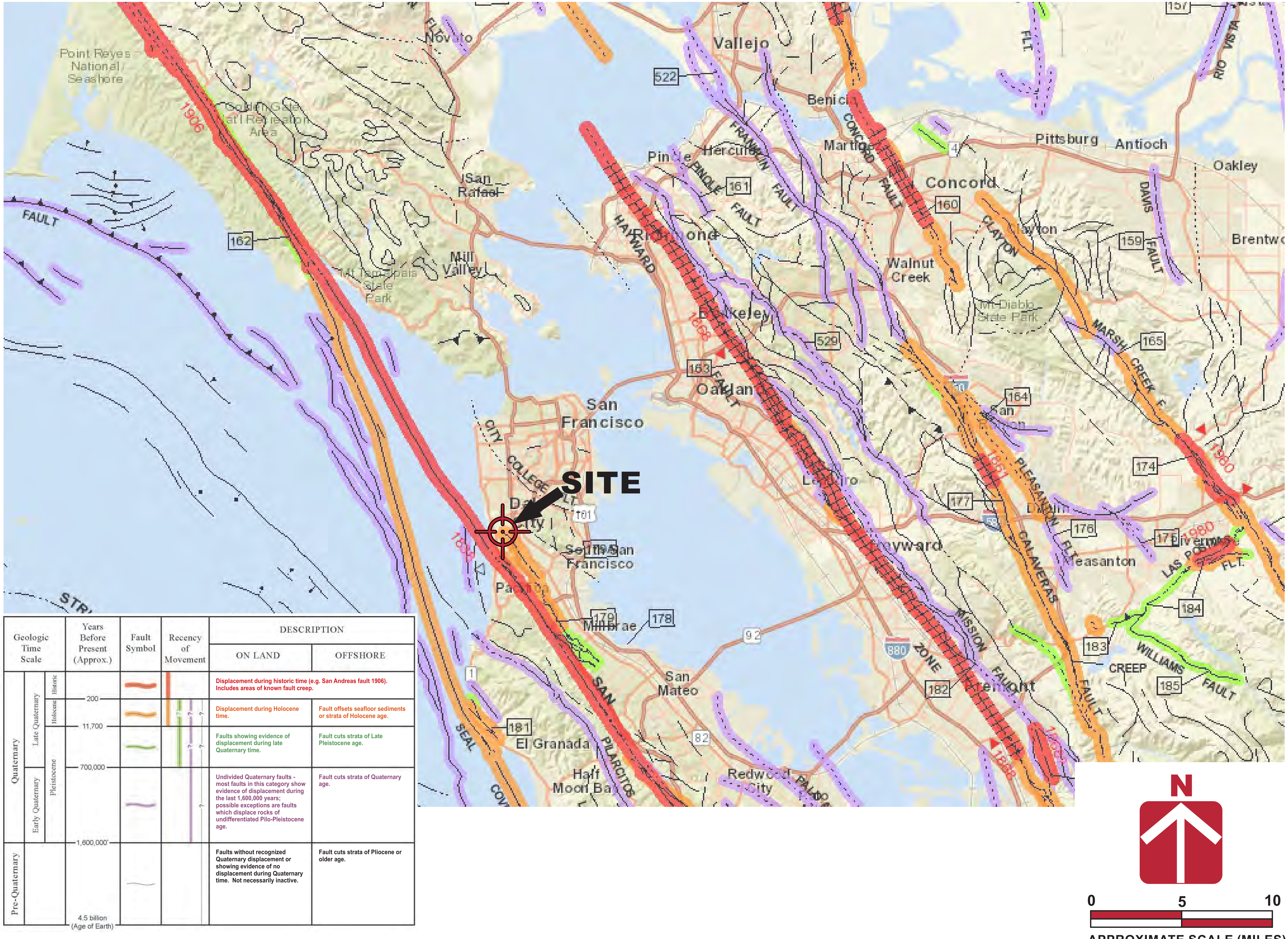
Date May 2020

Drawn By RRN

Site Plan and Geologic Map

Westmoor Park
123 Edgemont Drive
Daly City, CA

CORNERSTONE EARTH GROUP



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

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

Project Number 250-4-9

Figure Number Figure 3

Drawn By RRN

Date May 2020

Regional Fault Map

Westmoor Park
123 Edgemont Drive
Daly City, CA

CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and hand sampling equipment. Two 8-inch-diameter exploratory borings were drilled on May 1, 2020 to depths of 20 to 29½ feet and one 3-inch-diameter hand-auger boring was drilled on May 1, 2020 to a depth of 5½ feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were based on Google Earth®. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

OTHER MATERIAL SYMBOLS

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

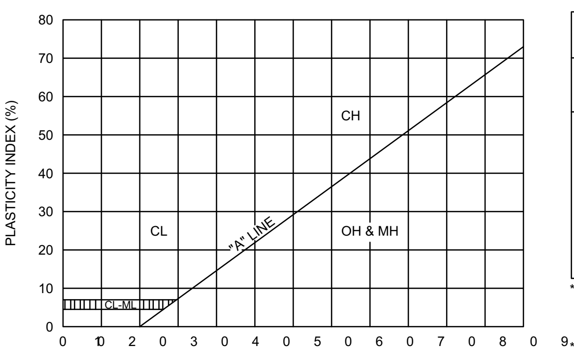
SAMPLER TYPES

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

ADDITIONAL TESTS

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

PLASTICITY CHART



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

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

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

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Westmoor High School
PROJECT NUMBER 250-4-9
PROJECT LOCATION Daly City, CA

DATE STARTED 5/1/20 **DATE COMPLETED** 5/1/20
GROUND ELEVATION 426 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.679895° **LONGITUDE** -122.483984°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
426.0	0	**Silty Sand (SM) [Fill]** medium dense, moist, brown with gray mottles, fine sand						
			38	MC-1B	109	12		
			26	SPT-2		8		9
421.0	5	**Poorly Graded Sand with Silt (SP-SM) [QTm]** medium dense to dense, moist, brown with gray mottles, finesand	38	MC-3B	101	9		
			40	SPT				
			36	SPT-4		8		
	10							
	15	becomes very dense	50/6"	MC-5B	97	7		
			72	SPT-6		5		7
406.0	20	Bottom of Boring at 20.0 feet.						
	25							
	30							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Westmoor High School
PROJECT NUMBER 250-4-9
PROJECT LOCATION Daly City, CA

DATE STARTED 5/1/20 DATE COMPLETED 5/1/20
GROUND ELEVATION 424 FT +/- BORING DEPTH 29.4 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.680204° LONGITUDE -122.482356°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
424.0	0	**Silty Sand (SM) [Fill]** medium dense, moist, gray with brown mottles, fine sand	30	MC-1B	111	12		
421.5		**Poorly Graded Sand with Silt (SP-SM) [QTm]** medium dense to dense, moist, brown with gray mottles, fine sand	35	SPT-2		9		
	5		67	MC-3B	108	6		
		becomes very dense	69	SPT-4		6		
	10							
			50/5"	MC-5B	102	5		
	15							
			50/2"	SPT-6		5		
	20							
			50/5"	MC-7B	97	7		
	25							
			50/5"	SPT-8		5		
394.6		Bottom of Boring at 29.4 feet.						
	30							

BORING NUMBER HA-1

CORNERSTONE EARTH GROUP

PROJECT NAME Westmoor High School
PROJECT NUMBER 250-4-9
PROJECT LOCATION Daly City, CA

DATE STARTED 5/1/20 **DATE COMPLETED** 5/1/20
GROUND ELEVATION 426 FT +/- **BORING DEPTH** 5.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.680197° **LONGITUDE** -122.482994°
DRILLING METHOD 3 inch diameter Hand Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
426.0	0	**Silty Sand (SM) [Fill]** moist, dark brown, fine sand, some fine subrounded gravel		GB-1		6		32
				GB				
423.0		**Poorly Graded Sand with Silt (SP-SM) [QTm]** moist, brown with gray mottles, fine sand		GB-3		10		
				GB				
420.5	5			GB-5		12		
		Bottom of Boring at 5.5 feet.						

Depth scale: 10, 15, 20, 25, 30

APPENDIX B: LABORATORY TEST PROGRAM

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

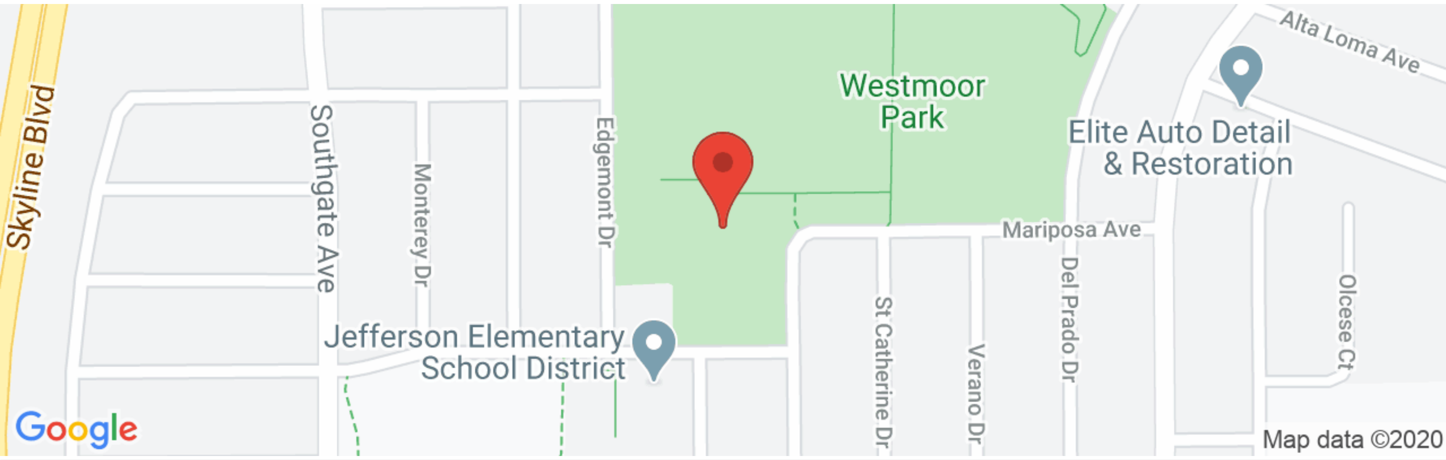
Moisture Content: The natural water content was determined (ASTM D2216) on 17 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

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

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

APPENDIX C: CBC 2019 SEISMIC DESIGN PARAMETER ON-LINE OUTPUT

Latitude, Longitude: 37.67997, -122.483191



Date	5/18/2020, 10:46:29 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	2.366	MCE_R ground motion. (for 0.2 second period)
S_1	0.99	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.839	Site-modified spectral acceleration value
S_{M1}	1.386	Site-modified spectral acceleration value
S_{DS}	1.892	Numeric seismic design value at 0.2 second SA
S_{D1}	0.924	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	1.012	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.214	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	2.663	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.01	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.366	Factored deterministic acceleration value. (0.2 second)
S1RT	1.124	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.291	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.99	Factored deterministic acceleration value. (1.0 second)
PGAd	1.012	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.885	Mapped value of the risk coefficient at short periods
C_{R1}	0.87	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER