

GEO-ENGINEERING SOLUTIONS, INC.

Geotechnical Engineering • Engineering Geology • Materials Testing

GEOTECHNICAL ENGINEERING STUDY

Newark Memorial High School-Athletic Field Improvements
39375 Cedar Blvd
Newark, California 94560

July 13, 2023

Prepared for:

Newark Unified School District
5715 Musick Avenue
Newark, California 94560

Prepared by:

GEO-ENGINEERING SOLUTIONS, INC.
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Project No. 142-1485

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July 13, 2023

Newark Unified School District
5715 Musick Avenue
Newark, California 94560

Attention: Ms. Marie Dela Cruz

Subject: Geotechnical Engineering Study
Newark Memorial High School-Field Replacement Projects
39375 Cedar Blvd, Newark, CA
Geo-Eng Project No. 142-1485

Dear Ms. Dela Cruz:

In accordance with your authorization, **Geo-Engineering Solutions, Inc. (Geo-Eng)** has completed a Geotechnical Engineering Study for the planned field replacement projects at the Newark Memorial High School Campus, in Newark, California. This report has been prepared in accordance with the requirements set forth in California Geological Survey Note 48 and the 2022 California Building Code. Transmitted herewith are the results of our findings, conclusions, and recommendations for the field replacement projects at the Newark Memorial High School. In general, the proposed improvements to the site are considered to be geotechnically and geologically feasible provided the recommendations of this report are implemented in the design and improvements of the project.

Should you or members of the design team have questions or need additional information, please contact the undersigned by email: nhaddad@geo-eng.net or eswenson@geo-eng.net or at (925) 433-0450. We greatly appreciate the opportunity to provide our services to the Piedmont Unified School District and to be involved in the design of this project.

Sincerely,

GEO-ENGINEERING SOLUTIONS, INC.



Nicolas B. Haddad, PE
Senior Geotechnical Engineer



Eric J. Swenson, GE, CEG
Principal Engineer and Geologist

TABLE OF CONTENTS

1.0 INTRODUCTION	1
1.1 Purpose and Scope	1
1.2 Site Description	1
1.3 Proposed Improvements	1
2.0 PROCEDURES AND RESULTS	2
2.1 Literature Review	2
2.2 Field Exploration	2
2.3 Laboratory Testing	3
3.0 GEOLOGIC AND SEISMIC OVERVIEW	4
3.1 Geologic Setting	4
3.2 Geologic Evolution of the Northern Coast Ranges	4
3.3 Regional Faulting and Tectonics	5
3.4 Historic Seismicity	7
4.0 SUBSURFACE CONDITIONS	8
4.1 Subsurface Soil Conditions	8
4.2 Atterberg Limits	8
4.3 Groundwater	8
5.0 GEOLOGIC HAZARDS	9
5.1 Seismic Induced Hazards	9
5.2 Other Hazards	11
6.0 CONCLUSIONS AND RECOMMENDATIONS	13
6.1 Conclusions	13
6.2 Seismic Parameters	13
6.3 Site Grading	14
6.4 Field Reconstruction Recommendations	15
6.5 Utility Trench Construction	16
6.6 Temporary Excavation Slopes	17
6.7 Foundation Recommendations	17
6.8 Site Pavements	18
6.9 Plan Review	20
6.10 Observation and Testing During Construction	20
6.11 Validity of Report	20
7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS	21
8.0 REFERENCES	22

TABLE OF CONTENTS (cont'd.)

FIGURES

- Figure 1 – Site Vicinity Map
- Figure 2 – Site Map and Boring Locations
- Figure 3 – Site Vicinity Geologic Map
- Figure 4 – Regional Fault Map
- Figure 5 – Seismic Hazard and AP Fault Zone Map
- Figure 6 – Flood Hazard Map
- Figure 7 – Percolation Test Locations

APPENDIX A

FIELD EXPLORATION

- Key to Boring Log Symbols
- Boring Logs

APPENDIX B

LABORATORY TEST RESULTS

- Atterberg Limit's Results
- Grain Size Distribution
- Unconfined Compression Test

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of our work was to prepare a Geotechnical Engineering Study, evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed field replacement projects at the Newark Memorial High School. We have provided specific recommendations regarding suitability and geotechnical concerns relative to the proposed structural design.

The scope of this study included the field exploration, laboratory testing, engineering analysis of the collected samples and test results, and preparation of this report. The conclusions and recommendations presented in this report are based on the limited samples collected and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an in-depth assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The Newark Memorial High School is located at 39375 Cedar Blvd in Newark, California, as shown on the attached Figure 1-*Site Vicinity Map*. The Newark Memorial High School is bordered by Cedar Blvd to the north, Balentine Drive to the east, Cherry Street to the south, and residential developments to the west.

The topography of the site is generally flat, with an approximate elevation of +26 feet based on Google Earth elevations. The average geographic coordinates of the site, used in our engineering analyses, are 37.5191 degrees north latitude and -122.0030 degrees west longitude.

1.3 Proposed Improvements

We understand that, based on the information provided by the District that the proposed improvements will consist of the renovation of the existing track and field and conversation from natural grass field to an artificial turf. We also understand that a new track will be constructed. We anticipate that some minor structures such as fences or score boards may be designed, and we are therefore providing foundation design recommendations for this project. It is assumed that there will also be associated minor site improvements such as site grading, landscaping and utilities.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area, and previous geotechnical studies performed by others for projects in the site vicinity were reviewed. These included United States Geological Survey (USGS), California Geological Survey (CGS), and other online resources, and other applicable government and private publications and maps, as included in the References section.

2.2 Field Exploration

A total of 3 borings were drilled at the site on May 3, 2023 at the approximate locations shown on Figure 2-*Site Map and Boring Locations*. The borings were drilled by California Geotech Services, LLC to a maximum depth of approximately 30 feet below existing ground surface across the project site using a truck mounted, CME-55 drill rig equipped with four-inch diameter solid flight augers.

A Geo-Eng Staff Engineer visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners. A two-inch outside diameter Standard Penetration Test (SPT) sampler was used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a 140-pound safety hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All of the blow counts recorded using Modified California split spoon samplers in the field were converted to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 assuming an inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts. Bulk samples were obtained in the upper few feet of the borings from the auger cuttings as needed.

The boring logs with descriptions of the various materials encountered in each boring, the penetration resistance values, and some of the laboratory test results are presented in Appendix A. The ground surface elevations indicated on the soil boring logs were determined using project survey data. Actual surface elevations at the boring locations may differ slightly than indicated. The locations of the borings should only be considered accurate to the degree implied by the means and methods used to define them.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented on the boring logs, and are included in the appendices. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2937 and D2216) – In-situ density and/or moisture tests were conducted on several samples to measure the in-place dry density and moisture content of the subsurface materials at the tested sample locations and depths. These properties provide information for evaluating the physical characteristics of the subsurface soil. Test results are presented in Appendix B and shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) - Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, including evaluating the expansive characteristics of the soil, and determining the soil type according to the USCS. Four tests were performed, and the test results are presented in Appendix B and summarized on the boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Fines Content (ASTM D422 and D1140) - Sieve analysis or fines content (minus No. 200 sieve) tests were conducted on several selected samples to measure the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented on the boring logs or in Appendix B.

Unconfined Compressive Strength (ASTM D2166) – Unconfined compressive strength tests were run on several liner samples to obtain strength parameters for use in foundation and retaining wall design.

3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1 Geologic Setting

The site is located in the northern Coast Ranges geomorphic province of California. The Coast Ranges lies between the Pacific Ocean to the west and the Great Valley Geomorphic Province to the east. It extends from the Transverse Ranges in southern California to the Oregon border and is comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (Page, 1966).

Locally, the site is located between the San Francisco Bay to the west and the East Bay hills to the east. The subject property is located in a generally flat area that overlies alluvial gravel, sand, and clay of Holocene alluvium (Dibblee and Minch, 2007), Figure 3-Site Vicinity Geologic Map.

3.2 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time generally through east-dipping reverse faults.

Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults including the San Andreas, Hayward, and Calaveras. The project site is situated 18 miles east of the San Andreas fault and about 3 mile west of the Hayward Fault. The Hayward Fault is the closest fault to the subject property with Holocene movement. The site is not mapped within an Alquist-Priolo Earthquake Fault Zone.

3.3 Regional Faulting and Tectonics

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone located about 18 miles (29 km) west of the site. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Hayward, and Calaveras faults in the greater San Francisco Bay area.

The subject property is in a seismically active region dominated by the Hayward Fault, which is located approximately 3 miles (4.8 km) to the east of the subject property. Other major faults within the vicinity of the site include the San Andreas Fault, located about 18 miles (29 km) west of the site, the northern portion of the Calaveras Fault, located 9 miles (14.5 km) east of the site. Other active faults include the Mt. Diablo Thrust Fault, located approximately 27.5 miles (44.25 km) northeast of the site, the Concord-Green Valley Fault, located approximately 32 miles (51.5 km) north of the site, and the Greenville-North (Clayton-Marsh Creek) Fault, located approximately 42 miles (67.6 km) northeast of the site. The site location relative to active and potentially active faults in the San Francisco Bay Area is shown on Figure 4-*Regional Fault Map*. A discussion of these faults, follows.

3.3.1 Green Valley Fault

The north to northwest trending Green Valley Fault is thought to be an extension of the active Concord Fault, which extends from the approximate central Walnut Creek and Concord border, northward into the Green Valley Fault. The Green Valley Fault extends northward from Suisun Bay up to just west of Lake Curry, northeast of Napa. The site is located 32 miles (22.5 km) south of the Green Valley Fault. The slip rate of the Green Valley Fault (south segment) is estimated to be about 5 mm/year and has been assigned a moment magnitude (M_{max}) of 6.2 (CGS, 2002). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a three percent probability of at least one magnitude 6.7 or greater earthquake before 2037 occurring on the Concord – Green Valley Fault system (USGS, 2008).

3.3.2 Greenville-Marsh Creek-Clayton Fault

The northwest-trending Greenville-Marsh Creek-Clayton Fault system lies within the Diablo Range and extends from Arroyo Mocho southeast of Livermore, across the eastern edge of the Livermore Valley and the northeast edge of Mt. Diablo into the Clayton Valley area. On the east side of Mt. Diablo, the fault has been referred to as the Marsh Creek Fault, connecting to the Clayton Fault in the Clayton Valley. The fault zone has been divided (CGS,

2002) into two segments, Greenville-North and Greenville-South. The site is located 42 miles (67.6 km) southwest of the Greenville-Marsh Creek-Clayton Fault. The slip rate of the Greenville-North segment is estimated to be about 2 mm/year and has been assigned a moment magnitude (M_{max}) of 6.6 (CGS, 2002). The Working Group on California Earthquake Probabilities (WGCEP2007) has estimated that there is a three percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the Greenville Fault (USGS, 2008).

3.3.3 Hayward Fault

The Hayward Fault trends northwesterly and extends from the Milpitas area to San Pablo Bay, totaling about 55 miles (88 km) in length. The Hayward Fault has been divided into two main segments, the Northern and Southern segments. The Rodgers Creek Fault, considered as a possible extension of the Hayward Fault, extends northward from beneath San Pablo Bay up to near Healdsburg, where it is aligned with the Healdsburg Fault zone, currently considered to be inactive. The site is located approximately 3 miles (4.8 km) west of the northern segment of the Hayward Fault. The slip rate on this segment of the Hayward Fault is estimated to be about 9 mm/year and has been assigned a moment magnitude (M_{max}) of 6.4 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a 31 percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the Hayward – Rodgers Creek Fault system.

3.3.4 Mt. Diablo Thrust Fault

The northwest trending Mt. Diablo Fault extends from near the Alamo and Walnut Creek border along Highway 680, extending southward to the vicinity of Cottonwood Canyon, in northern Dublin. The Mt. Diablo Fault is not currently mapped as active by CGS, but fault coordinates have been published. The fault is different from the majority of Bay Area faults in that it is not a strike-slip fault exhibiting horizontal plate motion, but rather a “blind thrust” fault. A blind thrust fault is a compressional fault without apparent surface expression. In the case of the Mt. Diablo Fault, the compression is caused by the differential horizontal motions of the Concord and Greenville faults which abut Mt. Diablo. The fault locations for this blind thrust fault are inferred. The 1994 Northridge earthquake is a recent example of faulting on a blind thrust fault. The site is located 27.5 miles (44.25 km) west of the Mt. Diablo Fault. The slip rate of the Mt. Diablo Thrust Fault is estimated to be about 2 mm/year and has been assigned a moment magnitude (M_{max}) of 6.7 (CGS, 2002). The Working Group on California Earthquake Probabilities (WGCEP2007) has estimated that there is a one percent probability of at least one magnitude 6.7 or greater earthquake on the Mt. Diablo Thrust Fault before 2037 (USGS, 2008).

3.3.5 Calaveras Fault

The Calaveras Fault trends northwesterly and extends from near Hollister to north of the Danville area, totaling about 76 miles (123 km) in length. The Calaveras Fault has been divided into three segments, the Northern, Central, and Southern segments. The site is located 9 miles (14.5 km) west of the Calaveras Fault. The slip rate on the north segment of the Calaveras Fault is estimated to be about 6 mm/year and has been assigned a moment magnitude (M_{max}) of 6.8 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a seven percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the Calaveras Fault.

3.3.6 San Andreas Fault

The northwest-trending San Andreas Fault runs along the western coast of California is about 800 miles (1,287 km) in length, running from the north near Point Arena to the Salton Sea area in southern California (Jennings, 1994). The fault zone has been divided into 11 segments. The site is located about 18 miles (29 km) east of the Peninsula segment. The slip rate on the Peninsula segment of the San Andreas Fault is estimated to be about 17 mm/year and has been assigned a moment magnitude (M_{max}) of 7.1 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a 21 percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the San Andreas Fault.

3.4 Historic Seismicity

As discussed above, the San Francisco Bay Area is subject to a high level of seismic activity. Between the year 1900 to present time there were an estimated five earthquakes exceeding a Richter magnitude of 6.0 within an approximate 100-mile radius of the site, with two exceeding 6.5 and one exceeding 7.5. In addition, there have been four major Bay Area Earthquakes from the year 1800 to 1900; this includes earthquakes in 1836 and 1868 on the Hayward-Rodgers Creek Fault, one in 1861 on the Calaveras Fault, and one in 1838 on the San Andreas Fault. Notable Bay Area earthquakes from 1900 to present time include the 1906 and 1989 earthquakes on the San Andreas Fault.

4.0 SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

During our subsurface exploration program, we investigated the subsurface materials and evaluated soil and rock conditions to a maximum depth of 30 feet in the borings performed for this study. From the collected data, we conclude that the project site is generally underlain by stiff to medium stiff silty and sandy clay to a depth of 22 feet below ground surface, underlain by a layer of loose clayey sand to about 27 feet below ground surface, underlain by very stiff silty clay to the maximum depth explored of 30 feet below ground surface.

4.2 Atterberg Limits

- A soil sample of the near surface fine grained material from boring B-1 at 4 feet below ground surface was tested for Atterberg Limits, with measured Liquid Limit (LL) of 31, Plastic Limit (PL) of 15, and corresponding Plasticity Index (PI) of 16.
- A soil sample of the near surface fine grained material from boring B-2 at 2 feet below ground surface was tested for Atterberg Limits, with measured LL of 25, PL of 15, and corresponding PI of 10.
- A soil sample of the near surface fine grained material from boring B-3 at 2 feet below ground surface was tested for Atterberg Limits, with measured LL of 27, PL of 14, and corresponding PI of 13.
- A soil sample of fine-grained material from boring B-3 at 14 feet below ground surface was tested for Atterberg Limits, with measured LL of 30, PL of 16, and corresponding PI of 14.

Based on these results and our experience with the area, we judge that the near-surface soils are considered to have a low to moderate plasticity and a low to moderate expansion (shrink/swell) potential. Additional details of the soils encountered in the exploratory borings are included in the boring logs presented in Appendix A.

4.3 Groundwater

Groundwater was encountered during our field exploration in borings B-1 and B-3 at 7 feet below ground surface and in boring B-2 at 12 feet below ground surface. Based on the Seismic Hazard Zone Report for the Newark Quadrangle prepared by the California Geologic Survey (CGS) historic ground water in the vicinity of the subject site, groundwater is expected to be on the order of 10 feet below the ground surface. For design purposes, assume a groundwater elevation of 7-feet. We note that the borings may not have been left open for a sufficient period to establish equilibrium groundwater conditions. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement, fault ground rupture and fault creep, dam inundation, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. Nonetheless, potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site may experience moderate to strong ground shaking from a major earthquake originating from one or more of the close or major Bay Area faults such as the Hayward (approximately 3 miles from the site) or Calaveras (approximately 9 miles from the site) or San Andreas approximately (18 miles from the site).

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, because of the higher inter-granular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances. Lateral spreading can cause ground cracking and settlement.

The site is mapped by the CGS as being in a liquefaction hazard zone, as shown on Figure 5-*Seismic Hazard and AP Fault Zone Map*. Within the upper 30 feet explored, the site was underlain by stiff to medium stiff silty and sandy clay with high plasticity clay (CL) to 30 feet below ground surface. Groundwater was encountered in borings B-1 and B-3 at 7 feet below ground surface and in boring B-2 at 12 feet below ground surface. Based on the clay soil characteristics we anticipate the liquefaction potential of the site subsurface soils to be very low. Due to the very stiff nature of the subsurface soils, the lack of liquefaction-susceptible subsurface soils, the potential for lateral spreading is also considered to be very low to nil.

5.1.3 Dynamic Compaction (Settlement)

Dynamic compaction is a phenomenon where loose, relatively clean sandy soil located above the water table is densified from vibratory loading, typically from seismic shaking or vibratory equipment. The underlying materials consist primarily of silty and sandy clay. Therefore, in our opinion, dynamic settlement and/or any potential effect of dynamic settlement on the proposed construction is not expected to be significant.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones, or Earthquake Fault Zone surrounding faults or fault segments judged to be sufficiently active, well-defined, and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently mapped or within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). The closest Earthquake Fault Zones are associated with the Hayward Fault, about 3 miles east of the site, as shown on Figure 4. Since the site is not within or near an A-P Earthquake Fault Zone, the potential for fault ground rupture and fault creep hazards are judged to be very low.

5.1.5 Tsunamis and Seiches

Tsunamis are long-period sea waves generated by seafloor movements from submarine earthquakes or volcanic eruptions that rapidly displace large volumes of water. Coastal communities along the Pacific Ocean are particularly susceptible to such phenomena. The project site is located at an elevation of about +100. Therefore, the potential for tsunami inundation at the site is considered to be nil.

Earthquake-induced waves generated within enclosed bodies of water are called seiches. Such waves may overtop dam embankments or extreme cases, cause dam failure, and in either case result in downstream inundation. The site is not within the downstream drainage area of any significant body of water. Therefore, the site is not considered to be susceptible to seiches.

5.2 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include ground failure and subsidence, consolidation settlement, landslides under static loading conditions, expansive and collapsible soils, flooding, naturally occurring asbestos (NOA) and soil erosion. These are discussed and evaluated in the following sections.

5.2.1 Ground Cracking and Subsidence

Withdrawal of groundwater and other fluids (i.e. petroleum and the extraction of natural gas) from beneath the surface has been linked to large-scale land subsidence and associated cracking on the ground surface. Other causes for ground cracking and subsidence include the oxidation and resultant compaction of peat beds, the decline of groundwater levels and consequent compaction of aquifers, hydro-compaction and subsequent settlement of alluvial deposits above the water table from irrigation, or a combination of any of these causes. Determining the impacts from subsidence on the project is beyond the scope of this study however, subsidence generally impacts a region, and should not produce excessive differential settlement in a single location. Local and regional locations prone to subsidence generally subside equally over time.

5.2.2 Consolidation Settlement

Consolidation is the densification of soil into a denser arrangement from additional loading, such as from new fills or foundation loads. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a load. Consolidation of soft and loose soil layers and lenses can cause settlement of the ground surface or buildings. Based on stiff to hard subsurface site soils and bedrock, and lack of a near-surface groundwater table, the near-surface soils are considered to have a minimal potential for consolidation settlement.

5.2.3 Landslides

Landslides can occur under a variety of loading conditions, including both static and seismic, but involve sloping ground. As shown on Figure 5 the subject site is not adjacent to an “earthquake-induced landslide zone.” The site and immediate vicinity are moderately sloping and do not exhibit landslide features as determined by our geologic site reconnaissance and literature review. Therefore, except for possible minor surface sloughing of the perimeter cut slopes after sustained rainfall events, the site is not susceptible to land sliding.

5.2.4 Expansive and Collapsible Soils

Soils observed and/or sampled from the more clayey soils in the upper five feet of the site were observed to be of primarily low plasticity. Lab tests indicated liquid limits ranging from 25 to 31, with corresponding plasticity indices ranging from 10 to 16. Such soils typically exhibit a low to moderate expansion potential with moisture variation. Therefore, special measures to mitigate the potential effects of expansive soils are not expected to be required for the project.

5.2.5 Flooding

The site is located in a mapped area of flooding hazard, as shown on Figure 6-*Flood Hazard Map*, based on the FIRM (2009) map produced for the site vicinity. The site is located in a dotted Zone X, which delineates areas determined to be inside the 0.2% Annual Chance Flood Hazard.

5.2.6 Soil Erosion

Present construction techniques and agency requirements have provisions to limit soil erosion and resultant siltation during construction. These measures will reduce the potential for soil erosion at the site during the various construction phases. Long-term erosion at the site will be reduced by designing landscaping and/or hardscape areas such as parking lots and walkways with appropriate surface drainage facilities.

5.2.7 Naturally Occurring Asbestos (NOA)

The project site is not located near a mapped geographic ultramafic rock unit (GURU) or serpentinite, as shown on Figure 3, it is our opinion that the subject site should be exempt from any special regulation or mitigation procedures relating to NOA before and during construction.

5.2.8 Other Geologic Hazards

Due to the site's location and geology, subsurface soil conditions, groundwater levels and land use factors, the site is not subject to the potential geologic hazards of loss of mineral resources, volcanism, cyclic softening of soils or loss of unique geologic features.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

6.1 Conclusions

The site is considered suitable from a geotechnical and geologic perspective for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that will need to be addressed at this site are summarized below.

Field Drainage and Groundwater – Groundwater at the site is anticipated to be irregular and variable based off our previous experience in the project area and may impact construction processes such as the new field, excavations, foundations, and utility trenches. Proper groundwater mitigation techniques as described in this report may need to be put in place if groundwater is encountered during construction. We understand that extensive reconstruction of the subdrain systems below the field will take place as part of this project and detailed recommendations are provided in the following sections of this report.

Winter Construction – In the event grading occurs in the winter rainy season, appropriate erosion control measures will be required, and foundation excavations should be considered. Winter rains may also impact foundation excavations and underground utilities.

Seismic Considerations – The site is located within a seismically active region and should be designed to account for earthquake ground motions, as described in this report.

6.2 Seismic Parameters

The subject site is located within a seismically active region and should be designed to account for earthquake ground motions as described in this report. Based on the subsurface conditions encountered and our evaluation of the geology of the site, Site Class “D”, representative of stiff soil averaged over the uppermost 100 feet of the subsurface profile would be appropriate for this site.

For seismic analysis of the proposed site in accordance with the seismic provisions of the 2022 California Building Code (CBC), we recommend the following seismic ground motion values be used for design shown in Table 4, which are based on procedures outlined in ASCE 7-16 Section 11.4 and Table 11.4-2 of Supplement 1. ASCE 7-16 Section 11.4.8 states that a site-specific ground motion hazard analysis should be performed for all structures on

Site Class D soils with S_1 greater than or equal to 0.2, unless the exceptions outlined in Section 11.4.8 are followed and the seismic response coefficient is properly modified during design. A site-specific ground motion hazard analysis was not performed for this site and is outside the scope of this report. If a site-specific ground motion hazard analysis is required for this project or if the project is designed under a different building code than CBC 2022, we should be notified so that we may properly update the seismic design parameters.

Table 4: Seismic Design Parameters Based on 2022 CBC (per ASCE 7-16)

Item	Value	2022 CBC Source ^{R1}	ASCE 7-16 Table/Figure ^{R2}
Site Class	D	Table 1613A.3.2	Table 20.3-1
Mapped Spectral Response Accelerations			
Short Period, S_5	1.818 g		Figure 22-1
1-second Period, S_1	0.692 g		Figure 22-2
Site Coefficient, F_a	1.2	Table 1613A.3.3(1)	Table 11.4-1
Site Coefficient, F_v^*	1.7	Table 1613A.3.3(2)	Table 11.4-2
MCE (S_{MS})	2.182 g	Equation 16A-37	Equation 11.4-1
MCE (S_{M1})	1.176 g	Equation 16A-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.454 g	Equation 16A-39	Equation 11.4-3
1-second Period, S_{D1}^{**}	0.784 g	Equation 16A-40	Equation 11.4-4
Peak Ground Acceleration (PGA_M)	0.917 g	-	Equation 11.8-1

R1: California Building Standards Commission (CBSC), "California Building Code," 2022 Edition.

R2: U.S. Seismic "Design Maps" Web Application, <https://seismicmaps.org/>

* F_v are based off Table 11.4-2 from the ASCE 7-16 Supplement 1

**The above design spectral response acceleration parameters may only be used provided that the exception outlined in section 11.4.8 of ASCE 7-16 is met.

6.3 Site Grading

6.3.1 General Grading and Material Requirements

Site grading is generally anticipated to consist of utility trench excavation and backfills, limited preparation of supporting subgrades for site pavements and hardscape overlying utilities; and placement of aggregate base (baserock) sections for hardscape and pavements overlying utilities.

On-site soils having an organic content of less than three percent by weight can be reused as fill as approved by the Geotechnical Engineer. Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of environmental contaminants, organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. Import fill materials should be approved by the Geotechnical Engineer prior to use on site.

6.3.2 Project Compaction Recommendations

The following table summarizes the recommended minimum compaction requirements for this project. Not all soils, aggregates, and scenarios listed below may be applicable for this project. Specific grading recommendations will be discussed individually within applicable sections of this report.

Table 5: Project Compaction Recommendations

Description	Percent Relative Compaction	Minimum Percent Above Optimum Moisture Content
AC or Concrete Pavement, Subgrade, Upper 6"	95	3 to 5
AC or Concrete Pavement, Onsite Soil or Fill	90	2
AC or Concrete Pavement, Class 2 Baserock	95	2
AC or Concrete Pavement, Treated Soil, Subgrade	93	2
Concrete Flatwork, Class 2 Baserock	90	2
Concrete Flatwork, Subgrade Soil	90	3 to 5
Underground Utility Trench Backfill	90	2
Underground Utility Trench Backfill - Landscape Areas (not including areas below flatwork)	85	2
Underground Utility Trench Backfill, Clean Sand	95	4
Underground Utility Trench Backfill, Upper 3' Feet below Existing Pavement Sections or 6" below New Pavement Sections	95	2

Fill materials should be properly moisture conditioned in accordance with Table 5 as determined using ASTM D-1557 and placed in uniform loose lifts not to exceed eight inches. Smaller lifts may be necessary to achieve the minimum required compaction using lighter weight compaction equipment. It should be noted that the use of on-site soils for fill will require moisture conditioning (drying or wetting). Moisture conditioning may be difficult to achieve during cold, wet periods of the year, or during extreme temperatures and after precipitation events.

6.4 Field Reconstruction Recommendations

We conducted two infiltration tests as shown on Figure 7, Percolation Testing Map. The infiltration tests were completed using a double ring infiltrometer, consisting of two concentric metal rings. During testing the rings were driven two inches into the ground and filled with water three times in order to saturate the soil. The drop in water level in the interior ring was measured in 15-minute increments over a 60-minute period and used to calculate an infiltration rate. The infiltration rate is determined as the amount of water per surface area and time unit that penetrates the soil. Measurements were taken until the rate at which the water level dropped became consistent.

The table below summarizes the results of the percolation/infiltration test performed at the subject site.

Table 6: Percolation Test Results

Location	Depth (in)	Soil Description	Infiltration (in/hr)
P-1	6"	Brown Silty Clay	0.125
P-2	6"	Brown Silty Clay	0.25

The project civil engineer and landscape architect should review the results and confirm if they meet the project drainage requirements. The excavated drainage layer should be replaced completely with Class 2 permeable material. The fill should be placed in lifts of 8-inches and compacted to a minimum of 90% relative compaction per ASTM D1557. The field should be designed with a crowned grade to promote surface drainage from the center to a surrounding perimeter trench drain system. In addition, a new subdrainage perforated pipe network should be constructed to drain the “bathtub” of the field as the underlying native soils have low permeability and should not be counted on for infiltration. The system should be designed by a civil engineer or landscape architect experienced in the drainage design and system.

6.5 Utility Trench Construction

Utility trenches may be backfilled with approved native soil above the utility bedding and shading materials compacted to the recommended compaction presented in Table 5. If rocks larger than four inches in maximum size are encountered, these should be removed from the fill prior to placement in the utility trenches. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction and as recommended by the pipe manufacturers. Jetting of trench backfill is not recommended.

Pea gravel, rod mill (pea gravel with sand), or other similar self-compacting material should NOT be utilized as trench backfill at the site. This material may act as a conduit for subsurface moisture migration. Utility trenches ideally should be completely sealed/cut-off with concrete, clayey soil, sand-cement slurry, or controlled density fill (CDF) where the utility enters the building under the perimeter foundation. This would reduce the potential for migration of water beneath the building through the shading material in the utility trench.

If rain is expected and the trench will remain open, the bottom of the trench may be lined with one to two inches of gravel. This would provide a working surface in the trench bottom. The trench bottom may have to be sloped to a low point to pump the water out of the trench.

6.6 Temporary Excavation Slopes

The Contractor should incorporate all appropriate requirements of OSHA/ Cal OSHA into the design of any temporary construction slopes used during construction. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, the subsurface materials in the areas of the site where excavation may take place may be assumed to consist of stiff clay categorized as OSHA Type B with temporary slope inclination of no steeper than 1:1 (horizontal to vertical). This maximum slope ratio is assumed to be uniform from top to toe of the slope. The type of slope material and actual temporary construction slopes should be confirmed during construction by a competent engineering geologist responsible to the grading contractor.

If temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes, and that such temporary slopes are protected from excessive drying or saturation during construction. In addition, adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

6.7 Foundation Recommendations

6.7.1 General

We anticipate that the foundations for the new scoreboards and fences will consist of drilled piers. Recommendations for drilled pier foundations are discussed below in section 6.7.2. We understand that there might be several ancillary structures constructed as part of the athletic field improvements. Therefore, we are also including recommendations for shallow foundations for such structures in section 6.6.3.

6.7.2 Drilled Pier Foundation: Scoreboard and Fences

The foundation for the proposed scoreboards and fences may consist of drilled pier foundations, deriving their vertical supporting capacity through skin friction between the side surfaces of the foundations and the adjacent soil. The piers should have a minimum diameter of 24 inches, with an overall minimum depth of 10 feet below finished grade. For design purposes, the allowable skin friction for gravity loads may be assumed to be 750 psf for the portion of pier embedded in native soils, with the upper two feet of the pier to be neglected. This value may be increased by one-third for seismic or transient loads. Uplift loads should be limited to 0.8 times these values. For piers situated adjacent to or on slopes, the portion of pier with horizontal cover less than 10 feet, measured from outside perimeter of the pier to the slope surface should be neglected in computing vertical capacity.

For a 4-foot post the depth will be based on the spacing of the posts and design wind loads. We recommend that the fence posts be designed in accordance with section 1807 of the CBC and the soil design values type 5 Table 1806.2 (Clay, sandy Clay) be used for design. Vertical foundation pressure 1500 psf, lateral bearing pressure 100 psf/ft and vertical cohesion of 130 psf. The Chain Link Fence Manufacturers Institute guideline (<https://www.chainlinkinfo.org/wp-content/uploads/2016/06/WLG-Updated-61316.pdf>) should be used for the design of the chain link fence.

Lateral resistance for drilled pier foundations may be determined for onsite soils using an allowable passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation for lateral load resistance against the sides of foundations perpendicular to the direction of loading where the foundation is poured neat against undisturbed material (i.e., native soils, engineered fills or existing fills). For pier foundations, passive pressure can be assumed to act across two times the pier diameter. Geo-Eng personnel should be retained to observe and confirm that soil or bedrock encountered during footing excavations, prior to formwork and reinforcing steel placement, is consistent with the assumptions of this report. If unsuitable soil or bedrock is present, the excavation should be deepened until suitable supporting material is encountered.

6.8 Site Pavements

Recommendations for the design of asphalt concrete pavement sections were developed in accordance with the procedures outlined in Caltrans High Design Manual. The Caltrans Manual utilizes a combination of loading as expressed as a Traffic Index and soil subgrade condition expressed as an R-value for the design of the pavement thicknesses. For this project, we have assumed an R-value of 5 for pavement design. We recommend that during the initial grading process as-built R-value tests be performed to better refine and value engineer this design. The project civil engineer should select the appropriate design TI based on traffic loading design. We anticipate that

lower level traffic such as parking lot areas would have a TI on the order of 5.0. The following table provides recommended design sections based on our assumptions.

Table 7: Flexible Pavement Design Alternatives

Load Application (TI)	Asphalt Concrete (in.)	Aggregate Base (in.)	Total Section (in.)
Parking Stall (4.0)	3.0	8.0	10.0
Parking Drive (5.0)	3.0	11.0	13.0
Main Driveway (6.0)	3.5	13.0	16.5

The pavement section design assumed a 20-year design life and a reliability factor of 75%. Factors such as surface and subsurface drainage influence the overall life of a pavement section. Additionally, all pavements require regular maintenance to provide this design life. This maintenance includes crack sealing, slurry, and overlay at appropriate times in accordance with a predefined pavement management plan.

As an alternative to HMA flexible pavements, rigid PCC pavement can also be used. We recommend that for light vehicular load conditions a section of 6-inches of concrete over 6-inches of class 2 aggregate base. We recommend that for heavier vehicular loads, such as trucks, a section of this pavement should be designed as Jointed Plain Concrete Pavement with lateral contraction joints at lane widths and longitudinal contraction joints no more that 20-feet center to center. Joint reinforcement should be designed in accordance with the latest AASHTO Guide for Design of Pavement Structures.

Pavement subgrade and AB should be compacted in accordance with the recommendations in section 6.0. In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

AB used in pavement sections should be compacted to a minimum 95 percent relative compaction (ASTM D1557) and should be firm and unyielding at the time of asphalt concrete placement. The subgrade beneath pavements should be scarified to a minimum depth of 8 inches and moisture conditioned and re-compacted to 95% relative compaction.

6.9 Plan Review

It is recommended that Geo-Eng be provided the opportunity to review the foundation, grading, and drainage plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and the incorporation of these recommendations into the project plans and specifications.

6.10 Observation and Testing During Construction

It is recommended that Geo-Eng be retained to provide observation and testing services during site preparation, site grading, utility construction, and foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

6.11 Validity of Report

This report is valid for three years after publication. If construction begins after this time period, Geo-Eng should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geo-Eng should be notified to determine if additional recommendations are required. Additionally, if Geo-Eng is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid since Geo-Eng's geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are similar to the subsurface conditions revealed during construction. Geo-Eng's involvement should include foundation and grading plan review; observation of foundation excavations; grading observation and testing; testing of utility trench backfill and retaining wall backfill as applicable to the project; and subgrade preparation in flatwork areas.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, Geo-Eng should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions 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. Accordingly, the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geo-Eng after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geo-Eng should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geo-Eng be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geo-Eng will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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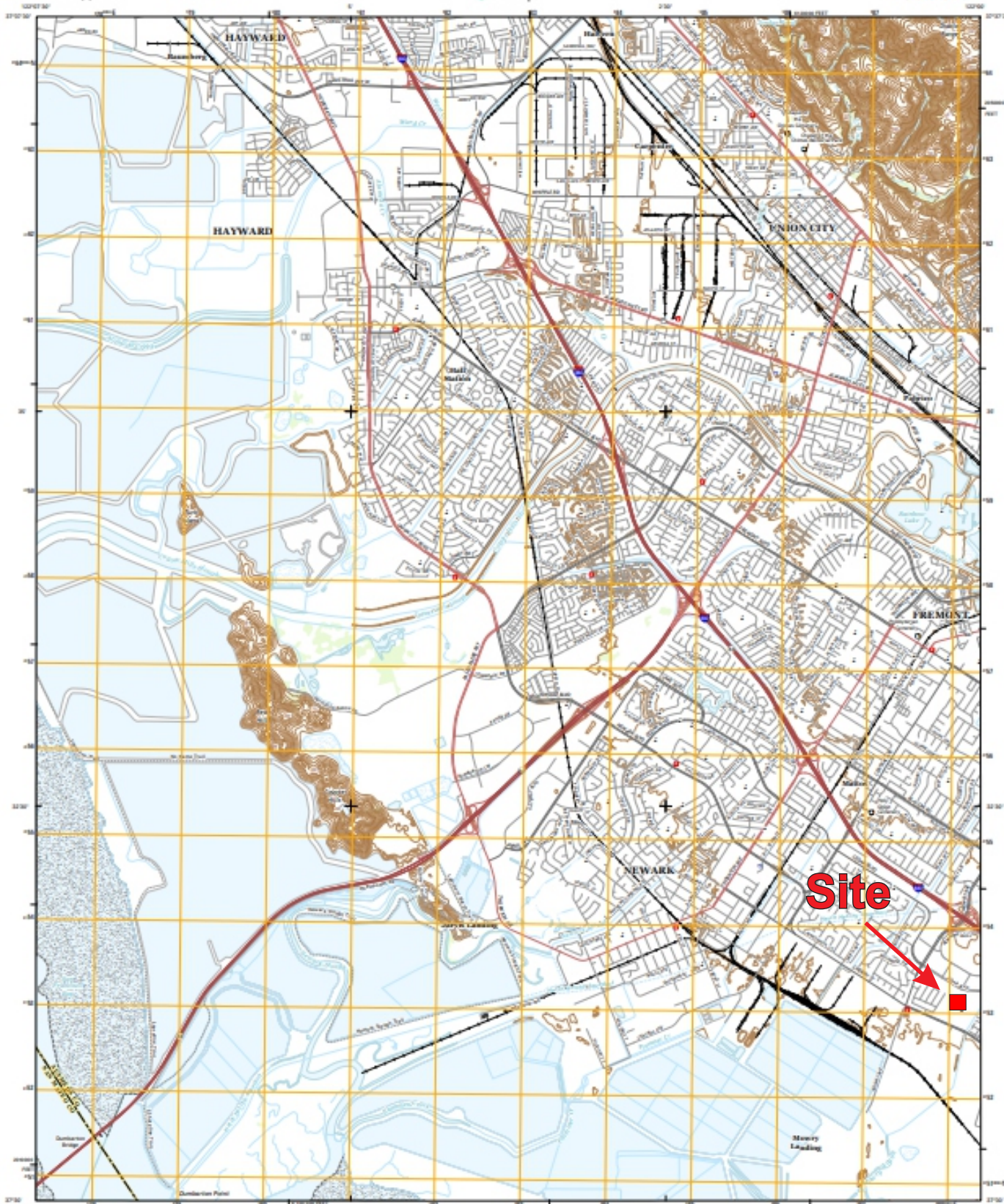
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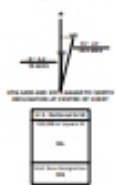
Publications may have been used as general reference and not specifically cited in the report text.

FIGURES

- Figure 1 – Site Vicinity Map**
- Figure 2 – Site Map and Boring Locations**
- Figure 3 – Site Vicinity Geologic Map**
- Figure 4 – Regional Fault Map**
- Figure 5 – Seismic Hazard and AP Fault Zone Map**
- Figure 6 – Flood Hazard Map**
- Figure 7 – Percolation Test Locations**



Produced by the United States Geological Survey
North American Datum of 1983 (NAD83)
This map is not a legal document. Assistance may be
generated for this map only. It is not to be used for
any other purpose. Where permission is required,
contact your local office.



SCALE 1:34,000
CONTINUOUS SERIES OF FEET
NORTH AMERICAN DATUM (NAD 83) OF 1983



ROAD CLASSIFICATION	
Primary Road	Secondary Road
Interstate	State Route
Local Road	Other Road

1	1	1
2	2	2
3	3	3
4	4	4
5	5	5

NEWARK, CA
2015



Newark Memorial High School
39375 Cedar Blvd
Newark, CA

142-1485

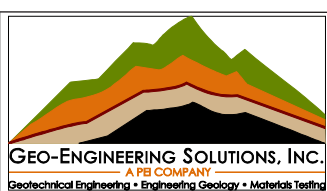
May 2023

Site Vicinity Map

Figure 1



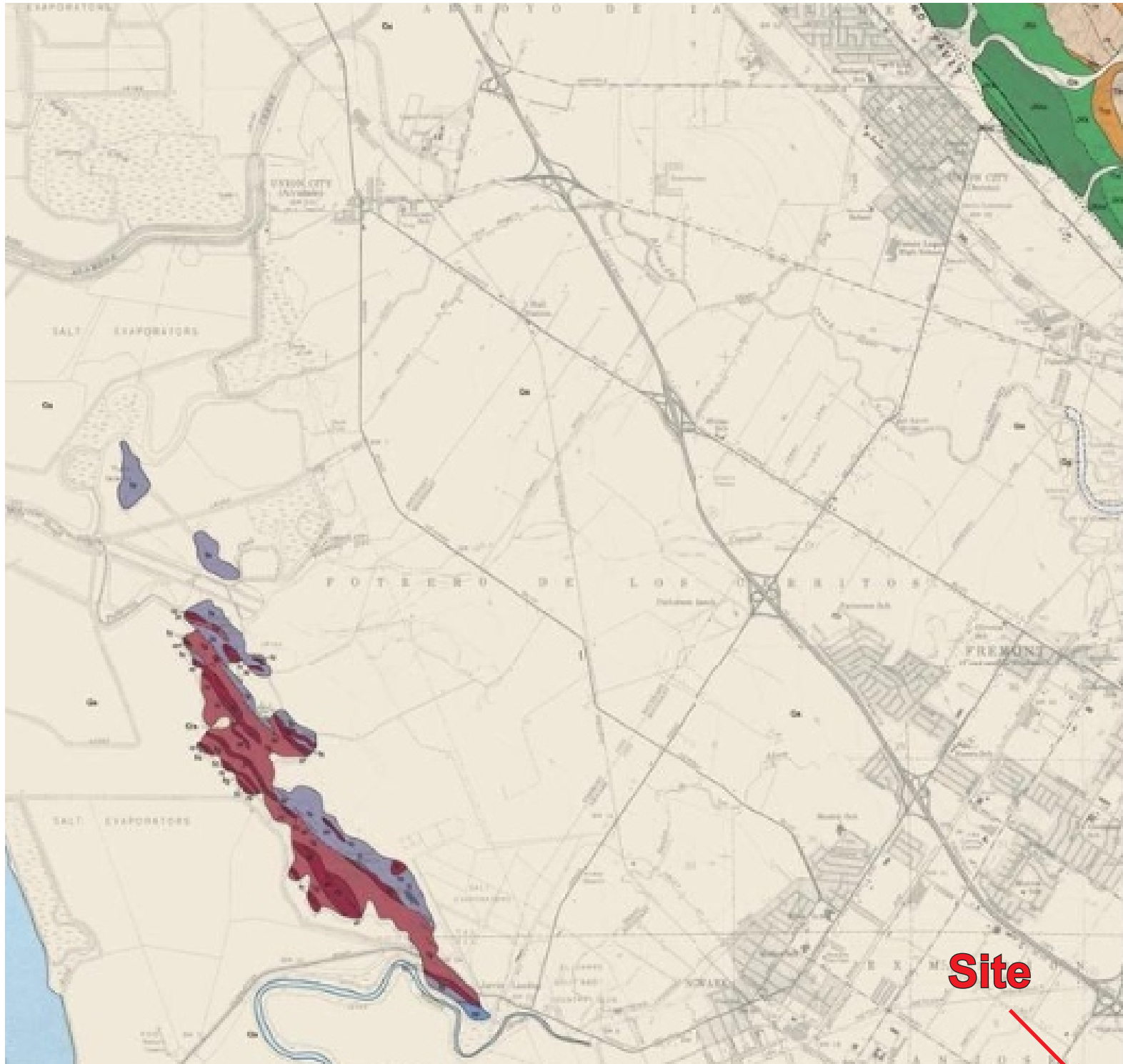
 Approximate Boring Location



Newark Memorial High School
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Site Map and Boring Locations	Figure 2
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NEWARK MAP (DF-150)

LEGEND



SURFICIAL SEDIMENTS

Qg Sand and gravel of major stream channels

Qa Alluvial gravel, sand and clay of valley areas



BRIONES SANDSTONE

Marine-clastic, littoral; age late Miocene

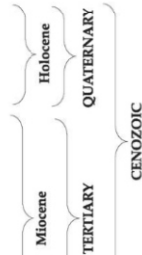
Tbr Sandstone, light gray, medium grained, thick bedded, arkosic, includes some thin interbeds of gray siltstone



MONTEREY FORMATION

Marine biogenic and clastic, lithified; age late to middle Miocene

Tm Siliceous shales, white to tan weathered, thin bedded, partly, locally called Claremont Formation



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Site Vicinity
Geologic Map

Figure 3



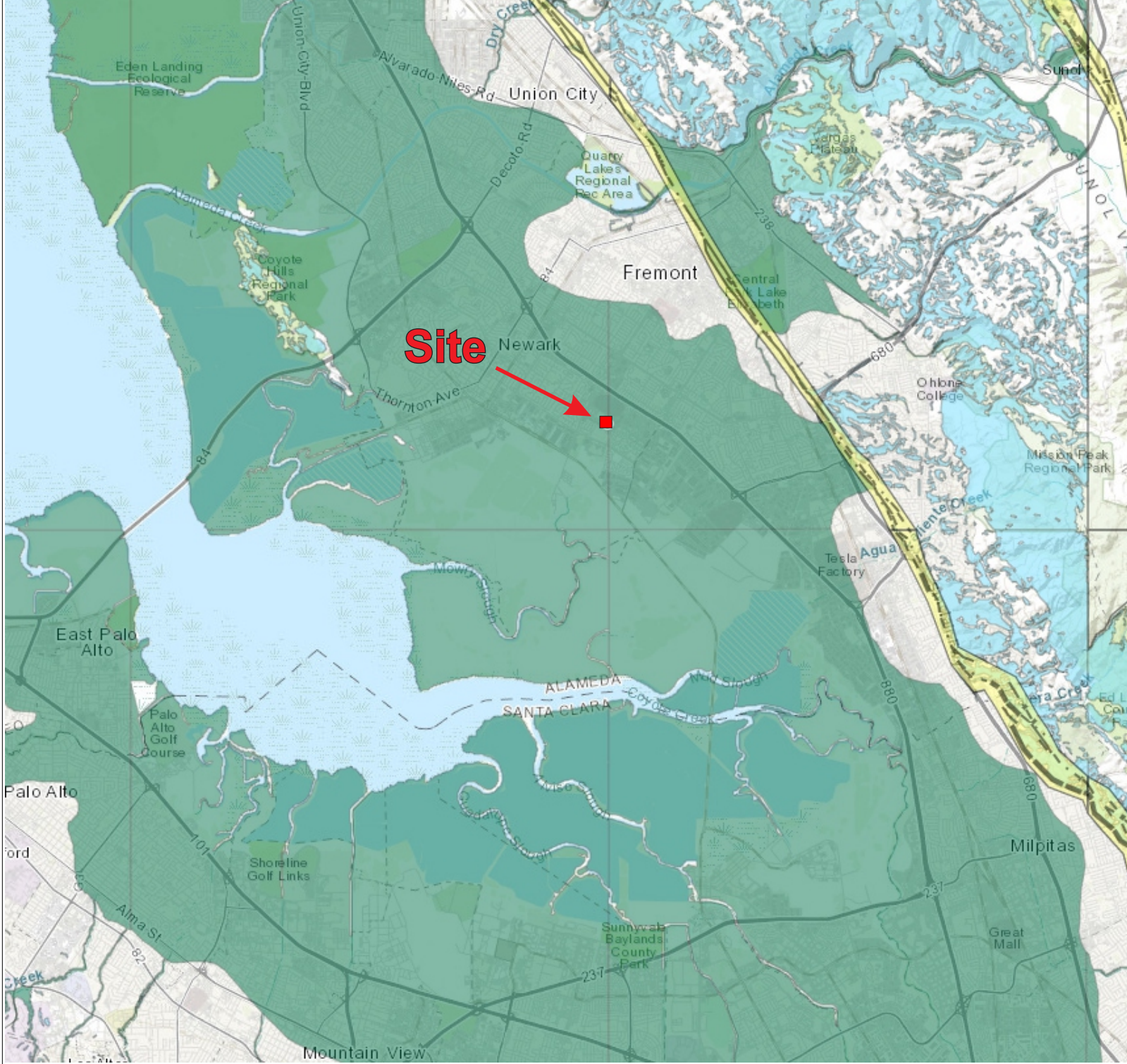
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May 2023

Regional Fault Map

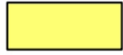
Figure 4



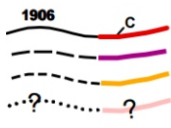
Liquefaction Zones
 Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.



Earthquake-Induced Landslide Zones
 Areas where previous occurrence of 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.



Earthquake Fault Zones
 Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.



Active Fault Traces
 Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.



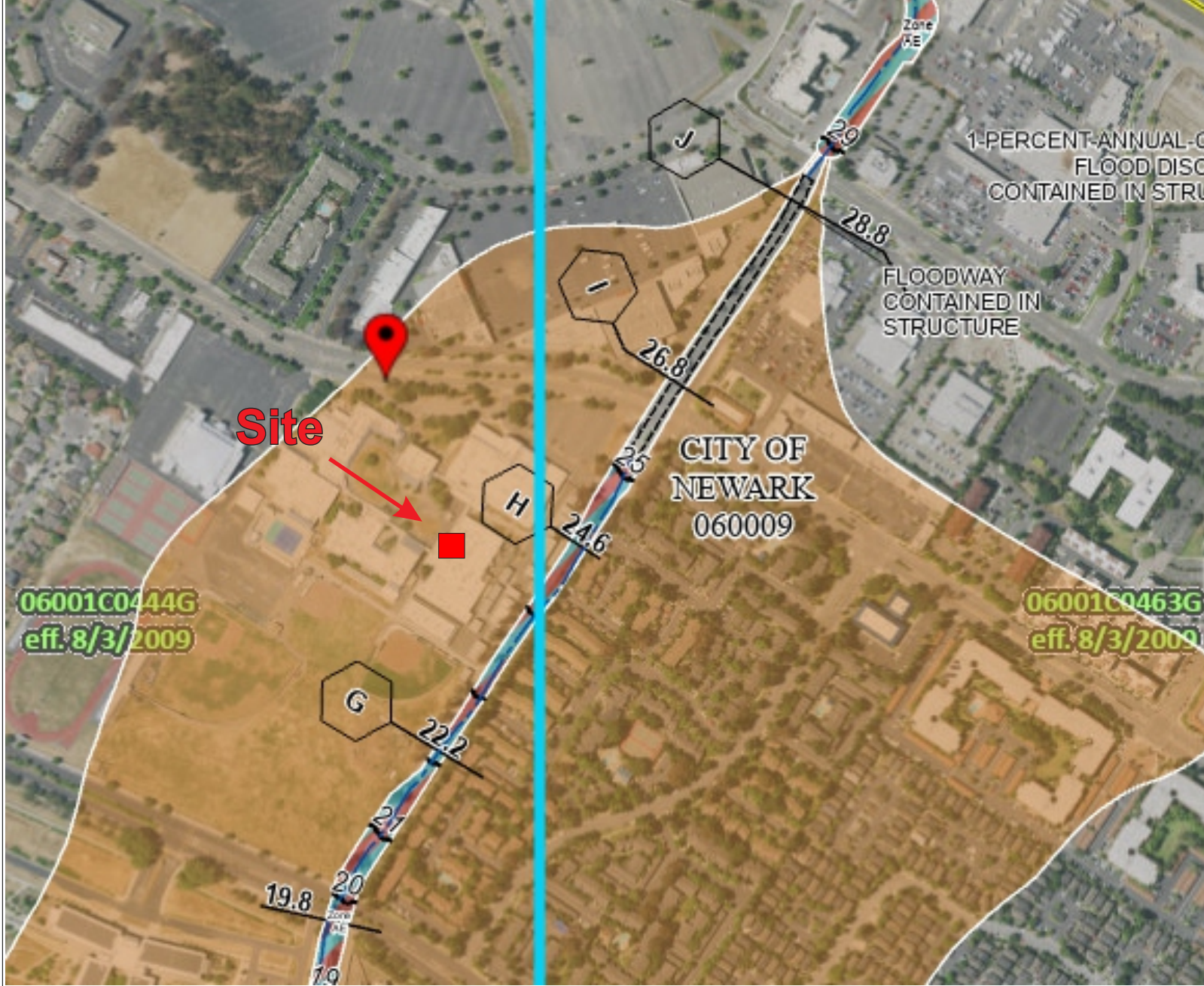
Newark Memorial High School
39375 Cedar Blvd
Newark, CA

142-1485

May 2023

Seismic Hazard and AP Fault Zone Map

Figure 5



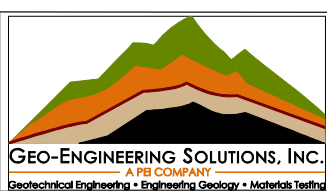
- PIN**
 - Approximate location based on user input and does not represent an authoritative property location
- MAP PANELS**
 - Selected FloodMap Boundary
 - Digital Data Available
 - No Digital Data Available
 - Unmapped
- OTHER AREAS**
 - Area of Minimal Flood Hazard Zone X
 - Effective LOMRs
 - Area of Undetermined Flood Hazard Zone D
 - Otherwise Protected Area
 - Coastal Barrier Resource System Area

- SPECIAL FLOOD HAZARD AREAS**
 - Without Base Flood Elevation (BFE) Zone A, V, A99
 - With BFE or Depth
 - Regulatory Floodway Zone AE, AO, AH, VE, AR
- OTHER AREAS OF FLOOD HAZARD**
 - 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile Zone X
 - Future Conditions 1% Annual Chance Flood Hazard Zone X
 - Area with Reduced Flood Risk due to Levee. See Notes. Zone X
 - Area with Flood Risk due to Levee Zone D

- OTHER FEATURES**
 - Cross Sections with 1% Annual Chance Water Surface Elevation
 - Coastal Transect
 - Base Flood Elevation Line (BFE)
 - Limit of Study
 - Jurisdiction Boundary
 - Coastal Transect Baseline
 - Profile Baseline
 - Hydrographic Feature
- GENERAL STRUCTURES**
 - Channel, Culvert, or Storm Sewer
 - Levee, Dike, or Floodwall

NATIONAL FLOOD INSURANCE PROGRAM
FLOOD INSURANCE RATE MAP

CONTRA COSTA COUNTY, California



Newark Memorial High School
39375 Cedar Blvd
Newark, CA


142-1485

May 2023

Flood Hazard Map

Figure 6



 Approximate percolation test locations (Geo-Eng 2023)



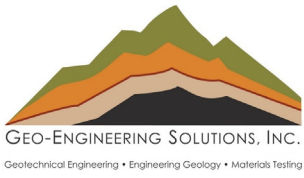
Newark Memorial High School
39375 Cedar Blvd
Newark, CA

142-1485

May 2023

Percolation Testing
Map

Figure 7



APPENDIX A

FIELD EXPLORATION
Key to Boring Log Symbols
Boring Logs

Unified Soil Classification (USC) System (from ASTM D 2487)

Major Divisions			Typical Names	
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	Gravels 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	Silts and Clays Liquid Limit 50% or less	ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands	
		CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays	
		OL	Organic silts and organic silty clays of low plasticity	
	Silts and Clays Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
		CH	Inorganic clays or high plasticity, fat clays	
		OH	Organic clays of medium to high plasticity	
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)				
SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

Particle Sizes		
Components	Size or Sieve Number	
Boulders	Over 12 inches	
Cobbles	3 to 12 inches	
Gravels	Coarse	3/4 to 3 inches
	Fine	Number 4 to 3/4 inch
Sand	Coarse	Number 10 to Number 4
	Medium	Number 40 to Number 10
	Fine	Number 200 to Number 40
Fines (Silt and Clay)	Below Number 200	

- Bulk Sample
- Standard Penetration Test
- 2.5 Inch Modified California Sampler
- Shelby Tube

Blow Count

The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5" indicates 50 blows recorded for 5 inches of penetration. Note all of the field blow counts recorded using a Modified California sampler were converted to equivalent SPT blow counts.

N-Value

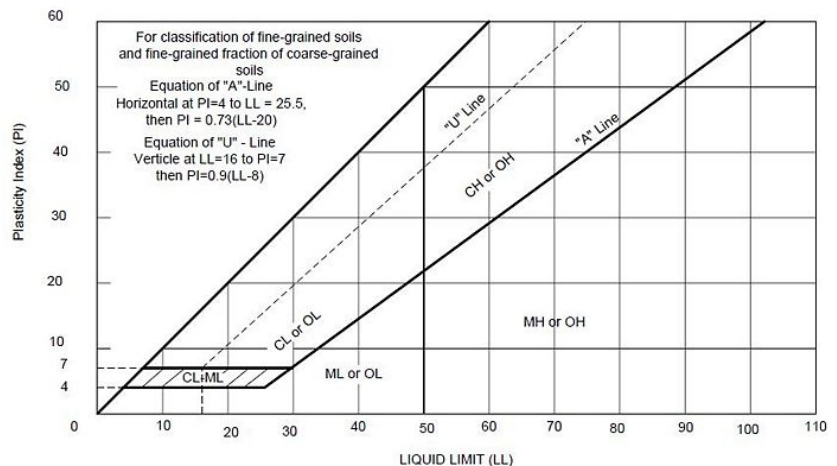
Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 inch I.D.) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test).

Soil Moisture	
Descriptor	Description
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Standard Proctor Optimum
Saturated	Free Water in Sample

- First Water Level Reading
- Final Water Level Reading

General Notes:

- The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source identified in the report. The location and elevation of borings should be considered accurate only to the degree implied by the method.
- The stratification lines represent the approximate boundary between soil types. The transition may be gradual.
- Water level readings in the drill holes were recorded at the time and under the conditions stated on the boring logs. It should be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, tides and other factors at the time measurements were made



Key to Exploratory Boring Logs



2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA

BORING NUMBER B-1

PAGE 1 OF 2

CLIENT Newark Unified School District	PROJECT NAME Newark Memorial High School
PROJECT NUMBER 142-1485	PROJECT LOCATION 39375 Cedar Blvd, Newark, CA 94560
DATE STARTED 5/3/23 COMPLETED 5/3/23	GROUND ELEVATION 26 ft HOLE SIZE 4"
DRILLING CONTRACTOR California Geotech Services, LLC	GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75	▽ AT TIME OF DRILLING 7.00 ft / Elev 19.00 ft
LOGGED BY MT CHECKED BY NH	AT END OF DRILLING ---
NOTES	AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		(CL) Silty CLAY : Dark brown, moderate to high plasticity, stiff, moist										
	▽		MC 1-1		8-7-7 (14)	2.5	104	18				
			MC 1-2		3-3-4 (7)	0.50	100	18	31	15	16	
5		Becomes medium stiff and grayish green										
		Becomes stiff		SPT 1-3		4-6-7 (13)						
10				SPT 1-4		3-4-3 (7)			25			
15		Becomes medium stiff and increase in sand content										
20			SPT 1-5		3-3-3 (6)			23				

(Continued Next Page)



2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA

BORING NUMBER B-1

PAGE 2 OF 2

CLIENT Newark Unified School District

PROJECT NAME Newark Memorial High School

PROJECT NUMBER 142-1485

PROJECT LOCATION 39375 Cedar Blvd, Newark, CA 94560

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
20		(CL) Silty CLAY : Dark brown, moderate to high plasticity, stiff, moist <i>(continued)</i>										
25		(SC) Clayey SAND : Dark grayish green, loose, moist	SPT 1-6		3-3-5 (8)			24				47
30		(CL) Silty CLAY : Grayish green, moderate to high plasticity, very stiff, moist	SPT 1-7		4-11-11 (22)			34				

Bottom of borehole at 30.0 feet.



2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA

BORING NUMBER B-2

PAGE 1 OF 1

CLIENT Newark Unified School District
PROJECT NUMBER 142-1485
DATE STARTED 5/3/23 **COMPLETED** 5/3/23
DRILLING CONTRACTOR California Geotech Services, LLC
DRILLING METHOD Solid Flight CME-75
LOGGED BY MT **CHECKED BY** NH
NOTES

PROJECT NAME Newark Memorial High School
PROJECT LOCATION 39375 Cedar Blvd, Newark, CA 94560
GROUND ELEVATION 26 ft **HOLE SIZE** 4"
GROUND WATER LEVELS:
 ▽ **AT TIME OF DRILLING** 7.00 ft / Elev 19.00 ft
AT END OF DRILLING ---
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		(CL) <u>Silty CLAY</u> : Dark brown, moderate to high plasticity, very stiff, moist										
			MC 2-1		7-10-10 (20)	>4.5	116	14	25	15	10	
			MC 2-2		5-7-8 (15)	2.5		13				
5		Becomes grayish green Becomes Sandy Clay										
				MC 2-3		4-5-6 (11)	1.5	101	21			
10		Becomes stiff and Silty Clay										
			SPT 2-4		5-5-6 (11)			24				
15												
		Becomes medium stiff	SPT 2-5		3-3-3 (6)			26				
20												

Bottom of borehole at 20.0 feet.



2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA

BORING NUMBER B-3

PAGE 1 OF 1

CLIENT Newark Unified School District
PROJECT NUMBER 142-1485
DATE STARTED 5/3/23 **COMPLETED** 5/3/23
DRILLING CONTRACTOR California Geotech Services, LLC
DRILLING METHOD Solid Flight CME-75
LOGGED BY MT **CHECKED BY** NH
NOTES

PROJECT NAME Newark Memorial High School
PROJECT LOCATION 39375 Cedar Blvd, Newark, CA 94560
GROUND ELEVATION 26 ft **HOLE SIZE** 4"
GROUND WATER LEVELS:
 ∇ **AT TIME OF DRILLING** 7.00 ft / Elev 19.00 ft
AT END OF DRILLING ---
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		(CL) Sandy CLAY : Grayish green, moist, moderate to high plasticity, medium stiff										
			MC 3-1		3-3-5 (8)	4.0	118	13	27	14	13	
			MC 3-2		2-3-4 (7)							
5		∇ (CL) Becomes stiff and Silty Clay										
			SPT 3-3		3-4-6 (10)			21				
10		Becomes medium stiff										
			SPT 3-4		3-3-4 (7)			24	30	16	14	
15		Becomes stiff										
			SPT 3-5		3-4-5 (9)			25				
20												

Bottom of borehole at 20.0 feet.

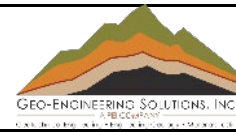
APPENDIX B

LABORATORY TEST RESULTS
Atterberg Limit's Results
Grain Size Distribution
Unconfined Compression Test

Geo-Engineering Solutions Inc.

Unconfined Compression Test

Test Methods: ASTM D2166



Project Name	Newark Memorial High School	142-1485
Tested By	MT	5/8/2023
Soil Type	Dark Brown Clay	B1-1 @ 3'

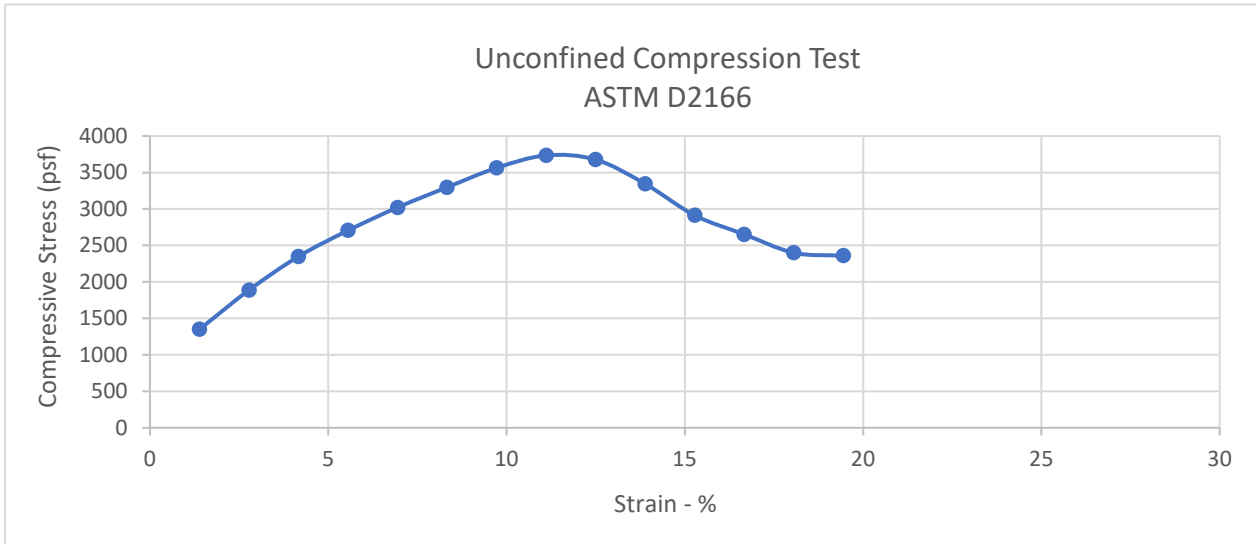
Water Content		Initial Measurements	
Dish ID	D1	Sample Diameter (in)	2.40
Mass of Dish (g)	309.1	Area (ft ²)	0.03
Moist Soil + Dish (g)	976.8	Sample Height (in)	4.50
Mass of Dry Soil + Dish (g)	873.9	Volume (ft ³)	0.01

Elapsed Time (Seconds)	Load Dial (lbs)	Strain Dial (in)	Axial Strain (%)	Corrected Area (ft ²)	Stress (psf)
0	43	4.4375	1.39	0.03186	1349.72
60	61	4.375	2.78	0.03231	1887.75
120	77	4.3125	4.17	0.03278	2348.86
180	90	4.25	5.56	0.03326	2705.63
240	102	4.1875	6.94	0.03376	3021.29
300	113	4.125	8.33	0.03427	3297.16
360	124	4.0625	9.72	0.03480	3563.30
420	132	4	11.11	0.03534	3734.84
480	132	3.9375	12.50	0.03590	3676.48
540	122	3.875	13.89	0.03648	3344.02
600	108	3.8125	15.28	0.03708	2912.54
660	100	3.75	16.67	0.03770	2652.58
720	92	3.6875	18.06	0.03834	2399.70
780	92	3.625	19.44	0.03900	2359.03
840			100.00	#DIV/0!	#DIV/0!
900			100.00	#DIV/0!	#DIV/0!
960			100.00	#DIV/0!	#DIV/0!
1020			100.00	#DIV/0!	#DIV/0!
1080			100.00	#DIV/0!	#DIV/0!
1140			100.00	#DIV/0!	#DIV/0!
1200			100.00	#DIV/0!	#DIV/0!
1260			100.00	#DIV/0!	#DIV/0!
1320			100.00	#DIV/0!	#DIV/0!
1380			100.00	#DIV/0!	#DIV/0!
1440			100.00	#DIV/0!	#DIV/0!
1500			100.00	#DIV/0!	#DIV/0!
1560			100.00	#DIV/0!	#DIV/0!
1620			100.00	#DIV/0!	#DIV/0!

Geo-Engineering Solutions Inc.

Unconfined Compression Test

Test Methods: ASTM D2166

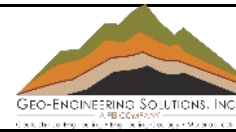


Speciman Failure Photograph		Speciman Test Data		
		Moisture Content:	%	18.2%
		Dry Density:	pcf	104.0
		Diameter:	in.	2.40
		Height:	in.	4.5
		Height/ Diameter Ratio:		1.875
		Calculated Saturation:	%	79%
		Calculated Void Ratio:		0.619596
		Assumed Specific Gravity:		2.7
		Failure Strain:	%	11.11
		Unconfined Compressive Strength:	(psf)	3734.84
		Undrained Shear Strength	(psf)	1867.42
		Strain Rate:	in/min	0.0625
		Remarks:		
Sample Type:	Mod Cal	Sample Location:	B1-1 @ 3'	
Description:	Brown Sandy CLAY	LL:	PL:	PI:
Moisture Density Data		Water Content	0.182	
Number of Rings	1	Total Density (Mg/m ³)	1.971	
Mass of Moist Sample + Rings (g)	1176.7	Dry Density (Mg/m ³)	1.667	
Specimen Diameter (mm)	62	Total Unit Weight (lbs/ft ³)	122.978	
Specimen Height (mm)	152	Dry Unit Weight (lbs/ft ³)	104.026	
Total Ring Mass (g)	272.3	Saturation	0.794	
Soil Mass (g)	904.4	Void Ratio	0.620	
Soil Volume (cm ³)	458.90	Porosity	0.383	

Geo-Engineering Solutions Inc.

Unconfined Compression Test

Test Methods: ASTM D2166



Project Name	Newark Memorial High School	142-1485
Tested By	MT	5/8/2023
Soil Type	Brown Sandy Clay	B2-2 @5'

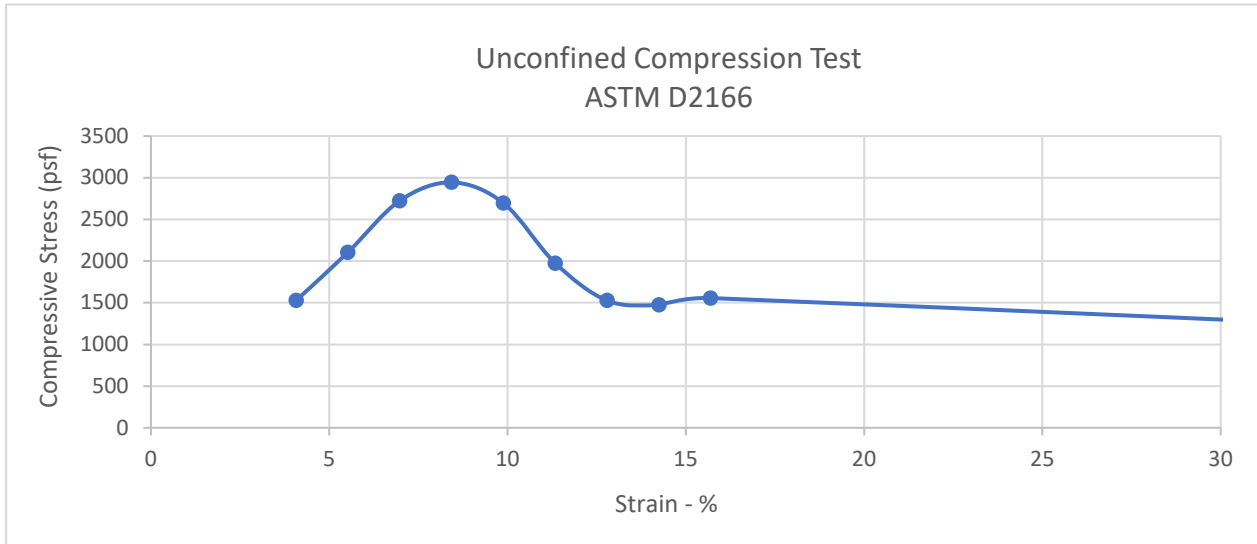
Water Content		Initial Measurements	
Dish ID	D6	Sample Diameter (in)	2.40
Mass of Dish (g)	300.5	Area (ft ²)	0.03
Moist Soil + Dish (g)	997.7	Sample Height (in)	4.30
Mass of Dry Soil + Dish (g)	919.9	Volume (ft ³)	0.01

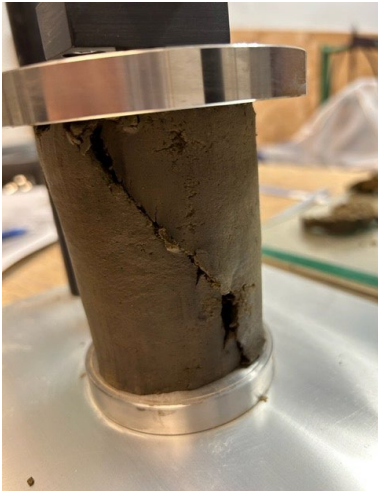
Elapsed Time (Seconds)	Load Dial (lbs)	Strain Dial (in)	Axial Strain (%)	Corrected Area (ft ²)	Stress (psf)
0	50	4.125	4.07	0.03275	1526.78
60	70	4.0625	5.52	0.03325	2105.10
120	92	4	6.98	0.03377	2724.14
180	101	3.9375	8.43	0.03431	2943.90
240	94	3.875	9.88	0.03486	2696.38
300	70	3.8125	11.34	0.03543	1975.56
360	55	3.75	12.79	0.03602	1526.78
420	54	3.6875	14.24	0.03663	1474.03
480	58	3.625	15.70	0.03727	1556.39
540			100.00	#DIV/0!	#DIV/0!
600			100.00	#DIV/0!	#DIV/0!
660			100.00	#DIV/0!	#DIV/0!
720			100.00	#DIV/0!	#DIV/0!
780			100.00	#DIV/0!	#DIV/0!
840			100.00	#DIV/0!	#DIV/0!
900			100.00	#DIV/0!	#DIV/0!
960			100.00	#DIV/0!	#DIV/0!
1020			100.00	#DIV/0!	#DIV/0!
1080			100.00	#DIV/0!	#DIV/0!
1140			100.00	#DIV/0!	#DIV/0!
1200			100.00	#DIV/0!	#DIV/0!
1260			100.00	#DIV/0!	#DIV/0!
1320			100.00	#DIV/0!	#DIV/0!
1380			100.00	#DIV/0!	#DIV/0!
1440			100.00	#DIV/0!	#DIV/0!
1500			100.00	#DIV/0!	#DIV/0!
1560			100.00	#DIV/0!	#DIV/0!
1620			100.00	#DIV/0!	#DIV/0!

Geo-Engineering Solutions Inc.

Unconfined Compression Test

Test Methods: ASTM D2166



Speciman Failure Photograph	Speciman Test Data		
	Moisture Content:	%	12.6%
	Dry Density:	pcf	#DIV/0!
	Diameter:	in.	2.40
	Height:	in.	4.3
	Height/ Diameter Ratio:		1.792
	Calculated Saturation:	%	#DIV/0!
	Calculated Void Ratio:		#DIV/0!
	Assumed Specific Gravity:		2.7
	Failure Strain:	%	8.43
	Unconfined Compressive Strength:	(psf)	2943.90
	Undrained Shear Strength	(psf)	1471.95
	Strain Rate:	in/min	0.0625
Remarks:			

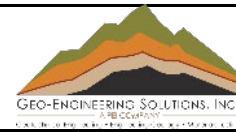
Sample Type:	Mod Cal	Sample Location:	B2-2 @5'
Description:	Brown Sandy CLAY	LL:	PL:
		PI:	

Moisture Density Data		Water Content	
Number of Rings		Total Density (Mg/m ³)	0.126
Mass of Moist Sample + Rings (g)		Dry Density (Mg/m ³)	#DIV/0!
Specimen Diameter (mm)		Total Unit Weight (lbs/ft ³)	#DIV/0!
Specimen Height (mm)		Dry Unit Weight (lbs/ft ³)	#DIV/0!
Total Ring Mass (g)		Saturation	#DIV/0!
Soil Mass (g)	0	Void Ratio	#DIV/0!
Soil Volume (cm ³)	0.00	Porosity	#DIV/0!

Geo-Engineering Solutions Inc.

Unconfined Compression Test

Test Methods: ASTM D2166



Project Name	Newark Memorial High School	142-1485
Tested By	MT	5/8/2023
Soil Type	Grayish Green Sandy Clay	B1-1 @ 3'

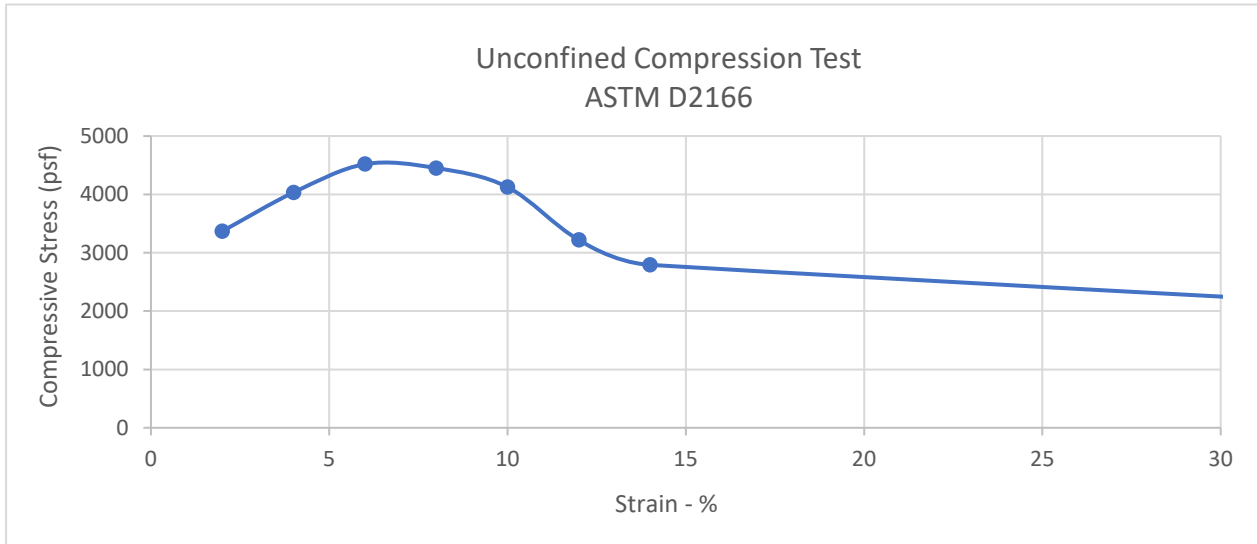
Water Content		Initial Measurements	
Dish ID	D8	Sample Diameter (in)	2.40
Mass of Dish (g)	300	Area (ft ²)	0.03
Moist Soil + Dish (g)	1253.7	Sample Height (in)	3.13
Mass of Dry Soil + Dish (g)	1144.9	Volume (ft ³)	0.01

Elapsed Time (Seconds)	Load Dial (lbs)	Strain Dial (in)	Axial Strain (%)	Corrected Area (ft ²)	Stress (psf)
0	108	3.0625	2.00	0.03206	3368.99
60	132	3	4.00	0.03272	4033.62
120	151	2.9375	6.00	0.03342	4518.09
180	152	2.875	8.00	0.03415	4451.25
240	144	2.8125	10.00	0.03491	4125.30
300	115	2.75	12.00	0.03570	3221.30
360	102	2.6875	14.00	0.03653	2792.21
420			100.00	#DIV/0!	#DIV/0!
480			100.00	#DIV/0!	#DIV/0!
540			100.00	#DIV/0!	#DIV/0!
600			100.00	#DIV/0!	#DIV/0!
660			100.00	#DIV/0!	#DIV/0!
720			100.00	#DIV/0!	#DIV/0!
780			100.00	#DIV/0!	#DIV/0!
840			100.00	#DIV/0!	#DIV/0!
900			100.00	#DIV/0!	#DIV/0!
960			100.00	#DIV/0!	#DIV/0!
1020			100.00	#DIV/0!	#DIV/0!
1080			100.00	#DIV/0!	#DIV/0!
1140			100.00	#DIV/0!	#DIV/0!
1200			100.00	#DIV/0!	#DIV/0!
1260			100.00	#DIV/0!	#DIV/0!
1320			100.00	#DIV/0!	#DIV/0!
1380			100.00	#DIV/0!	#DIV/0!
1440			100.00	#DIV/0!	#DIV/0!
1500			100.00	#DIV/0!	#DIV/0!
1560			100.00	#DIV/0!	#DIV/0!
1620			100.00	#DIV/0!	#DIV/0!

Geo-Engineering Solutions Inc.

Unconfined Compression Test

Test Methods: ASTM D2166



Speciman Failure Photograph	Speciman Test Data		
	Moisture Content:	%	12.9%
	Dry Density:	pcf	118.1
	Diameter:	in.	2.40
	Height:	in.	3.1
	Height/ Diameter Ratio:		1.302
	Calculated Saturation:	%	81%
	Calculated Void Ratio:		0.426831
	Assumed Specific Gravity:		2.7
	Failure Strain:	%	6
	Unconfined Compressive Strength:	(psf)	4518.09
	Undrained Shear Strength	(psf)	2259.05
	Strain Rate:	in/min	0.0625
	Remarks:		

Sample Type:	Mod Cal	Sample Location:	B1-1 @ 3'		
Description:	Brown Sandy CLAY	LL:	PL:	PI:	

Moisture Density Data		Water Content	
Number of Rings	1	Total Density (Mg/m ³)	2.136
Mass of Moist Sample + Rings (g)	1218.1	Dry Density (Mg/m ³)	1.892
Specimen Diameter (mm)	62	Total Unit Weight (lbs/ft ³)	133.285
Specimen Height (mm)	152	Dry Unit Weight (lbs/ft ³)	118.080
Total Ring Mass (g)	237.9	Saturation	0.815
Soil Mass (g)	980.2	Void Ratio	0.427
Soil Volume (cm ³)	458.90	Porosity	0.299