Pierce HS Pool Complex

Geotechnical Engineering Report and Geologic Hazards Evaluation

January 11, 2023 | Terracon Project No. NB225033

Prepared for:

Pierce Joint Unified School District
540-A 6th Street
Arbuckle, CA 95912

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January 11, 2023

Pierce Joint Unified School District
540-A 6th Street
Arbuckle, CA 95912

Attn: George Parker
P: (530) 788-3533
E: gparker@pjusd.com

Re: Geotechnical Engineering Report and Geologic Hazards Evaluation
Pierce HS Pool Complex
966 Wildwood Road
Arbuckle, CA 95912
Terracon Project No. NB225033

Dear Mr. Parker:

We have completed the Geotechnical Engineering Report and Geologic Hazards Evaluation for the above referenced project in general accordance with Terracon Proposal No. PNB225033 dated October 27, 2022. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of the proposed pool, foundations, and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon

Staysha P. Delgado, E.I.T.
Senior Staff Engineer

Noah T. Smith, P.E., G.E.
Principal

Curtis Hall, P.G., C.E.G.
Project Geologist

Table of Contents

Figures

GeoModel

Attachments

Exploration and Testing Procedures
Site Location and Exploration Plans
Exploration and Laboratory Results
Supporting Information

Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

Report Summary

Topic [1]	Overview Statement [2]
Project Description	The project will consist of razing three tennis courts to accommodate the construction of a new outdoor competition swimming pool approximately 7,560 square foot (sf) in size, a 1,950 sf one-story restroom/storage building, and associated hardscapes. Estimated maximum loads: ■ Walls: 2 kips per linear foot (klf) ■ Slabs: 100 pounds per square foot (psf)
Geotechnical Characterization	Subgrade soil conditions encountered in our borings generally consisted of interbedded layers of medium dense to dense clayey sands with variable amounts of gravel and stiff to hard sandy lean clay with variable amounts of gravel to the maximum depth explored of 21½ feet bgs. Groundwater was not encountered in the borings while drilling or for the short duration they remained open.

Earthwork	Clays are sensitive to moisture variation. Earthwork is anticipated to consist of cut and fills of 2 feet or less to achieve final grade. Excavations up to 12 feet deep are anticipated associated with construction of the new pool, pump pit, and surge tank.
Swimming Pool	The swimming pool may be constructed utilizing conventional in-ground construction bearing into firm native soil. We have assumed the pool will be L-shaped with a deck level gutter and approximately 3½ to 12 feet deep. Loose/soft soil encountered in the bottom of the pool excavation should be over-excavated to firm native soil.
Shallow Foundations	The restroom/storage building may be supported by **Shallow Foundations** provided the footings extend a minimum 18 inches bgs and bear on firm native soil. Allowable bearing pressure = 2,500 psf Expected settlements: < 1-inch total, < ½-inch differential
Slabs	Interior and exterior concrete slabs-on-grade should be underlain by at least 12 inches of compacted granular structural fill.
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Introduction

This report presents the results of our subsurface exploration and Geotechnical Engineering Report and Geologic Hazards Evaluation services performed for the proposed pool complex to be located at 966 Wildwood Road in Arbuckle, CA 95912. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Seismic site classification per the 2022 California Building Code (CBC)
- Site preparation and earthwork
- Demolition considerations
- Swimming pool design and construction

- Foundation design and construction
- Floor slab design and construction
- Lateral earth pressures

The geotechnical engineering Scope of Services for this project included the advancement of (4) test borings, laboratory testing, engineering analysis, and preparation of this report.

Drawings showing the site and boring locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	Email from George Parker with Pierce Joint Unified School District sent on February 8, 2022 providing a brief project description and preliminary site plan. An additional email from George Parker was sent on October 11, 2022 providing an updated site plan.
Project Description	The project will consist of the demolition of three tennis courts to facilitate the construction of a new pool complex which will include: one outdoor competition pool, one restroom/storage building, and a corresponding pool deck.
Proposed Structures	Structures associated with the project include: ▪ One L-shaped, outdoor competition pool with a deck level gutter and a footprint of approximately 7,560 square feet. We anticipate the pool will vary in depth from 3½ to 12 feet. ▪ Restroom/storage building with a footprint of approximately 1,950 square feet.
Proposed Construction	The restroom/storage building will be one-story and consist of concrete masonry construction with a slab on grade floor. We anticipate the swimming pool will consist of both concrete and shotcrete construction.

Item	Description
Finished Elevations	Not provided; we have assumed finished elevations for the restroom/storage building floor and pool deck will not be more than 2 feet above/below existing grades.
Maximum Loads	Anticipated structural loads were not provided. In the absence of information provided by the design team, we have used the following loads in estimating settlement based on our experience with similar projects. ■ Walls: 2 kips per linear foot (klf) ■ Slabs: 100 pounds per square foot (psf)
Grading	A preliminary grading plan was not available for review at the time this report was prepared. We have assumed construction of the proposed restroom/storage building and pool deck will require relatively minor grading with cuts and fills on the order of 2 feet or less. Excavations varying from 5 to 12 feet deep bgs will likely be required to construct the proposed pool, pump pit, and surge tank.
Below-Grade Structures	We anticipate a pump pit and a surge tank will be installed as part of the pool construction. The locations of the pump pit and surge tank are not known at this time. We have assumed the pump pit will have a maximum depth of 5 feet bgs and the surge tank will have a maximum depth of 8 feet bgs.
Exterior Hardscape	Rigid (concrete) flatwork is being considered for pool deck and accessible concrete walkways. We assume that the hardscape will not experience vehicular traffic loads and is meant for pedestrian use only.

Terracon should be notified if any of the above information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located within the Pierce High School campus located at 960 Wildwood Rd Arbuckle, California Assessor Parcel Numbers (APN): 020-140-002-000 The area of the proposed development is approximately 0.45 acres in area. Latitude and Longitude (approximate): 39.0119°N, 122.0556°W See **Site Location**
Existing Terracon Reports	Terracon previously provided the following reports prepared for development at the Pierce High School: ■ Geotechnical Engineering Report, Pierce High School Proposed Cafeteria Building, prepared by Terracon, Project No. NB165007, Dated October 25, 2016. ■ Geotechnical Investigation, New Agriculture Building Pier High School, prepared by Neil O. Anderson & Associates, A Terracon Company, Project No. SG03-042, Dated April 28, 2003. ■ Geologic Hazards Study, Proposed New Cafeteria Building Pierce High School, prepared by Stephen E. Jacobs, Project No. 16004, Dated February 8, 2016. ■ Updated Geotechnical Investigation, Pierce High School – Sports Field Lighting, prepared by Neil O. Anderson & Associates, A Terracon Company, Project No. SGE090518, Dated April 14, 2009. ■ Geotechnical Engineering Report, PJUSD Solar Carport Canopies, prepared by Terracon, Project No. NB215002, Dated May 21, 2021. ■ Geotechnical Engineering Report, Proposed New Locker Room and Weight Room, prepared by Terracon, Project No. NB175116, Dated September 18, 2017. ■ Geologic Hazards Study, Proposed New Locker Room and Weight Room, prepared by Stephen E. Jacobs, Project No. 16004a, Dated September 11, 2017. ■ Geotechnical Engineering Report, New Athletic Field Bleachers, prepared by Terracon, Project No. NB185100, Dated August 10, 2018. ■ Geologic Hazards Study, Proposed New Bleachers, prepared by Stephen E. Jacobs, Project No. 18038, Dated August 9, 2018. These reports were reviewed in preparation of this report.

Item	Description
Existing Improvements	The project area is developed with three tennis courts, light posts and overhead powerlines. The tennis court surfaces are cracked and in disrepair. The site is bordered to the west by Wildwood Rd, to the north by the existing school swimming pool, to the east by additional tennis courts and Interstate 5, and to the south by the Pierce High School campus.
Current Ground Cover	Paved tennis courts.
Existing Topography	The project area is relatively flat with approximately 2 feet of topographic relief across the site.

Geotechnical Characterization

We have developed a general characterization of the subsurface soil and groundwater conditions based upon the results of our subsurface exploration, laboratory data, our review of available data, our understanding of the geologic setting, and our understanding of the project.

Subsurface Profile

The subsurface soils generally consist of interbedded layers of stiff to hard lean clay with variable amounts of gravel and medium dense to dense sand with variable amounts of clay and gravel. Section A-A’ and Section B-B’ illustrate the general soil profile underlying the proposed structures.

GeoModel

As part of our analyses, we identified the following model layers within the subsurface profile. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. The objective of the GeoModel is to group like soil layers on the logs for discussion in the report and not to provide a layer number for each soil class. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** and the GeoModel can be found in the **Figures** attachment of this report.

For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	Surfacing	Approximately 6 inches of asphalt overlying 6 inches of aggregate base course.
2	Sandy Lean Clay	Medium plasticity, stiff to hard sandy lean clay with variable amounts of gravel. Gravel up to 2 inches in dimension.
3	Poorly Graded Sand with Clay	Medium dense poorly graded sand with clay and variable amounts of gravel. Gravel up to 1.5 inches in dimension.
4	Clayey Sand	Medium dense to dense clayey sand with variable amounts of gravel. Gravel up to 2 inches in dimension.

Additional borings, auger probes, test pits, or geophysical testing could be performed to obtain more specific subgrade information.

Groundwater Conditions

Groundwater and/or seepage were not encountered within the maximum depths of the borings at the time of our field exploration, or for the short duration the borings were open prior to backfilling. Groundwater level contour mapping for Spring 2022[1] indicates a recent groundwater depth at the site of approximately 107 feet bgs.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structures may be higher or lower than anticipated. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

Historic Groundwater Conditions

Available groundwater data were reviewed in order to estimate the historic groundwater conditions for the site. Groundwater data for State monitored wells are summarized in the following table.

[1] California Department of Water Resources (DWR); "SGMA Data Viewer"; accessed December 22, 2022; *https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#gwlevels*

Summary of Groundwater Data[1]					
State Well Number	**Date Measured (high/recent)**	**Ground Surface Elevation (feet)**	**Water Surface Elevation (feet)**	**Depth to Water (feet)**	**Distance from Site (miles)**
14N02W34N001M	10/07/1992	162.60	123.60	39.00	1.3 WNW
	10/20/1995		103.80	58.80	
14N02W36N001M	11/23/1942	115.77	78.97	36.80	0.9 ENE
	03/31/1955		53.07	62.70	
14N02W36N002M	03/03/1988	112.97	59.67	53.30	0.9 ENE
	03/11/1997		59.17	53.80	

In addition, two Caltrans projects are located near the site. Groundwater conditions, as indicated on the project boring logs, are summarized in the following table.

Summary of Groundwater Data[2]					
Caltrans Project and Project Number	**Document Date**	**Ground Surface Elevation (feet)**	**Water Surface Elevation (feet)**	**Depth to Water (feet)**	**Distance from Site (miles)**
Hillgate Road UC 150047	04/16/1956	138.4	72.9	65.5	0.1 N
Hall Street OC 150046	04/16/1956	139.6	Not Encountered	N/A	0.4 NNW

Based on our review of the available data, the estimated historic-high groundwater depth within 0.9 miles of the project is approximately 37 feet bgs. No groundwater was encountered in the borings advanced to a depth of 21½ feet bgs at the site as part of this study. However, during a previous Terracon geotechnical investigation conducted at the site in April 2003, groundwater was encountered at a depth of 47 feet bgs in a monitoring well located west of the school's maintenance barn.

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[1] California Department of Water Resources (DWR); "SGMA Data Viewer"; accessed December 22, 2022; *https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#gwlevels*

[2] California Department of Transportation (Caltrans); "GeoDOG – Digital Archive of Geotechnical Data"; accessed January 10, 2023; *https://geodog.dot.ca.gov/index.php*

Geology and Geologic Hazards

Regional Geologic Setting

The Great Valley geomorphic province is situated between the Sierra Nevada and Coast Ranges geomorphic provinces and can be separated into the (northern) Sacramento Valley and (southern) San Juaquin Valley. The Great Valley, commonly referred to as the Central Valley, can best be described as a trough into which sediments from the Coast Ranges and Sierra Nevada have been almost continuously deposited since the Jurassic Period, forming an alluvial plain, approximately 50 miles wide and 400 miles long in the central portion of California (CGS, 2002).

Site Geology

The site is located within the Great Valley geomorphic province of California, more specifically, the southern portion of the Sacramento Valley. The site lies approximately 65 kilometers west of the Sierra Nevada foothills, and 11 kilometers east of the Coast Ranges. As depicted in the Regional Geologic Map[1], the site is underlain by Holocene alluvium, consisting of unweathered gravel, sand, and silt. This material has primarily been deposited via present-day river and stream systems draining the Coast Ranges, Klamath Mountains, and Sierra Nevada. This deposition forms extensive, low relief alluvial fan deposits with thicknesses ranging from a few centimeters to 10 meters.

As part of the current Geotechnical Engineering investigation, four (4) geotechnical borings were advanced to a depth 0f 21½ feet below existing ground surface (bgs). The soils encountered in our borings are generally consistent with the mapped geology.

Faulting and Seismicity

Regional Faulting

The western boundary between Coast Ranges and the Central Valley consists of a seismically active, blind fold and thrust belt (Wakabayashi and Smith, 1994). Quaternary

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[1] Helley, E.J. and Harwood, D.S.; 1985; *Geologic map of the Late Cenozoic deposits of the Sacramento Valley and northern Sierran Foothills, California*; United States Geological Survey; Miscellaneous Field Studies Map MF-1790, Sheet 2 of 5; Scale 1:62,500.

deformation of the western Sacramento Valley associated with margin is characterized by uplift, tilting, asymmetric folding, and thrust faulting (Unruh and Moores, 1992). Numerous large earthquakes have occurred along this margin. A Fault Activity Map is presented in **Supplemental Maps**.

Fault Rupture Potential

This site is not located within a State of California Earthquake Fault Zone, as established by the California Geological Survey (Hart 1999; CGS, 2018). The nearest potentially active fault capable of surface rupture is the Dunnigan Hills fault, located approximately 12½ miles southeast of the site. Known faults or fault-related features are not located within this site; therefore, the potential for fault rupture within the site is considered low.

Great Valley Thrust Fault System

The Great Valley thrust fault system marks the boundary between the Coast Ranges and the Great Valley. This fault system can be characterized as a seismically active blind thrust fault and fold system with east-vergent shallow-dipping thrust faults and west-vergent backthrust faults. Based on the site's proximity to this boundary, and the comparatively long distance to strike-slip faulting associated with the San Andreas fault system, the Great Valley fault system may present the most significant seismic hazard to this project. Segregation of this fault system into 18 to 25 sections, based on geomorphology, structural geology, and historical seismicity, was first suggested by Wakabayashi and Smith in 1994. Currently, the fault system consists of 14 sections. The three sections nearest to the site are the Great Valley 03, the Great Valley 02, and the Great Valley 04a.

The Mysterious Ridge section of the Great Valley fault system (GV 03) is located approximately 8 miles to the west of the site and extends southeast from Salt Creek to Esparto area, for a total length of approximately 41 miles (Bryant, 2017). This segment consists of a southeast striking thrust fault dipping to the west. Data suggests that the most recent deformation along this segment occurred between 1.0 and 0.45 million years ago (Ma).

Great Valley 02 section is a west-dipping thrust fault striking approximately north-south. This segment is located approximately 13 miles northwest of the site and has a total length of 17 miles. Quaternary deposits are generally not present along this section, so the age of recent offset can only be estimated as sometime in the last 1.6 Ma (Bryant, 2017).

The Trout Creek and Gordon Valley sections (GV 04a and GV 04b) are southeast striking thrust faults, dipping to the west, located approximately 23 miles southwest of the site. This section is located beneath the English Hills (Bryant, 2017). The most recent displacement on the fault, though not well constrained, is believed to have been less than 130 thousand years ago (ka), based on the deformation of sediments overlying the Plio-Pleistocene Tehama formation.

Hunting Creek-Berryessa Fault System

The active (Holocene) Hunting Creek-Berryessa fault is a system of discontinuous right lateral strike-slip fault traces associated with the San Andreas fault system (Bryant, 2000). The fault zone is located in the Coast Ranges and extends south-southeast from the Wilson Valley region (east of Clear Lake) to the Cedar Roughs area (west of Lake Berryessa). The portion of the fault system closest to the site is located approximately 21 miles to the southwest. Geomorphic features associated with the Hunting Creek-Berryessa fault, as well as trench data showing offset of Holocene-age colluvial deposits, indicate displacement during the last 15 ka.

Bartlett Springs Fault System

The Bartlett Springs fault is an active, northwest striking fault zone at least 0.9 miles wide, consisting of discontinuous normal and right lateral strike-slip faults. The fault system is located in a topographic low that corresponds to a narrow belt of Franciscan mélange and ultramafic rocks. The system extends for at least 74.5 miles (120 km) from Round Valley to the Clear Lake area. This system may join with the Hunting Creek-Berryessa fault system. Evidence of strike-slip displacement during the latest Pleistocene and Holocene includes offset and deflected drainages, shutter ridges and offset landslide deposits (Bryant, 2017).

Historical Earthquakes

A map of earthquake epicenters, compiled from a search of the USGS Earthquake catalog is included with the **Supplemental Maps**. The following table summarizes historic seismic events with a magnitude of 6.0 or greater in the site region based on a search of the USGS Earthquake catalog for events of magnitude 4.5 to 9.0 since 1800, and within a 150-kilometer radius of the site. The search returned 92 results, including 56 events of magnitude 4.5 to 4.9, 28 events of magnitude 5.0 to 5.8, seven (7) events of 6.0 to 6.8, and one (1) event of magnitude 7.0 or greater.

Summary of Historic Seismicity				
Event ID	**Date**	**Magnitude**	**Distance from Site (miles)**	**Direction from Site**
The 1868 Hayward Fault Earthquake	10/21/1868	6.8	91	SSW
South of Cromberg	04/29/1888	6.2	87	NE
North of Antioch	05/19/1889	6.0	65	S
Near Vacaville	04/19/1892	6.6	42½	S
Near Winters	04/21/1892	6.4	36½	S
South of Sonoma	03/31/1898	6.4	61	SSW
The 1906 San Francisco Earthquake	04/18/1906	7.9	91½	SSW
South Napa	08/24/2014	6.0	57	SSW

San Francisco Bay Area Faults

Because of the presence of multiple active faults, the Bay Area region is considered seismically active. Numerous small earthquakes occur every year in the region and large earthquakes have been recorded and can be expected to occur in the future. The magnitude 6.8 Hayward fault earthquake occurred October 21, 1868, approximately 91 miles (145½ km) south of the site. The magnitude 7.9 San Francisco earthquake occurred April 18, 1906, approximately 91½ (146 km) miles southwest of the site.

Coast Range-Great Valley

Although 65% of the faults within the Great Valley fold and thrust system have yet to produce significant seismic events, eleven (11) earthquakes of magnitude 5.8 or greater have occurred on associated fault (Wakabayashi and Smith, 1994). Most of these are centered on the southern sections, including the 1985 Kettleman Hills-North Dome earthquake (Mw 6.1) and the 1993 Coalinga earthquake (Mw 6.5).

The 1892 Vacaville-Winters earthquake sequence likely occurred on the Gordon Valley thrust (O'Connell et al., 2001). Shaking from these earthquakes, as well as the magnitude 5.6 aftershock centered in Dixon, were felt as far north as Redding. According to Stover and Coffman (1993) cracks were observed in walls in Willows (about 36 miles north of the site), and in Esparto (about 22 miles to the south), "*every brick chimney fell and wood-frame buildings were wretched out of shape*".

Other Nearby Faults

Significant historical earthquakes have not been specifically attributed to the Hunting Creek-Berryessa or Bartlett Springs fault zones. The magnitude 5.1 Upper Lake earthquake occurred on August 10, 2016, approximately 2½ miles east of the Bartlett Springs fault and 45 miles northwest of the site.

Inundation by Tsunamis and Seiches

Tsunamis are long period waves, usually produced by a submarine earthquake, volcanic eruption, or landslide. Seiches are an oscillation of a body of water in an enclosed or semi-enclosed basin, mainly caused by local changes in atmospheric pressure aided by tidal currents, winds, and occasionally by earthquakes and landslides. The site is outside of any tsunami hazard zones[1,2], and there are no bodies of water in the immediate vicinity of the site; therefore, tsunamis and seiches are not a potential hazard to the site.

Flooding

The site is not located within a potential inundation zone for seismically-induced dam/reservoir failure. No large water storage facilities are known to exist in the area of the site. Therefore, there is no potential for seismically-induced flooding due to dam failure.

Based on a review of the Federal Emergency Management Agency (FEMA) National Flood Hazard Layer (NFHL), the project site is located within the mapped 100-year flood zone. The project site is in an area with a FEMA Flood Zone AH designation, based on a 1% annual chance flood (100-year flood) with flood depths of 1 to 3 feet and base flood elevations determined.

Liquefaction

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength.

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[1] California Governor's Office of Emergency Services (Cal OES); "My Hazards App"; accessed December 20, 2022; *https://myhazards.caloes.ca.gov/*

[2] American Society of Civil Engineers (ASCE); "ASCE 7 Hazard Tool"; accessed December 22, 2022; *https://asce7hazardtool.online/*

Liquefaction is typically a hazard where loose sandy soils or low plasticity fine grained soils exist below groundwater. The California Geological Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site has not yet been mapped by the CGS for liquefaction hazards.

As part of our evaluation of the liquefaction potential at this site, we reviewed the Geotechnical Investigation Report prepared by Neil O. Anderson and Associates, A Terracon Company, "New Agriculture Building, Pierce High School", dated April 28, 2003. Neil O. Anderson and Associates (NOA) advanced four (4) exploratory borings approximately 500 feet southeast of the proposed swimming pool to depths varying from 15 and 50 feet bgs. Soils encountered during the NOA investigation consisted of 19 feet of stiff to very stiff clay soils underlain by inter-bedded layers of very dense to dense silty sand, gravel, and clay to the maximum depth explored of 50 feet bgs. Soil conditions referenced in the NOA report were similar to those found during this exploration. We did not encounter groundwater during our current exploration but based on boring logs from the previous NOA report, groundwater in this area was measured to be 47 feet bgs.

Liquefiable soils may be present below the maximum depth explored of 50 feet bgs. However, the consequences of one-dimensional settlement may be largely mitigated by the presence of the thick non-liquefied layer above the potentially liquefiable soils (Ishihara 1985, Naesgaard et al. 1998, Bouckovalas and Dakoulas 2007). It is our opinion that the presence of stiff to hard sandy clay soils and dense clayey sand soils (non-liquefiable layer) found beneath the existing ground surface may act as a bridging layer that redistributes stresses and therefore results in more uniform ground surface settlement if there is a deeper liquefiable soil beneath the site. Based on the data presented in the NOA report, the depth to groundwater, and the stiff/dense nature of the underlying strata, the potential for seismically induced liquefaction at this site is considered negligible.

Landslides and Debris Flow

The site is relatively flat with no hills/mountains in the vicinity. The California Geological Survey (CGS) has designated certain areas within California as potential seismically-induced landslide hazard zones. These are areas considered at a risk of slope failure during a seismic event, based upon mapped surficial deposits. The project site has not yet been mapped by the CGS for seismically-induced landslide hazards. Given the site is relatively flat, landslides and debris flows are not considered potential hazards.

Subsidence Potential

Beginning late 2014, DWR began to measure vertical displacement for parts of California using Interferometric Synthetic Aperture Radar (InSAR). Data was collected by the European Space Agency (ESA) Sentinel-1A satellite and processed by TRE ALTAMIRA Inc. Based on a review of this data[1], it appears that the site has experienced approximately -1.39 feet of vertical displacement between February 2015 and July 2022.

While subsidence is potentially damaging to extensive structures, such as irrigation ditches or large water lines, damage to the proposed structures (having relatively small footprint areas) is not anticipated due to the distributed nature of the subsidence area. Organic-rich soils with significant collapse potential were not encountered during our exploration and are not anticipated to be present in the general area of the site. Therefore, the potential for regional subsidence effects at the site is considered low.

Erosion Potential

The subject site is covered with structures and flatwork. Erosion by wind and water is not considered to be a hazard at the site.

Seismic Considerations

Seismic Design Parameters

The 2022 California Building Code (CBC) Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool. This web-based software application calculates seismic design parameters in accordance with ASCE 7, and 2022 CBC. The 2022 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7 for Site Class D sites with a mapped S_s value greater than or equal 0.2.

However, Section 11.4.8 of ASCE 7 includes an exception from such analysis for specific structures on Site Class D sites. The commentary for Section 11 of ASCE 7 (Page 534 of Section C11 of ASCE 7) states that "In general, this exception effectively limits the

[1] California Department of Water Resources (DWR); "SGMA Data Viewer"; accessed December 22, 2022; *https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#landsub*

requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites." Based on our understanding of the proposed structures, it is our assumption that the exception in Section 11.4.8 applies to the proposed structure. However, the structural engineer should verify the applicability of this exception.

Based on this exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC.

Description	Value
2022 California Building Code (CBC) Site Classification[1]	D[2]
Risk Category	III
Site Latitude[3]	39.0119°
Site Longitude[3]	-122.0556°
S_S, Spectral Acceleration for a Short Period[4]	0.937
S_1, Spectral Acceleration for a 1-Second Period[4]	0.364
Fa, Site Coefficient	1.125
Fv, Site Coefficient (1-Second Period)	1.936
S_{DS}, Spectral Acceleration for a Short Period	0.703
S_{D1}, Spectral Acceleration for a 1-Second Period	0.470

1. Seismic site soil classification in general accordance with the *2022 California Building Code*, which refers to ASCE 7. Site Classification is required to determine the Seismic Design Category for a structure.

Description	Value
2. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the CBC. Subsurface explorations at this site were extended to a maximum depth of approximately 21½ feet bgs and the site classification was determined using the standard penetration resistance from the borings performed. The site properties below the maximum exploration depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. Additional deeper exploration or geophysical testing may be performed to confirm the conditions below the current maximum depth of exploration. 3. Provided coordinates represent a point located at the general center of the site. 4. These values were obtained using online seismic design maps and tools provided by SEAOC and OSHPD (**https://seismicmaps.org/**).	

Estimated Ground Motions

The site is located in the northern area of California, which is a relatively moderate seismicity region. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, the intensity, and the magnitude of the seismic event. A Mean Earthquake Magnitude of 6.75 may be considered for this site.

Based on the ASCE 7-16 Standard, the peak ground acceleration (PGA_M) at the subject site is approximately 0.482g. Based on the USGS 2014 interactive deaggregations, the PGA at the subject site for a 2% probability of exceedance in 50 years (return period of 2475 years) is expected to be about 0.505g. The site is not located within an Alquist-Priolo Earthquake Fault Zone based on our review of the State Fault Hazard Maps.[1]

Typically, a site-specific ground motion study may reduce construction costs. We recommend consulting with a structural engineer to evaluate the need for such a study and its potential impact on construction costs. Terracon should be contacted if a site-specific ground motion study is desired.

[1] California Geological Survey (CGS); "California Earthquakes Hazards Zone Application (EQ Zapp)"; accessed December 22, 2022*; https://maps.conservation.ca.gov/cgs/EQZApp/app/*

Corrosivity

The following table lists the results of laboratory soluble sulfate, soluble chloride, electrical resistivity, and pH testing. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary

Boring	Sample Depth (feet)	Soil Description	Soluble Sulfate (%)	Soluble Chloride (%)	Electrical Resistivity (Ω-cm)	pH
B-1	1.0	Sandy Lean Clay	0.01	0.05	3,098	6.7

Results of soluble sulfate testing can be classified in accordance with ACI 318 – Building Code Requirements for Structural Concrete. Numerous sources are available to characterize corrosion potential to buried metals using the parameters above. ANSI/AWWA is commonly used for ductile iron, while threshold values for evaluating the effect on steel can be specific to the buried feature (e.g., piling, culverts, welded wire reinforcement, etc.) or agency for which the work is performed. Imported fill materials may have significantly different properties than the site materials noted above and should be evaluated if expected to be in contact with metals used for construction. Consultation with a NACE certified corrosion professional is recommended for buried metals on the site.

Mapping by the NRCS includes qualitative severity of corrosion to concrete and steel. Based on this source, the near-surface materials are rated "Moderate" for corrosion to concrete and "Moderate" for corrosion of steel.

Geotechnical Overview

The subject site has geotechnical considerations that will affect the construction and performance of the proposed improvements that are discussed in this report. The primary geotechnical considerations that have been identified at the subject site that will affect development of the site are the following:

- Pool Considerations
- Expansive Soils

Pool Considerations

Given the native soils encountered within the area of the swimming pool, the pool may be constructed using a conventional pool shell provided the pool bears into firm native soil. We understand the new pool depths will vary from 3½ to 12 feet deep. Terracon should be contacted to provide additional recommendations as necessary if this is not the case.

Additional geotechnical design considerations for the swimming pool and items that may affect the future geotechnical stability of the pool system are listed below.

- **Isolate pool shell** – The proposed pool should be isolated from any source that could cause additional settlement of the pool. Foundations from buildings and other structures related to the pool should be kept a minimum distance equal to the depth of the pool from the pool's edge to reduce the effect of the foundation on the pool shell. Additionally, pool decks should not be tied into the pool shell.
- **Groundwater concerns** – Groundwater was not encountered in our borings at the time of our field exploration. The presence of groundwater could cause the pool shells to float if the pool is emptied. If groundwater or saturated soil conditions are encountered during construction or are anticipated at any time of the year, a hydrostatic pressure relief valve should be installed in the deep end of the pool and an underdrain should be placed below the floor of the pool in accordance with the recommendations provided in the **Pool Recommendations** section of this report.
- **Avoid fill material below the pool** – Fill material placed below the pool is to be avoided due to the potential for excessive differential settlements within the fill material. This includes documented fills that have been placed correctly.
- **Avoid surcharge loading on pool shell** – The addition of surcharge loads on the pool shell either during construction or after construction should be avoided to limit the possibility of damaging the pool walls.

Expansive Soils

Expansive soils are present on this site. This report provides recommendations to help mitigate the effects of soil shrinkage and expansion. However, even if these procedures are followed, some movement and (at least minor) cracking in the restroom/storage building should be anticipated. The severity of cracking and other damage such as uneven floor slabs will probably increase if modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. Some of these options include complete replacement of expansive soils, using a structural slab, or supporting the improvements on deep foundations.

The near surface, stiff to very stiff lean clay could become unstable with typical earthwork and construction traffic, especially after precipitation events. The effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

An Expansion Index test was ran on a bulk sample collected in the area of the proposed restroom/storage building and indicated that the near surface clay soils have low expansion potential. The soils which form the bearing stratum for shallow foundations have low to moderate plasticity and exhibit potential for shrink-swell movements with changes in moisture. Additional areas of localized moderately to highly plastic soils may be present where borings were not performed. Maintaining above optimum moisture conditions in the bearing soils and a minimum dead load pressure on footings should reduce the anticipated swell movements to tolerable levels. The **Shallow Foundations** section addresses support of the restroom/storage building bearing on firm native soil. We do not expect significant dead load on the floors and recommend overexcavation of near-surface clays to reduce the heave potential. The **Floor Slabs** and **Earthwork** sections address slab-on-grade support of the building using overexcavation techniques.

The recommendations contained in this report are based upon the results of field and laboratory testing (presented in the **Exploration Results**), engineering analyses, and our current understanding of the proposed project. The **General Comments** section provides an understanding of the report limitations.

Earthwork

We anticipate grading may consist of cuts and fills on the order of 2 feet or less and that site grades will remain at the same elevation as existing in the planned building and pool area. Grading for the new swimming pool may consist of excavations up to 12 feet below existing grades. Specific site grading information was unavailable at the time this report was prepared. If elevation and site grading differ from our stated assumptions, Terracon should be contacted to determine if additional earthwork recommendations are warranted, particularly with regard to potential ground settlement.

Earthwork is anticipated to include demolition, clearing, excavations, and engineered fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for the swimming pool, building foundations, floor slabs, and exterior hardscapes.

Demolition

The proposed building and swimming pool will be constructed within the footprint of existing tennis courts which will need to be demolished, along with exterior sidewalks, pavements, and utilities. We recommend existing tennis courts, pavements and utilities be removed from within the proposed building, swimming pool, and pool decking footprints and at least 5 feet beyond the outer edge of the improvements. If pipes are abandoned in-place, they should be filled completely with lean cement grout, or other suitable material, to avoid collapse in the future. Pipes that daylight into the planned pool excavation should be capped. All materials derived from the demolition of existing structures and pavements should be removed from the site and not be allowed for use as on-site fill, unless processed in accordance with the fill requirements included in this report.

For areas outside the proposed building, swimming pool, and pool decking footprints, existing pavements and utilities should be removed where they conflict with proposed utilities and hardscapes. In such cases, existing pavements and utilities should be removed to a depth of at least 2 feet below the affected utility or design hardscape subgrade elevation.

Site Preparation

Prior to placing fill, existing vegetation, topsoil, debris and pavements should be removed. Complete stripping of the topsoil should be performed in the proposed building, swimming pool, and pool decking areas. Stripping should extend laterally a minimum of 5 feet beyond the limits of proposed improvements.

Mature trees are located near the footprint of the proposed building, which may require removal at the onset of construction. Tree root systems can remove substantial moisture from surrounding soils. Where trees are removed, the full root ball and all associated dry and desiccated soils should be removed. The soil materials which contain less than 3 percent organics can be reused as engineered fill provided the material meets the specifications for general or structural fill.

Although no evidence of fill or underground facilities (such as septic tanks, cesspools, basements, and utilities) was observed during the exploration and site reconnaissance, such features could be encountered during construction. If unexpected fills or underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Subgrade Preparation

After clearing, any required cuts and over-excavation should be made.

We recommend that the soils within the floor slab area of the proposed building and exterior hardscape (pool decking) areas be removed to a minimum depth of 12 inches. The near-surface materials anticipated to be developed as excavation spoils are not considered suitable for use as structural fill.

Once cuts and over-excavation operations are complete, the resulting subgrade should be proofrolled with an adequately loaded vehicle such as a fully-loaded tandem-axle dump truck. The proofrolling should be performed under the observation of the Geotechnical Engineer or their representative. Areas excessively deflecting under the proofroll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed or modified by stabilizing as noted in the following section ***Soil Stabilization***. Excessively wet or dry material should either be removed, or moisture conditioned and recompacted.

Excavated material may be stockpiled for use as fill provided it is cleaned of organic material, debris, and any other deleterious material and meets the criteria for general or structural fill specified in the ***Fill Material Types*** section of this report.

Once proof rolling has been performed, all exposed areas which will receive fill, once properly cleared and benched where necessary, should be scarified and moisture conditioned as necessary, and compacted per the compaction requirements in this report. Scarification and compaction is not required in the bottom of the pool and pit excavations. The depth of scarification of subgrade soils and moisture conditioning of the subgrade is highly dependent upon the time of year of construction and the site conditions that exist immediately prior to construction. If construction occurs during the winter or spring, when the subgrade soils are typically already in a moist condition, scarification and compaction may only be 8 inches. If construction occurs during the summer or fall when the subgrade soils have been allowed to dry out deeper, the depth of scarification and moisture conditioning may be as much as 18 inches or more. A representative from Terracon should be present to observe the exposed subgrade and confirm the depth of scarification and moisture conditioning required.

The exposed subgrade will likely be wet because it has been covered by tennis courts and pavement. Subsequently, the depth of required scarification and moisture conditioning of the subgrade may only be 6 to 8 inches.

Following scarification, moisture conditioning, and compaction of the subgrade soils, compacted engineered fill soils should then be placed to the proposed design grade and the moisture content and compaction of subgrade soils should be maintained until foundation or hardscape construction.

Based upon the subsurface conditions determined from the geotechnical exploration, subgrade soils exposed during construction are anticipated to be relatively workable; however, the workability of the subgrade may be affected by precipitation, repetitive construction traffic or other factors. If unworkable conditions develop, workability may be improved by scarifying and drying.

Excavation

We anticipate that excavations for the proposed construction can be accomplished with conventional earthmoving equipment. The bottom of excavations should be thoroughly cleaned of loose soils and disturbed materials prior to backfill placement and/or construction.

Individual contractors are responsible for designing and constructing stable, temporary excavations. Excavations should be sloped or shored in the interest of safety following local, and federal regulations, including current OSHA excavation and trench safety standards.

Soil Stabilization

Depending on the time of year, precipitation may create excessively moist soils which may require improving the subgrade prior to constructing the proposed development. Methods of subgrade improvement, as described below, could include scarification, moisture conditioning and recompaction, removal of unstable materials and replacement with granular fill (with or without geosynthetics). The appropriate method of improvement, if required, would be dependent on factors such as schedule, weather, the size of area to be stabilized, and the nature of the instability. More detailed recommendations can be provided during construction as the need for subgrade stabilization occurs. Performing site grading operations during warm seasons and dry periods would help reduce the amount of subgrade stabilization required.

If the exposed subgrade is unstable during proofrolling operations, it could be stabilized using one of the following methods.

- **Scarification and Recompaction** - It may be feasible to scarify, dry, and recompact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Stable subgrades likely would not be achievable if the thickness of the unstable soil is greater than about 1 foot, if the unstable soil is at or near groundwater levels, or if construction is performed during a period of wet or cool weather when drying is difficult.
- **Aggregate Base** - The use of Caltrans Class II aggregate base is a common procedure to improve subgrade stability. Typical undercut depths would be expected to range from about 12 to 18 inches below finished subgrade elevation. The use of high modulus geosynthetics (i.e., engineering fabric or geogrid) could also be considered after underground work such as utility construction is

completed. Prior to placing the fabric or geogrid, we recommend that all below grade construction, such as utility line installation, be completed to avoid damaging the fabric or geogrid. Equipment should not be operated above the fabric or geogrid until one full lift of aggregate base is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should meet the manufacturer's specifications.

- **Chemical Stabilization** - Improvement of subgrades with Portland cement or quicklime could be considered for improving unstable soils. Chemical stabilization should be performed by a pre-qualified contractor having experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions. The hazards of chemicals blowing across the site or onto adjacent property should also be considered. Additional testing would be needed to develop specific recommendations to improve subgrade stability by blending chemicals with the site soils. Additional testing could include, but not be limited to, determining the most suitable stabilizing agent, the optimum amounts required, and the presence of sulfates in the soil. If this method is chosen to stabilize subgrade soils the actual amount of high calcium quicklime/Portland cement to be used should be determined by Terracon and by laboratory testing **at least three weeks prior** to the start of grading operations.

Further evaluation of the need and recommendations for subgrade stabilization can be provided during construction as the geotechnical conditions are exposed.

Fill Material Types

Fill required to achieve design grade should be classified as structural fill and general fill. Structural fill is material used below, or within 5 feet of structures or hardscapes. General fill is material used to achieve grade outside of these areas.

Reuse of On-Site Soil: Excavated on-site soil may be selectively reused as general or structural fill. Portions of the on-site soil have an elevated fines content and will be sensitive to moisture conditions (particularly during seasonally wet periods) and may not be suitable for reuse when above optimum moisture content.

Material property requirements for on-site soil for use as general fill and structural fill are noted in the following table:

Property	General Fill	Structural Fill
Composition	Free of deleterious material	Free of deleterious material

Property	General Fill	Structural Fill
Maximum particle size	6 inches (or 2/3 of the lift thickness)	3 inches
Fines content	Not limited	Less than 30% Passing No. 200 sieve
Plasticity	Not limited	Maximum plasticity index of 10 Expansion Index less than 20
GeoModel Layer Expected to be Suitable[1]	2, 3, 4	3, 4

1. Based on subsurface exploration. Actual material suitability should be determined in the field at time of construction.

Imported Fill Materials: Imported fill materials should meet the following material property requirements. Regardless of its source, compacted fill should consist of approved materials that are free of organic matter and debris. For all import material, the contractor shall submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current ACI criteria and is "mildly corrosive" to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the project.

Soil Type [1]	USCS Classification	Acceptable Parameters (for Structural Fill)
Low Plasticity	CL, SC	Liquid Limit less than 30 Plasticity index less than 10 Expansion index less than 20
Granular[2]	GW, GM, SW, SM	Less than 50% passing No. 200 sieve

1. Structural and general fill should consist of approved materials free of organic matter and debris and should contain no material larger than 3 inches in greatest dimension. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation at least two weeks prior to use on this site. Additional geotechnical consultation should be provided prior to use of uniformly graded gravel on the site.
2. Caltrans Class II aggregate base may be used for this material. Recycled aggregate base should not be used.

Fill Placement and Compaction Requirements

Compacted native soil and structural and general fill should meet the following compaction requirements.

Item	Structural Fill	General Fill
Maximum Lift Thickness	8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used	Same as structural fill
Minimum Compaction Requirements [1,2]	95% of max. for structural fill below foundations and slabs, within 1 foot of finished hardscape subgrade, for aggregate base and for fills thicker than 5 feet 90% of max. for all other locations	90% of max.
Water Content Range [1]	Low plasticity cohesive: +1% to +3% above optimum Medium plasticity cohesive: +2% to +4% above optimum Granular: -2% to +2% of optimum	As required to achieve min. compaction requirements

1. Maximum density and optimum water content as determined by the Modified Proctor test (ASTM D 1557).
2. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D 4253 and D 4254). Materials not amenable to density testing should be placed and compacted to a stable condition observed full time by the Geotechnical Engineer or representative.

Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with structural fill or bedding material in accordance with public works specifications for the utility be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1:1 projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

On-site materials may be used for backfill of utility and pipe trenches from 1 foot above the top of the pipe to the final ground surface in areas providing structural support to foundations, slabs, or exterior hardscape provided they meet the specifications for structural fill and are free of organic matter and deleterious substances and may be used as backfill outside site these areas provided the material meets the specifications for general fill.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Flooding or jetting for placement and compaction of backfill is not recommended.

All trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If utility trenches are backfilled with relatively clean granular material, they should be capped with at least 18 inches of cementitious flowable fill or cohesive fill in non-pavement areas to reduce the infiltration and conveyance of surface water through the trench backfill. Attempts should also be made to limit the amount of fines migration into the clean granular material. Fines migration into clean granular fill may result in unanticipated localized settlements over a period of time. To help limit the amount of fines migration, Terracon recommends the use of a geotextile fabric that is designed to prevent fines migration in areas of contact between clean granular material and fine-grained soils. Terracon also recommends that clean granular fill be tracked or tamped in place where possible in order to limit the amount of future densification which may cause localized settlements over time.

For low permeability subgrades, utility trenches are a common source of water infiltration and migration. Utility trenches penetrating beneath the restroom/storage building should be effectively sealed to restrict water intrusion and flow through the trenches, which could migrate below the building. The trench should provide an effective trench plug that extends at least 5 feet from the face of the building's exterior. The plug material should consist of cementitious flowable fill or low permeability clay. The trench plug material should be placed to surround the utility line. If used, the clay trench plug material should be placed and compacted to comply with the water content and compaction recommendations for structural fill stated previously in this report.

Grading and Drainage

All grades must provide effective drainage away from the improvements during and after construction and should be maintained throughout the life of the structure. Water retained next to the improvements can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation and pool shell movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge onto splash blocks a distance of at least 10 feet from the building, onto pavements, or are tied to tight lines that discharge into a storm drain system.

Exposed ground should be sloped and maintained at a minimum 5 percent away from the improvements for at least 10 feet beyond the perimeter of the improvements. If a minimum 5 percent slope cannot be achieved due to site grades, a minimum 2½ percent slope could be used provided pavement or hardscape surrounds and extends to the improvements, or a subdrain could be installed around the perimeter of the foundations that carries water away from the building. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structures should also be periodically inspected and adjusted, as necessary, as part of the structures' maintenance program. Where paving or flatwork abuts the structures, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Any planters located within 10 feet of the building or swimming pool should be self-contained or lined with an impermeable membrane to prevent water from accessing subgrade soils below the building and around the swimming pool. Sprinkler mains and spray heads should be located a minimum of 5 feet away from the pool and foundation lines.

No vegetation over six feet in height shall be planted within 20 feet of the building perimeter or swimming pool unless a root barrier is provided between the structure and tree to limit roots within 10 feet of building and swimming pool. Roots can draw additional moisture from the soils and cause excessive volume changes in the soil resulting in building and swimming pool movement.

Implementation of adequate drainage for this project can affect the surrounding developments. Consequently, in addition to designing and constructing drainage for this project, the effects of site drainage should be taken into consideration for the planned structures on this property, the undeveloped portions of this property, and surrounding sites. Extra care should be taken to ensure irrigation and drainage from adjacent areas do not drain onto the project site or saturate the construction area.

Earthwork Construction Considerations

Excavations for the proposed structures are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of grade-supported improvements such as floor slabs and exterior hardscapes. Construction traffic over the completed subgrades should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade should become desiccated, saturated, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to construction.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through April) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork operations may require additional mitigation measures beyond that which would be expected during the drier summer and fall months. This could include ground stabilization utilizing chemical treatment of the subgrade, diversion of surface runoff around exposed soils, and draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local and/or state regulations. Stockpiles of soil, construction materials, and construction equipment should not be placed near trenches or excavations. ***The Contractor is responsible for maintaining the stability of adjacent structures during construction.***

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Excavations or other activities resulting in ground disturbance have the potential to affect adjoining properties and structures. Our scope of services does not include review of available final grading information or consider potential temporary grading performed by the contractor for potential effects such as ground movement beyond the project limits. A preconstruction/ precondition survey should be conducted to document nearby property/infrastructure prior to any site development activity. Excavation or ground disturbance activities adjacent or near property lines should be monitored or instrumented for potential ground movements that could negatively affect adjoining property and/or structures.

Construction Observation and Testing

The earthwork efforts should be observed by the Geotechnical Engineer (or others under their direction). Observation should include documentation of adequate removal of surficial materials (vegetation, topsoil, debris, and pavements) as well as proofrolling and mitigation of unsuitable areas delineated by the proofroll.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, as recommended by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 1,500 square feet of compacted fill in the building and exterior hardscape areas. Where not specified by local ordinance, one density and water content test should be performed for every 50 linear feet of compacted utility trench backfill and a minimum of one test performed for every 12 vertical inches of compacted backfill.

In areas of pool and foundation excavations, the bearing subgrade should be evaluated by the Geotechnical Engineer. If unanticipated conditions are observed, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

Pool Recommendations

The pool shell may be constructed as a conventional in-ground pool shell provided the pool bears into firm native soil. The pool excavation should be observed by a Terracon engineer or geologist to verify suitable bearing material has been required. Loose or soft soils at the bottom of the pool excavation should be over-excavated to firm native soil. Areas where over-excavation may be required due to the presence of loose or soft soil may be backfilled with a 2-sack lean concrete mix or Caltrans Class II aggregate base compacted to a minimum 95 percent relative compaction.

Pool walls should be designed to resist a lateral earth pressure of 85 pounds per cubic foot (pcf) equivalent fluid pressure for walls with flat backfill. Use of this lateral earth pressure assumes the pool shell will be shot or placed directly against a firm native soil cut. No drainage is required behind the pool walls around the perimeter of the pools. Expansive soils within the pool excavation should be maintained in a moist condition during construction and should not be allowed to dry out.

If groundwater or saturated soils are encountered during construction or are anticipated in the pool area at any time of the year, a hydrostatic pressure relief system should be installed in the deep end of the pool and the pool should be underlain by a minimum 6-inch thick layer of 3/4-inch clean gravel underlain by Mirafi 140N filter fabric, or Caltrans Class II permeable material. A 4-inch diameter perforated Schedule 40 PVC or ABS pipe should be installed in the gravel at the deepest point. The perforated pipe should slope at a 2 percent minimum grade to a tight line at the edge of the pool that carries the drainage to an observation well where water can removed by pumping.

Shallow Foundations

The proposed restroom/storage building may be supported by spread footings. If the site has been prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable for shallow foundations.

Design Parameters – Compressive Loads

Item	Description
Maximum Net Allowable Bearing Pressure [1, 2]	2,500 psf
Required Bearing Stratum [3]	Firm native soil
Minimum Foundation Dimensions	Per CBC 1809.7
Maximum Foundation Dimensions	Columns: 6 feet Continuous: 3 feet
Passive Resistance[4] **(equivalent fluid pressures)**	250 pcf
Sliding Resistance [5]	130 psf allowable cohesion (native clay)
Minimum Embedment below Finished Grade [6]	18 inches
Estimated Total Settlement from Structural Loads [2]	Less than about 1 inch
Estimated Differential Settlement [2, 7]	About ½ of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. This bearing pressure can be increased by 1/3 for transient loads unless those loads have been factored to account for transient conditions. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.
2. Values provided are for maximum loads noted in **Project Description**. Additional geotechnical consultation will be necessary if higher loads are anticipated.
3. Unsuitable or soft soils should be overexcavated and replaced per the recommendations presented in **Earthwork**.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face. Assumes no hydrostatic pressure.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. For fine-grained materials, lateral resistance using cohesion should not exceed ½ the dead load.
6. Embedment necessary to minimize the effects of seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.

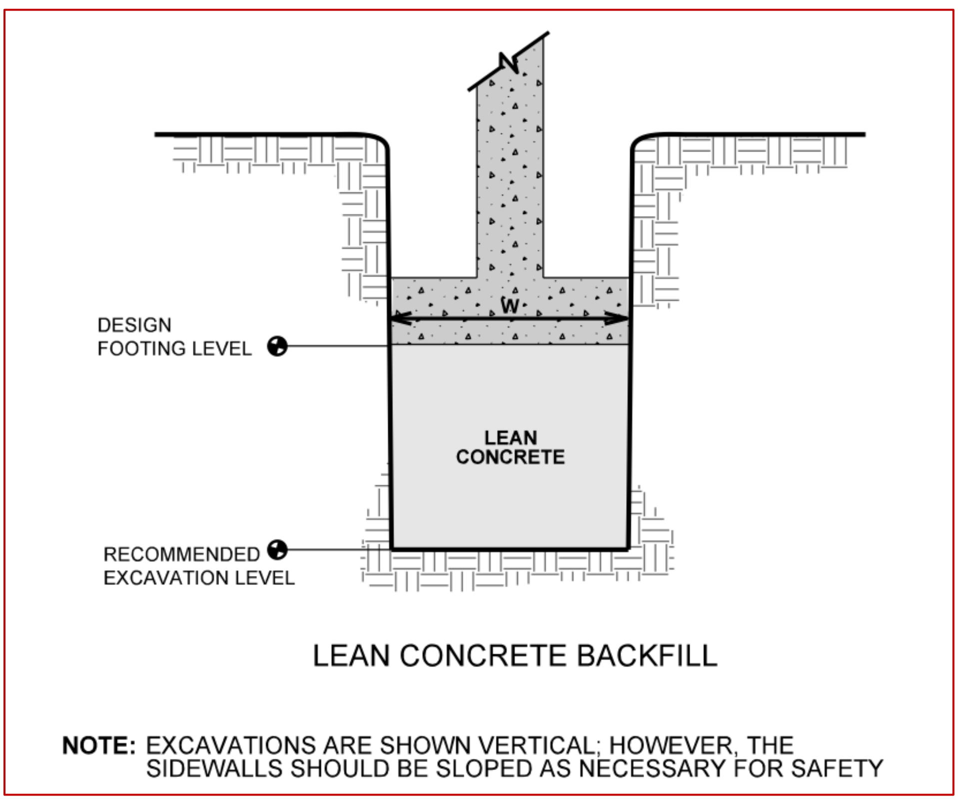
Item	Description

7. Differential settlements are noted for equivalent-loaded foundations and bearing elevation as measured over a span of 50 feet.

Foundation Construction Considerations

As noted in **Earthwork**, the footing excavations should be evaluated under the observation of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

If unsuitable bearing soils are observed at the base of the planned footing excavation, the excavation should be extended deeper to suitable soils, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. The lean concrete replacement zone is illustrated on the following sketch.



To ensure foundations have adequate support, special care should be taken when footings are located adjacent to trenches. The bottom of such footings should be at least

1 foot below an imaginary plane with an inclination of 1.5 horizontal to 1.0 vertical extending upward from the nearest edge of the adjacent trench.

Floor Slabs

Design parameters for floor slabs assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the floor slab support course beneath the floor slab.

The subgrade soils are comprised of plastic clays exhibiting the potential to shrink/swell with variations in water content. Construction of the floor slabs and revising site drainage creates the potential for gradual increased water contents within the clays. Increases in water content will cause the clays to swell and damage floor slabs. To reduce the potential effects of the plastic clays on the building floor slab, at least the upper 18 inches of subgrade soils below the floor slab (excluding the floor slab support course) should consist of an approved granular structural fill material.

Due to the potential for significant moisture fluctuations of subgrade material beneath floor slabs supported at-grade, the Geotechnical Engineer should evaluate the moisture condition of the material within 18 inches of the bottom of the structural fill zone immediately prior to placement of the structural fill. Samples of the subgrade soils should be obtained for moisture content testing. Soils below the specified water contents within this zone should be moisture conditioned or replaced with structural fill as stated in our **Earthwork** section.

Floor Slab Design Parameters

Item	Description
Floor Slab Support[1]	Minimum 4 inches of ¾ inch free draining crushed aggregate[3] overlying at least 12 inches of compacted granular structural fill. Subgrade compacted to the recommendations in **Earthwork**
Estimated Modulus of Subgrade Reaction [2]	50 pounds per square inch per inch (psi/in) for point loads

1. Floor slabs should be structurally independent of building footings or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
2. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the

Item	Description
	floor slab support as noted in this table. It is provided for point loads. For large area loads the modulus of subgrade reaction would be lower. 3. Free-draining granular material should have less than 5% fines (material passing the No. 200 sieve).

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, when the project includes humidity-controlled areas, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut contraction joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations, refer to the ACI Design Manual. Joints or cracks should be sealed with a waterproof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing, or other means.

Floor Slab Construction Considerations

Finished subgrade, within and for at least 10 feet beyond the floor slab, should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of the floor slab, the affected material should be removed, and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

The Geotechnical Engineer should observe the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

Exterior Hardscape

In order to help protect the exterior hardscape (pool decks) against the swell pressure of the surficial low to moderate plasticity clays, we recommend the subgrade soil below hardscapes be over-excavated to a minimum depth of 12 inches and replaced with compacted granular structural fill per the recommendations provided in this report. The Geotechnical Engineer should evaluate the moisture condition of the material within 18 inches of the bottom of the structural fill zone immediately prior to placement of the structural fill. Samples of the subgrade soils should be obtained for moisture content testing. Soils below the specified water contents within this zone should be moisture conditioned or replaced with structural fill as stated in our **Earthwork** section.

Exterior hardscape may experience some movement due to the volume change of the subgrade soils. To reduce the potential for damage caused by movement, we recommend:

- Slabs should be underlain by a minimum of 12 inches of compacted granular structural fill as indicated. At the contractor's discretion, gravel may be placed between the slab and granular structural fill to assist with constructability.
- Minimizing moisture increases in the subgrade soils and backfill;
- Controlling moisture-density during placement of fill;
- Using designs which allow vertical movement between the exterior features and adjoining structural elements;
- Placing effective control joints on relatively close centers.
- Ensure clay subgrade soils are in a moist condition prior to slab construction.
- Reinforce exterior slabs and flatwork with a minimum No. 4 bars at 12 inches on center.

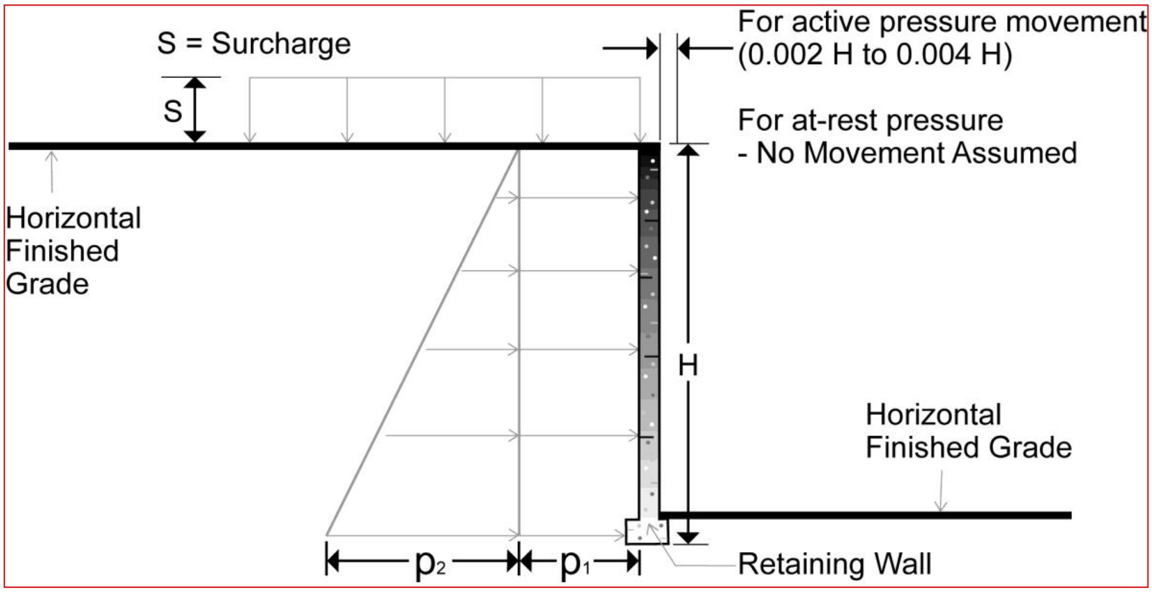
Pool decking slabs should remain structurally independent from the swimming pool shell.

Lateral Earth Pressures

Design Parameters

We understand a below-grade pump pit and surge tank will be constructed as part of the proposed swimming pool construction. We anticipate construction of these below-grade improvements will consist of cantilevered concrete or shotcrete retaining walls. The lateral earth pressure recommendations given in the following paragraphs are applicable to the design of retaining walls subject to slight rotation and rigid retaining or below grade walls, such as cantilever or gravity type concrete walls.

Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction, and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the diagram below. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters

Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3] p_1 (psf)	Equivalent Fluid Pressures (psf) [2,4]	
			Unsaturated [5]	Submerged [5]
Active (Ka)	Granular Structural Fill - 0.31	(0.31)S	(40)H	(85)H
	Native sandy lean clay soils – 0.49	(0.49)S	(65)H	(100)H
At-Rest (Ko)	Granular Structural Fill - 0.47	(0.47)S	(60)H	(100)H
	Native sandy lean clay soils -0.66	(0.66)S	(85)H	(115)H

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance. Fat clay or other expansive soils should not be used as backfill behind the wall.

Lateral Earth Pressure Design Parameters

Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3] p_1 (psf)	Equivalent Fluid Pressures (psf) [2,4]	
			Unsaturated [5]	Submerged [5]

2. Uniform, horizontal backfill, with a maximum unit weight of 125 pcf.
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included.
5. To achieve "Unsaturated" conditions, follow guidelines in the following **Subsurface Drainage for Below-Grade Walls** section of this report. "Submerged" conditions are recommended when drainage behind walls is not incorporated into the design.
6. Values in the above table are for <u>flat backfill only.</u>

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 degrees from vertical for the active case.

Total lateral earth pressure acting on below grade walls during a seismic event will likely include the active or at-rest static force and a dynamic increment. The dynamic increment should be applied to the wall as resultant force acting at 0.33H height from the base of the wall. Such increments should be added to the static earth pressures. A dynamic lateral earth resultant force of $7H^2$ (in units of pounds per linear foot (plf), where H (in units of feet) is the height of the soil behind the wall[1] should be used in design.

Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures more than those provided. Compaction of each lift adjacent to wall should be accomplished with hand-operated tampers for other lightweight compactors. Over-compaction may cause excessive lateral earth pressures which could result in wall movement.

Footings, floor slabs, or other loads bearing on backfill behind walls may have a significant influence on the lateral earth pressure. Placing footings within wall backfill and in the zone of active soil influence on the wall should be avoided unless structural analyses indicate the wall can safely withstand the increased pressure.

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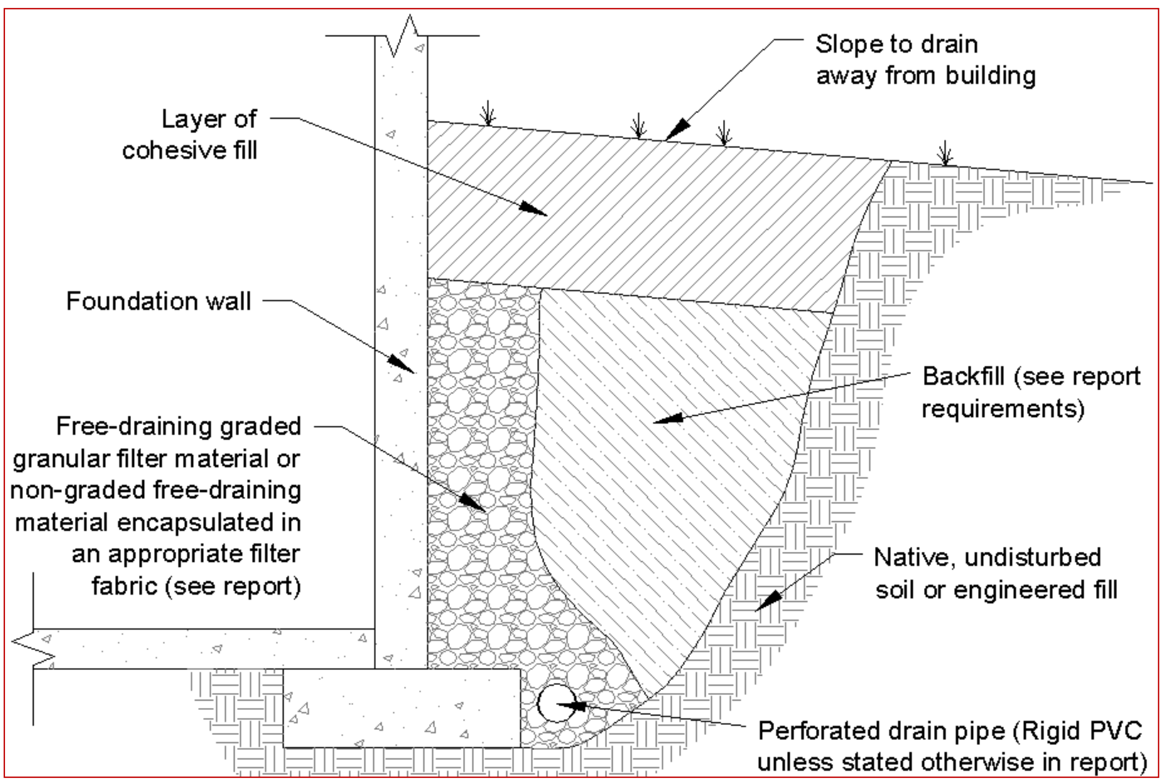
[1] *Seed & Whitman (1970)*

The lateral earth pressure recommendations given in this section are applicable to the design of rigid retaining walls subject to slight rotation, such as cantilever, or gravity type concrete walls. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill walls (also termed MSE walls). Recommendations covering these types of wall systems are beyond the scope of services for this assignment. However, we would be pleased to develop a proposal for evaluation and design of such wall systems upon request.

Subsurface Drainage for Below-Grade Walls

A perforated rigid plastic drain line installed behind the base of walls and extends below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around a below-grade building area or exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material having less than 5% passing the No. 200 sieve, such as No. 57 aggregate. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should extend to within 12 inches of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system.



As an alternative to free-draining granular fill, a prefabricated drainage composite may be used. A prefabricated drainage composite is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing. This report should not be used after 3 years without written authorization from Terracon.

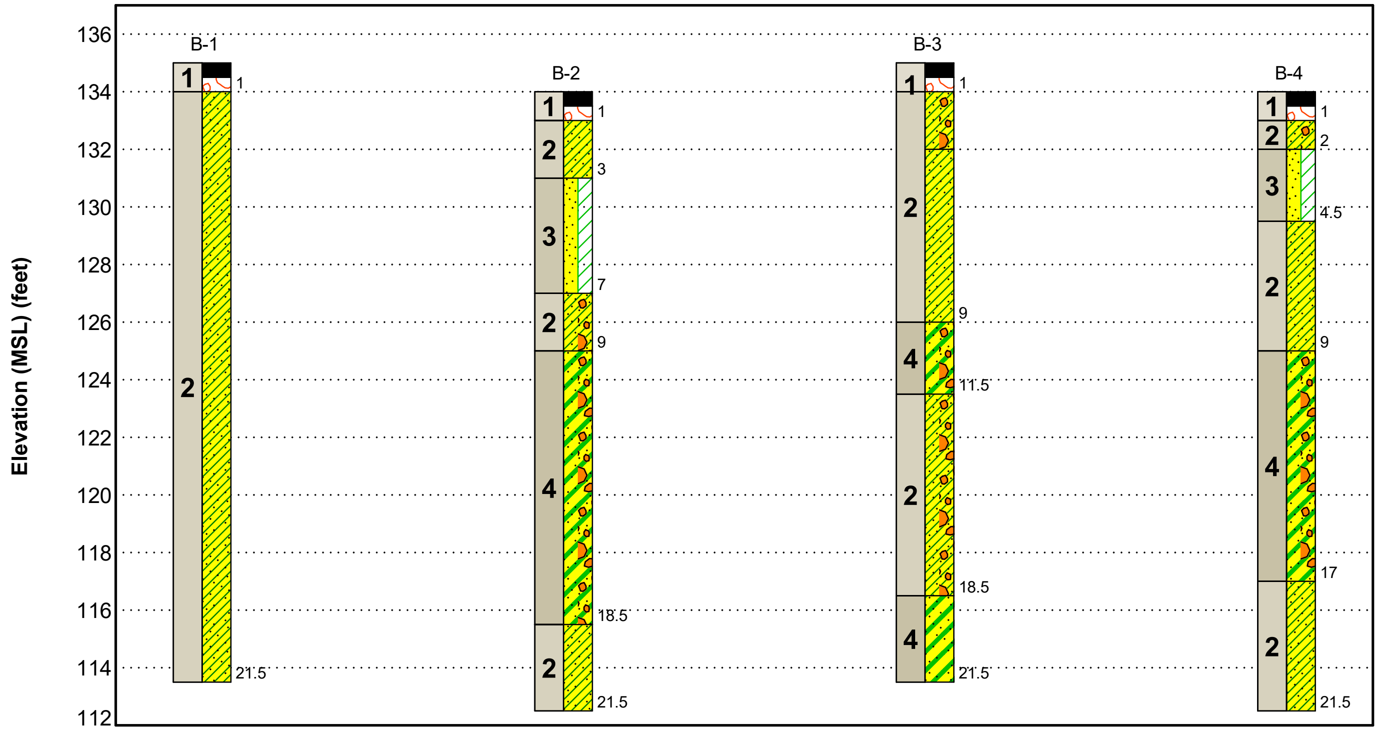
Figures

Contents:

Pierce HS Pool Complex
960 Wildwood Road | Arbuckle, CA
Terracon Project No. NB225033

50 Golden Land Ct Ste 100
Sacramento, CA

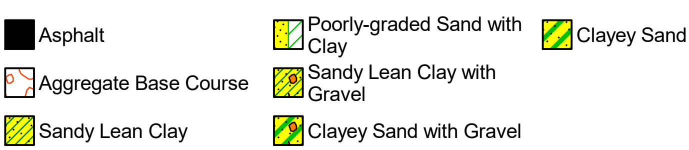
Geomodel



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	**Surfacing**	Approximately 6 inches of asphalt overlying 6 inches of aggregate base course.
2	**Sandy Lean Clay**	Medium plasticity, stiff to hard sandy lean clay with variable amounts of gravel. Gravel up to 2 inches in dimension.
3	**Poorly Graded Sand with Clay**	Medium dense poorly graded sand with clay with variable amounts of gravel. Gravel up to 1.5 inches in dimension.
4	**Clayey Sand**	Medium dense to dense clayey sand with variable amounts of gravel. Gravel up to 2 inches in dimension.

LEGEND



NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project. Numbers adjacent to soil column indicate depth below ground surface.

Attachments

Exploration and Testing Procedures

Field Exploration

Number of Borings	Approximate Boring Depth (feet)	Location
2	21½	Restroom/storage building
2	21½	Pool area

Boring Layout and Elevations: Terracon personnel provided the boring layout using handheld GPS equipment (estimated horizontal accuracy of about ±10 feet) and referencing existing site features. Approximate ground surface elevations were obtained by interpolation from Google Earth. If elevations and a more precise boring layout are desired, we recommend the exploration locations be surveyed.

Subsurface Exploration Procedures: We advanced the borings with a truck-mounted rotary drill rig using continuous solid stem flight augers. Four samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet or less thereafter. In the split barrel sampling procedure, a standard 2-inch outer diameter split barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. A 3.0-inch O.D. split-barrel sampling spoon with 2.5-inch I.D. ring lined sampler was also used for sampling. Ring-lined, split-barrel sampling procedures are similar to standard split spoon sampling procedure; however, blow counts are typically recorded for 6-inch intervals for a total of 12 inches of penetration. For safety purposes, all borings were backfilled with neat cement-grout after their completion. Pavements were patched with cold-mix asphalt, as appropriate.

We also observed the boreholes while drilling and at the completion of drilling for the presence of groundwater. Groundwater was not observed at these times in the boreholes.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by an engineer and geologist. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials observed during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM 7263 Standard Test Methods for Laboratory Determination of Density and Unit Weight of Soil Specimens
- ASTM D2166 Standard Test Methods for Unconfined Compressive Strength of Cohesive Soil
- ASTM D1140 Standard Test Method for Determining the Amount of Material Finer than No. 200 Sieve by Soil Washing
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils
- ASTM D4829 Standard Test Method for Expansion Index of Soils
- Corrosivity Testing including pH, chlorides, sulfates, sulfides, RedOx potential, and electrical lab resistivity

The laboratory testing program often included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the Unified Soil Classification System.

Site Location and Exploration Plans

Contents:

Note: All attachments are one page unless noted above.

Site Location

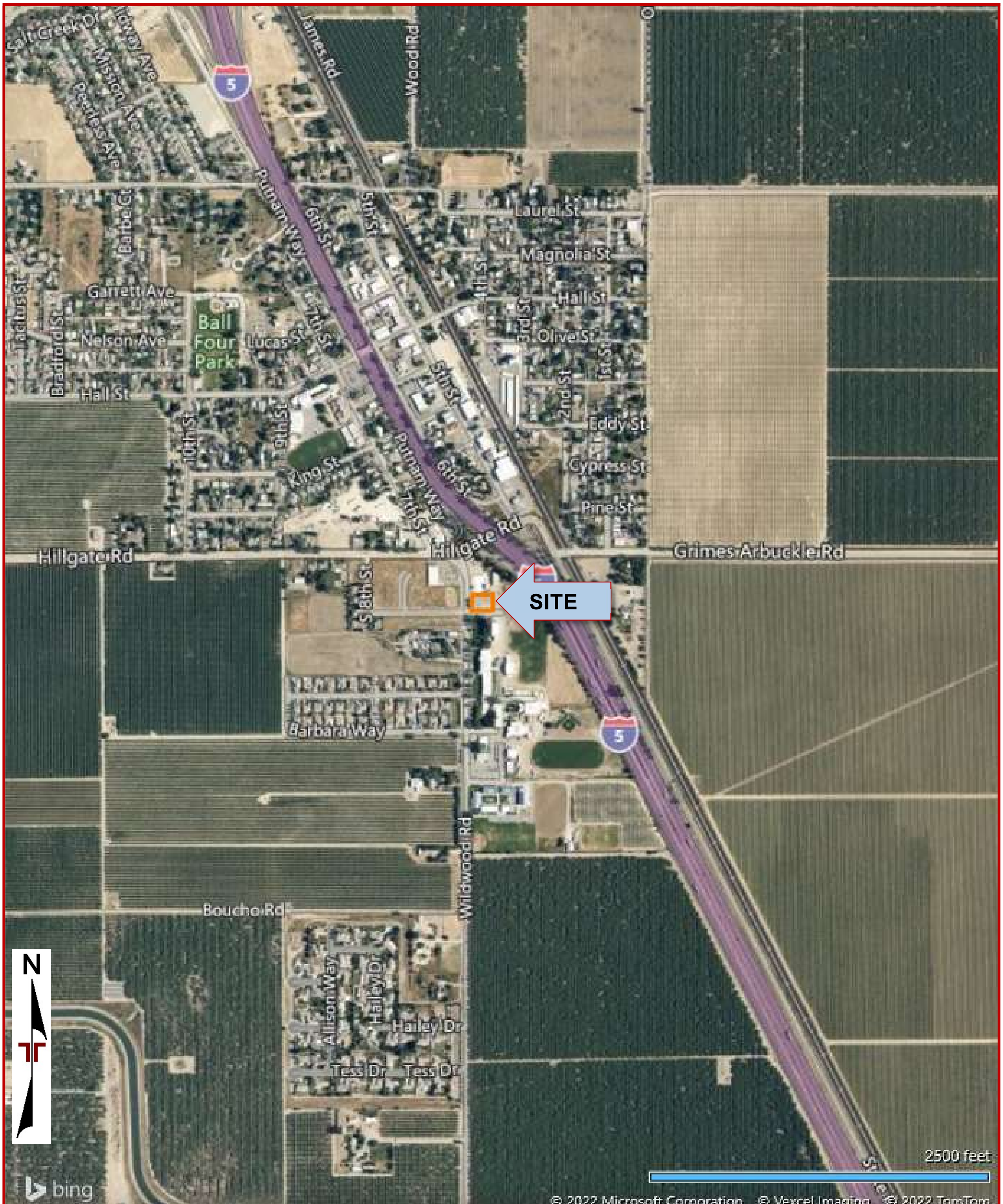


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

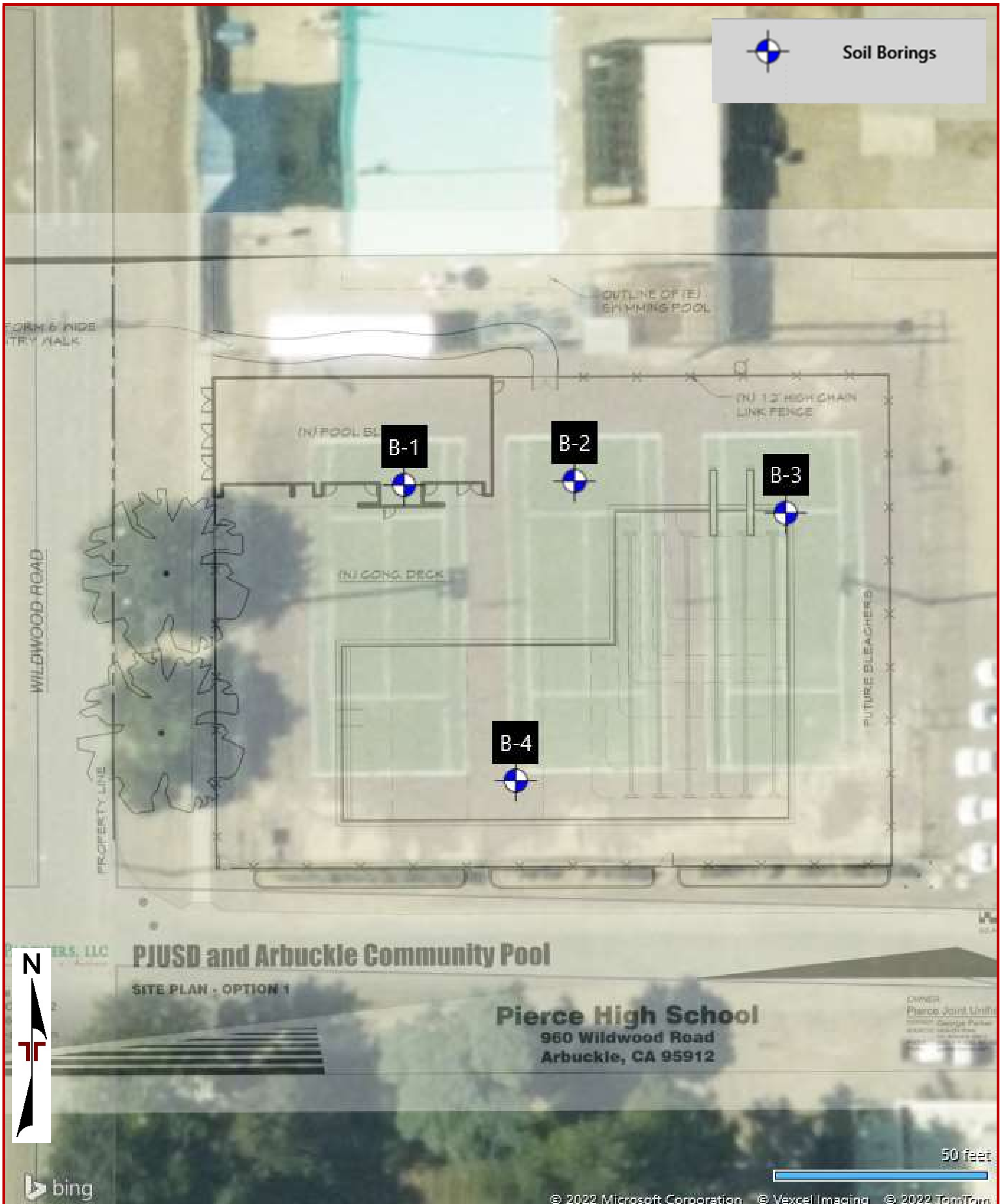


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan with Cross Section Locations

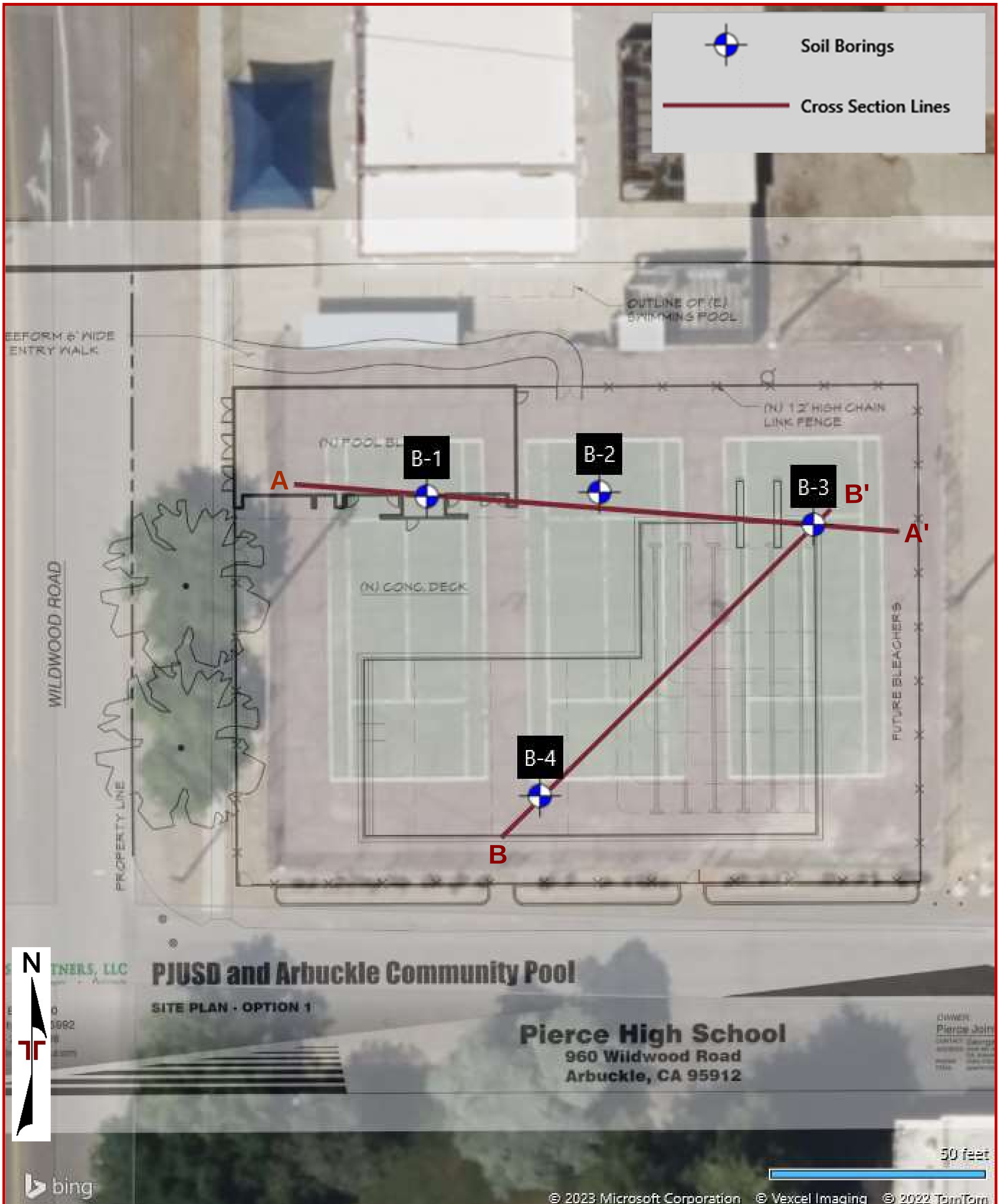


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Supplemental Maps

Contents:

Note: All attachments are one page unless noted above.

Site Location

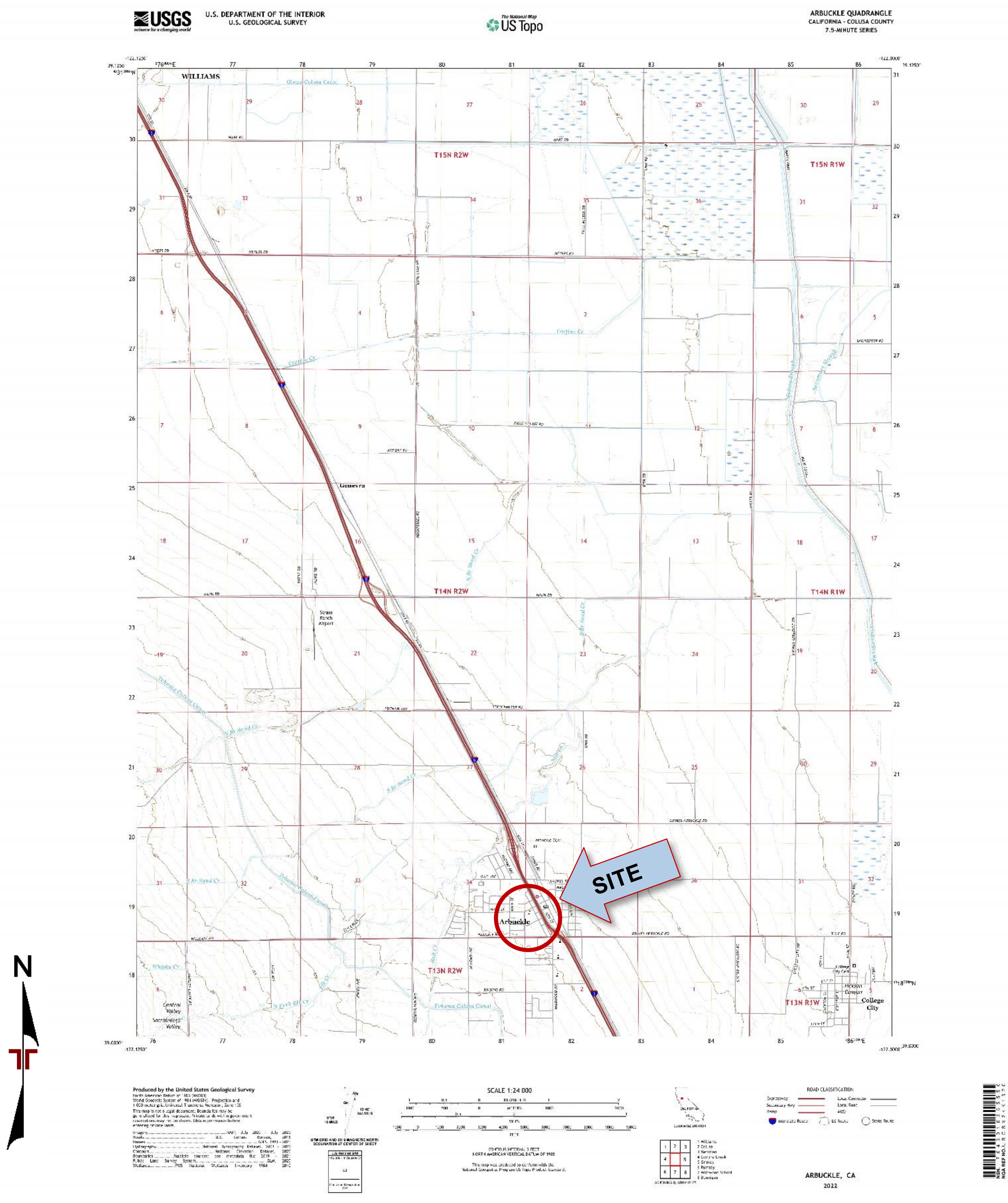
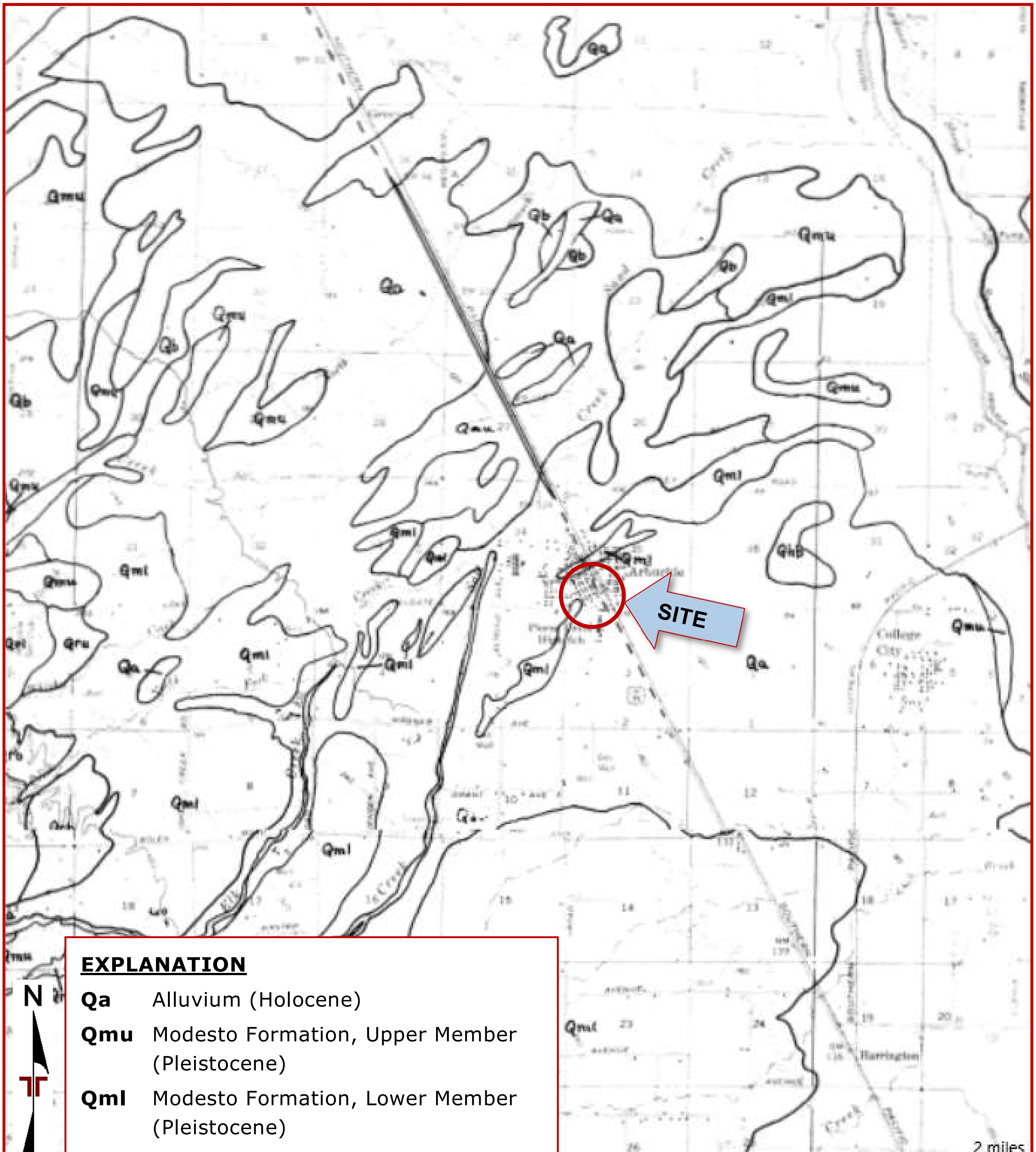


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Geologic Map



EXPLANATION

Qa Alluvium (Holocene)

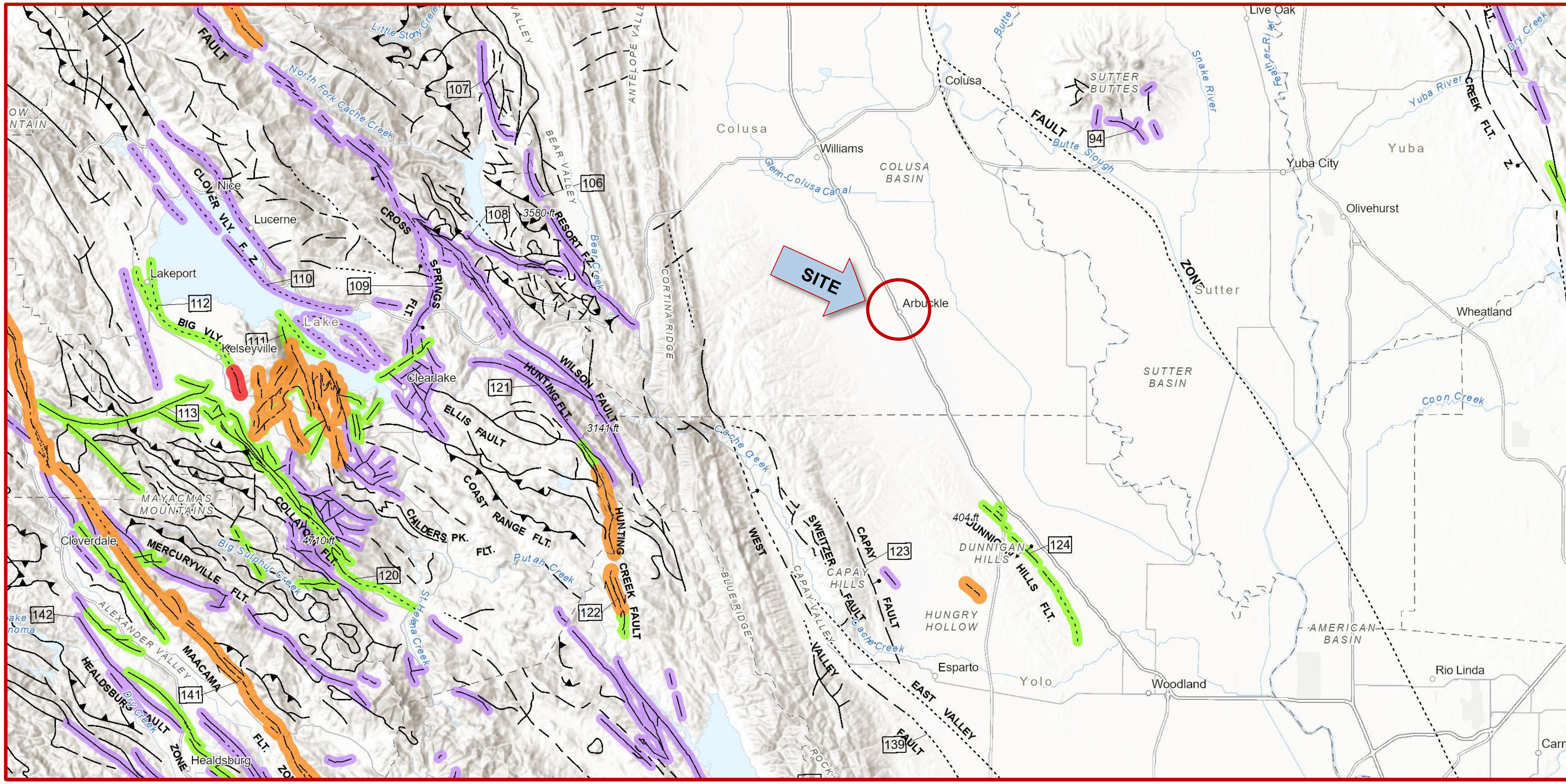
Qmu Modesto Formation, Upper Member (Pleistocene)

Qml Modesto Formation, Lower Member (Pleistocene)

Portion of Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierran Foothills, California
by
Edward J. Helley and David S. Harwood
(1985, Plate 2)

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Fault Activity Map



12/22/2022

Structural Discontinuity (Local)
structural discontinuity
Pre-Quaternary Faults
<all other values>
fault, certain
fault, approx. located
fault, concealed
thrust fault, certain
thrust fault, approx. located
thrust fault, approx. located, queried
fault, certain (ball and bar)
fault, approx. located (ball and bar)
fault, concealed (ball and bar)
Quaternary Faults
fault, certain
fault, approx. located
fault, approx. located, queried
fault, inferred, queried
fault, concealed
fault, concealed, queried
thrust fault, certain
thrust fault, approx. located
thrust fault, concealed
dextral fault, certain
dextral fault, approx. located
dextral fault, concealed
sinistral fault, certain
sinistral fault, approx. located
sinistral fault, concealed
thrust fault, certain (2)
thrust fault, approx. located (2)
thrust fault, concealed (2)
fault, certain (ball and bar)
fault, approx. located (ball and bar)
fault, concealed (ball and bar)
dextral fault, certain (ball and bar)
fault, concealed, queried (ball and bar)
fault, concealed, queried (ball and bar, 2)
fault,certain (dip)
fault, approx. located (dip)
fault, concealed (dip)
reverse fault, certain
reverse fault, approx. located
reverse fault, concealed

1:500,000

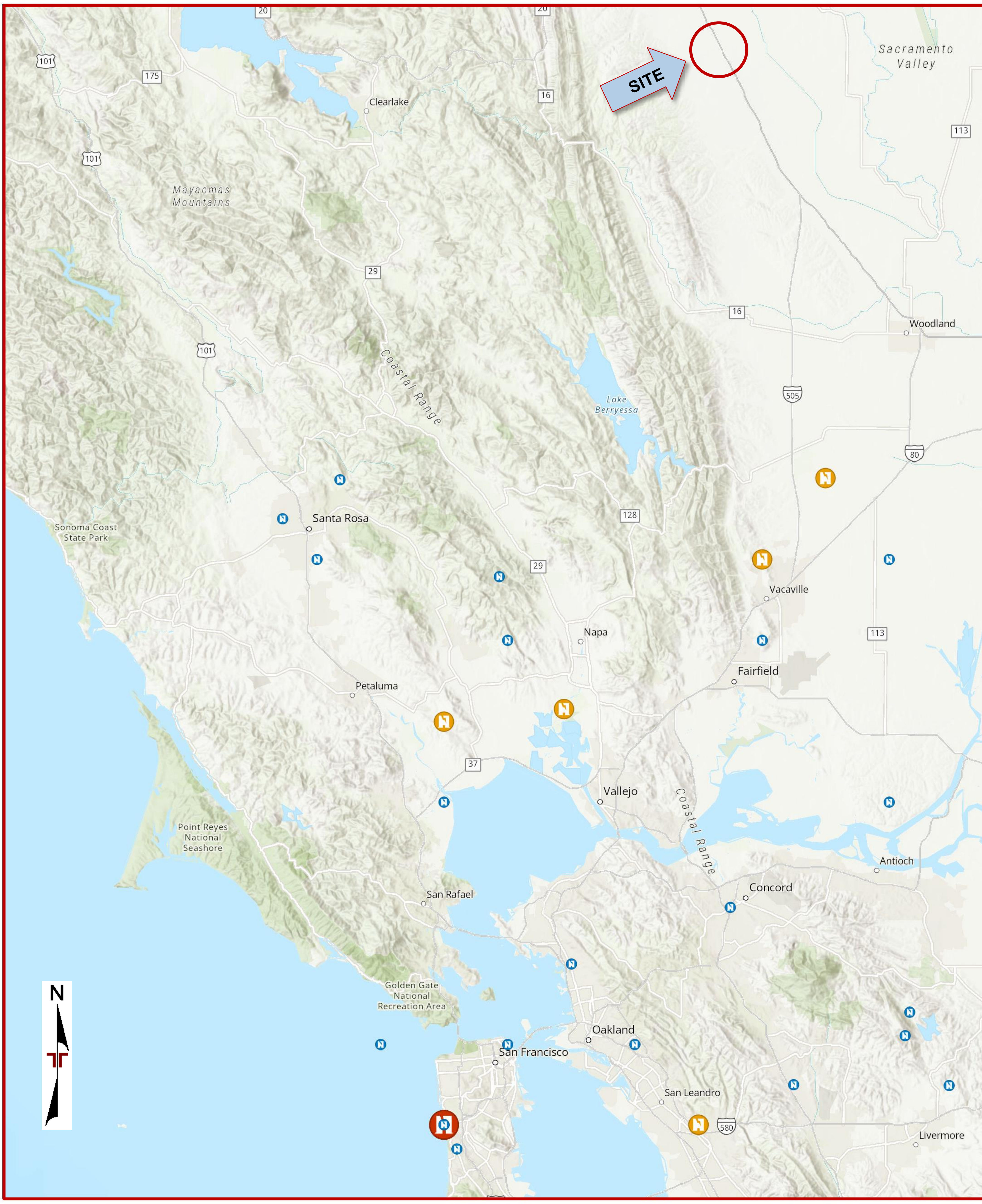
0 5 10 20 mi

0 5 10 20 km

California State Parks, Esri, HERE, Garmin, SafeGraph, FAO, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, California Geological Survey, C.W. Jennings, W.A. Bryant, Esri, CGIAR, USGS

N

Historical Earthquake Map



12/22/2022

CGS - Historic Earthquakes Epicenters - Epicenters

Magnitude 5 – 6

Magnitude 6 – 7

Magnitude >7

World Hillshade

1:625,000

0 5 10 20 mi

0 5 10 20 km

Esri, CGIAR, USGS, County of Napa, Yolo County, California State Parks, Esri, HERE, Garmin, SafeGraph, FAO, METI/NASA, USGS, Bureau of Land Management, EPA, NPS

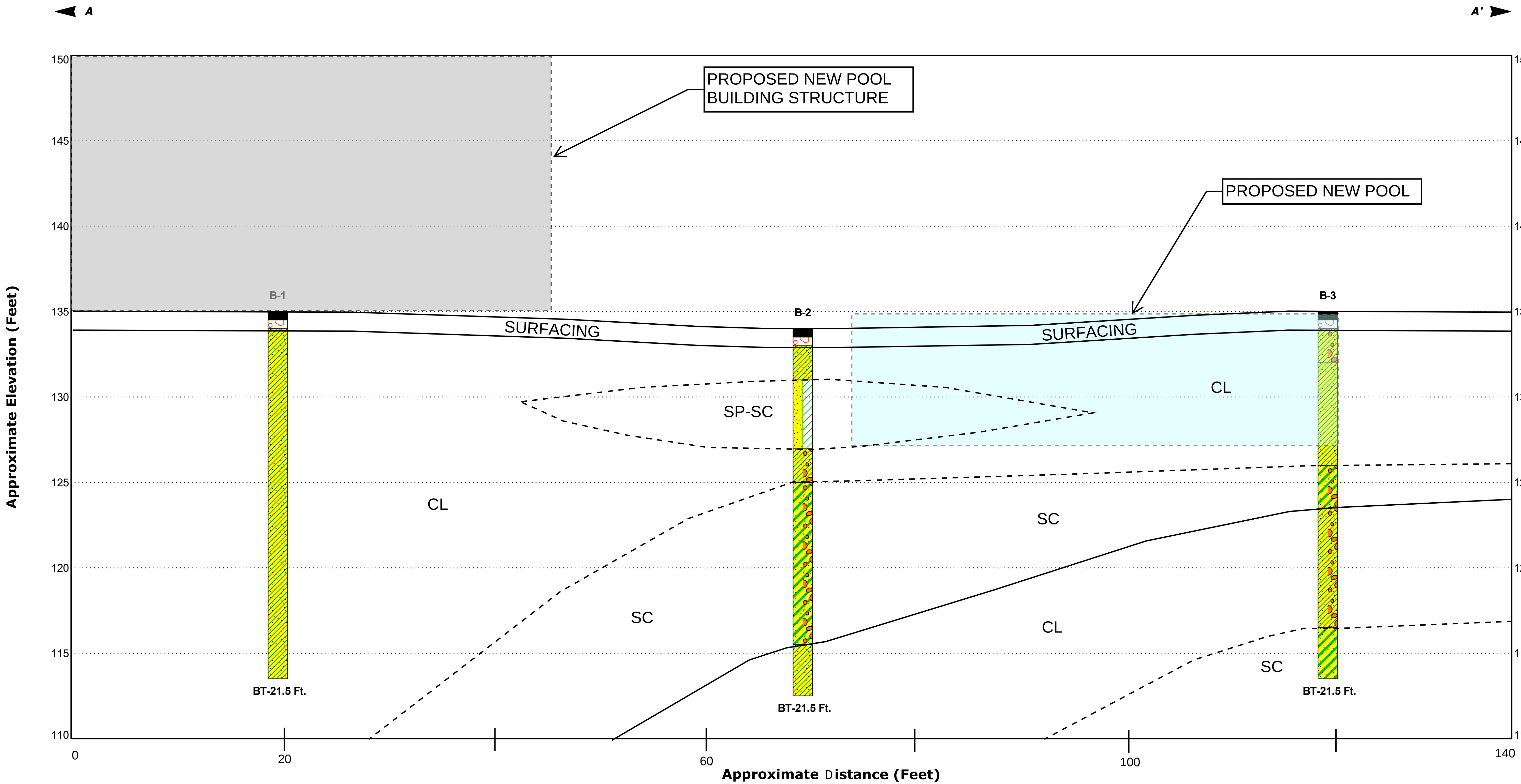
DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Pierce HS Pool Complex
960 Wildwood Road | Arbuckle, CA
Terracon Project No. NB225033

Subsurface Profile

Section A-A'

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA



Notes	Water Level Observations	Explanation
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information for symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	∇ Water Level Reading at time of drilling. Water Level Reading after drilling.	B-1 — Borehole Number Moisture Content — %w LL PL — Liquid and Plastic Limits Borehole Lithology Sampling (See General Notes) AR BT — Borehole Termination Type

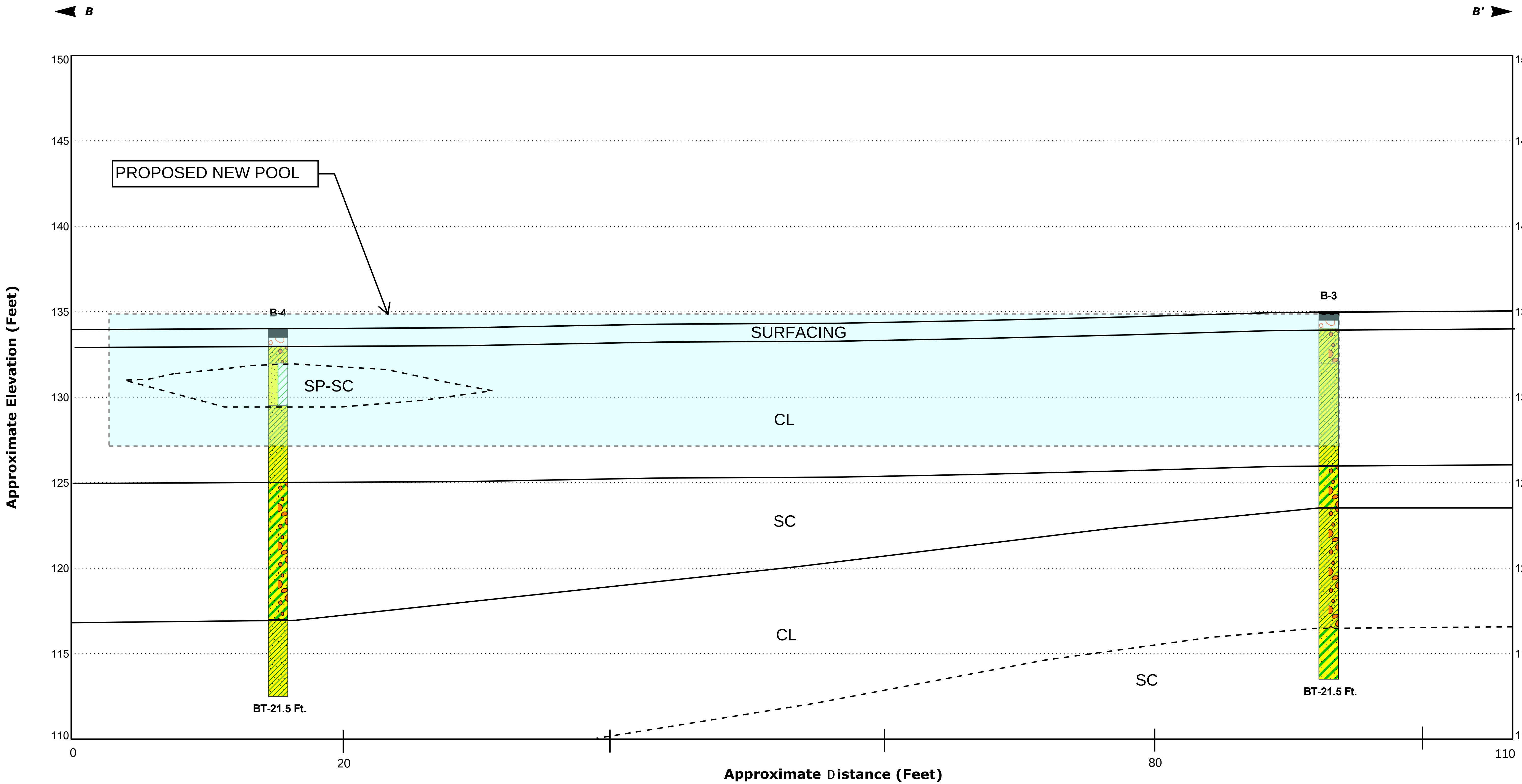
Material Legend

- Asphalt
- Aggregate Base Course
- Sandy Lean Clay
- Poorly-graded Sand with Clay
- Sandy Lean Clay with Gravel
- Clayey Sand with Gravel
- Clayey Sand
- - - - Uncertainty regarding location of geologic contact
- —— Confidence regarding location of geologic contact

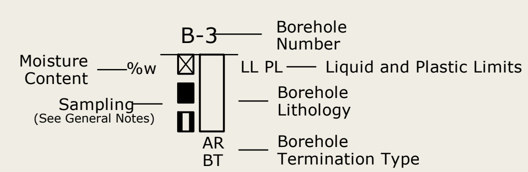
Subsurface Profile

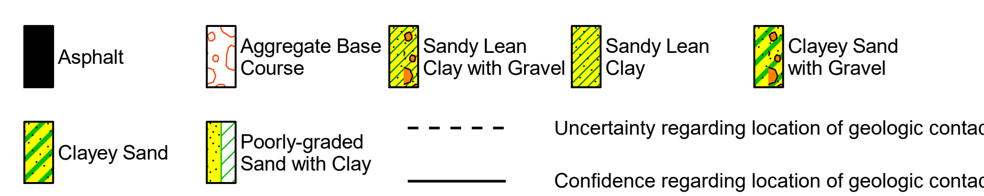
Section B-B'

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA



Notes	Water Level Observations	Explanation	Material Legend
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information for symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.		





Exploration and Laboratory Results

Contents:

Note: All attachments are one page unless noted above.

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA

Boring Log No. B-1

Location: See Exploration Plan

Latitude: 39.0120° Longitude: -122.0558°

Elevation: 135 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)
1	0.5	**ASPHALT**, approximately 6 inches in thickness	134.5
1	1.0	**AGGREGATE BASE COURSE**, approximately 6 inches in thickness	134
2	21.5	**SANDY LEAN CLAY (CL)**, trace gravel up to 1.5 inches in dimension, subrounded, dark brown to light brown, very stiff	113.5

Boring Terminated at 21.5 Feet

Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
0–1		Grab							
1–2.5		SPT	8-10-10			11.3	115		
2.5–4		SPT	12-14-17	4.5 (HP)		9.7	120		
5–6.5		SPT	11-13-20	4.5 (HP)		10.3	116		
7.5–9		SPT	6-9-13	3.0 (HP)		11.0	121		
10–11.5		SPT	8-12-21	4.5 (HP)		15.7	107		
15–16.5		SPT	7-15-21	4.0 (HP)		18.4	108		
20–21.5		SPT	8-15-21	3.0 (HP)		17.2	109		

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes

Elevation Reference: Elevations estimated from Google Earth Pro

Water Level Observations

Groundwater not encountered

Advancement Method
4" Solid Stem Auger

Abandonment Method
Boring backfilled with neat cement grout
Surface capped with asphalt

Drill Rig
CME 75

Hammer Type
Automatic

Driller
H1 Drilling Co.

Logged by
Nick Jamison

Boring Started
11-22-2022

Boring Completed
11-22-2022

50 Golden Land Ct Ste 100
Sacramento, CA

Boring Log No. B-2

Location: See Exploration Plan

Latitude: 39.0120° Longitude: -122.0556°

Elevation: 134 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)
1	0.5	**ASPHALT**, approximately 6 inches in thickness	133.5
1	1.0	**AGGREGATE BASE COURSE**, approximately 6 inches in thickness	133
2	3.0	**SANDY LEAN CLAY (CL)**, trace gravel up to 1 inch in dimension, subrounded, brown, stiff	131
3	7.0	**POORLY GRADED SAND WITH CLAY (SP-SC)**, trace gravel up to 1 inch in dimension, fine to coarse grained, subrounded, brown, medium dense	127
2	9.0	**SANDY LEAN CLAY WITH GRAVEL (CL)**, brown, stiff, gravel up to 1 inch in dimension, subrounded	125
4	18.5	**CLAYEY SAND WITH GRAVEL (SC)**, fine to coarse grained, brown, dense, gravel up to 2 inches in dimension, subrounded	115.5
2	21.5	**SANDY LEAN CLAY (CL)**, trace gravel up to 1 inch in dimension, subrounded, brown, very stiff	112.5

Boring Terminated at 21.5 Feet

Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
4-5-8		0.98	14.8	109	29-15-14	51
10-10-9			9.4	110		
11-10-11			4.5	112		7
6-8-13		1.67	12.7	123		
16-27-27			6.8	115		
14-23-22			6.0	117		
17-16-20	3.5 (HP)		14.9	113		

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
Groundwater not encountered

Notes
Elevation Reference: Elevations estimated from Google Earth Pro

Advancement Method
4" Solid Stem Auger

Abandonment Method
Boring backfilled with neat cement grout
Surface capped with asphalt

Drill Rig
CME 75

Hammer Type
Automatic

Driller
H1 Drilling Co.

Logged by
Nick Jamison

Boring Started
11-22-2022

Boring Completed
11-22-2022

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA

Boring Log No. B-3

Location: See Exploration Plan

Latitude: 39.0119° Longitude: -122.0555°

Depth (Ft.) Elevation: 135 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
1	0.5	**ASPHALT**, approximately 6 inches in thickness	134.5										
1	1.0	**AGGREGATE BASE COURSE**, approximately 6 inches in thickness	134										
2	3.0	**SANDY LEAN CLAY WITH GRAVEL (CL)**, brown, stiff, gravel up to 1 inch in dimension, subrounded	132	1–2.5			5-7-8			14.4	111		
				2.5–4			8-7-7			12.9	114		
2	9.0	**SANDY LEAN CLAY (CL)**, trace gravel up to 1 inch in dimension, subangular, brown, stiff	126	5–6.5			8-10-8	2.25 (HP)		6.3	113		
				7.5–9			4-6-10	2.0 (HP)		15.6	112	28-16-12	67
4	11.5	**CLAYEY SAND WITH GRAVEL (SC)**, fine to coarse grained, brown, dense, gravel up to 1 inch in dimension, subrounded	123.5	10–11.5			23-27-25			6.0	139		
2	18.5	**SANDY LEAN CLAY WITH GRAVEL (CL)**, brown, hard, gravel up to 2 inches in dimension, subrounded	116.5	15–16.5			15-29-17	4.5 (HP)		5.6	120		
4	21.5	**CLAYEY SAND (SC)**, fine grained, brown, medium dense	113.5	20–21.5			7-13-22			17.7	109		

Boring Terminated at 21.5 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes

Elevation Reference: Elevations estimated from Google Earth Pro

Water Level Observations

Groundwater not encountered

Advancement Method
4" Solid Stem Auger

Abandonment Method
Boring backfilled with neat cement grout
Surface capped with asphalt

Drill Rig
CME 75

Hammer Type
Automatic

Driller
H1 Drilling Co.

Logged by
Nick Jamison

Boring Started
11-22-2022

Boring Completed
11-22-2022

Pierce HS Pool Complex
960 Wildwood Road | Arbuckle, CA
Terracon Project No. NB225033

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA

Boring Log No. B-4

Location: See Exploration Plan

Latitude: 39.0118° Longitude: -122.0557°

Model Layer	Depth (Ft.)	Description	Elevation: 134 (Ft.) +/-
1	0.5	**ASPHALT**, approximately 6 inches in thickness	133.5
1	1.0		133
2	2.0	**AGGREGATE BASE COURSE**, approximately 6 inches in thickness	132
3		**SANDY LEAN CLAY WITH GRAVEL (CL)**, brown to dark brown, stiff, gravel up to 1 inch in dimension, subrounded	
3	4.5	**POORLY GRADED SAND WITH CLAY (SP-SC)**, trace gravel up to 1.5 inch in dimension, fine to coarse grained, subrounded, brown, medium dense	129.5
2	9.0	**SANDY LEAN CLAY (CL)**, trace gravel up to 1 inch in dimension, subrounded, brown, stiff	125
4	17.0	**CLAYEY SAND WITH GRAVEL (SC)**, fine to coarse grained, light brown, medium dense, gravel up to 1 inch in dimension, subrounded	117
2	21.5	**SANDY LEAN CLAY (CL)**, brown, very stiff	112.5
		Boring Terminated at 21.5 Feet	

Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
5-6-8			13.7	113		
6-7-7			14.2	110		
11-8-5	1.5 (HP)		3.6	115		
4-7-13		0.88	17.9	111	34-16-18	
7-10-13			15.2	115		47
7-16-26			7.0	120		
8-13-19	3.75 (HP)		14.8	108		

Depth (Ft.): 5, 10, 15, 20

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes
Elevation Reference: Elevations estimated from Google Earth Pro

Water Level Observations
Groundwater not encountered

Advancement Method
4" Solid Stem Auger

Abandonment Method
Boring backfilled with neat cement grout
Surface capped with asphalt

Drill Rig
CME 75

Hammer Type
Automatic

Driller
H1 Drilling Co.

Logged by
Nick Jamison

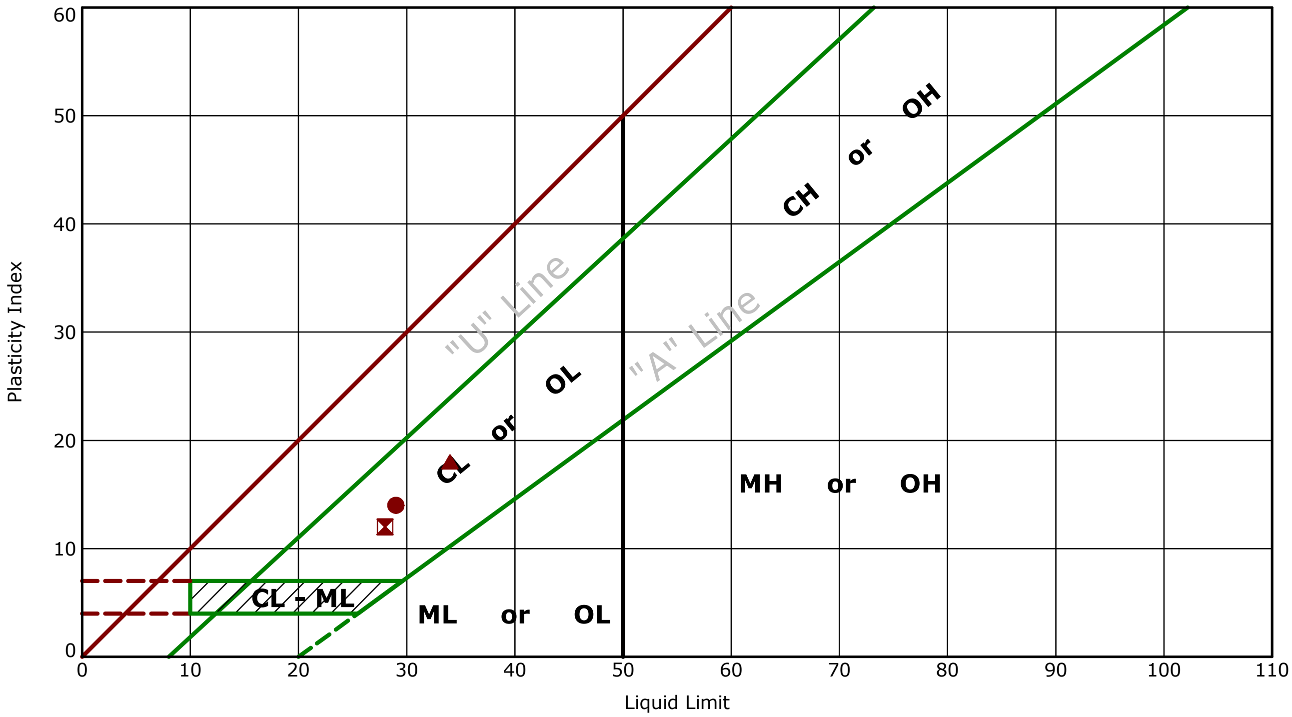
Boring Started
11-22-2022

Boring Completed
11-22-2022

50 Golden Land Ct Ste 100
Sacramento, CA

Atterberg Limit Results

ASTM D4318

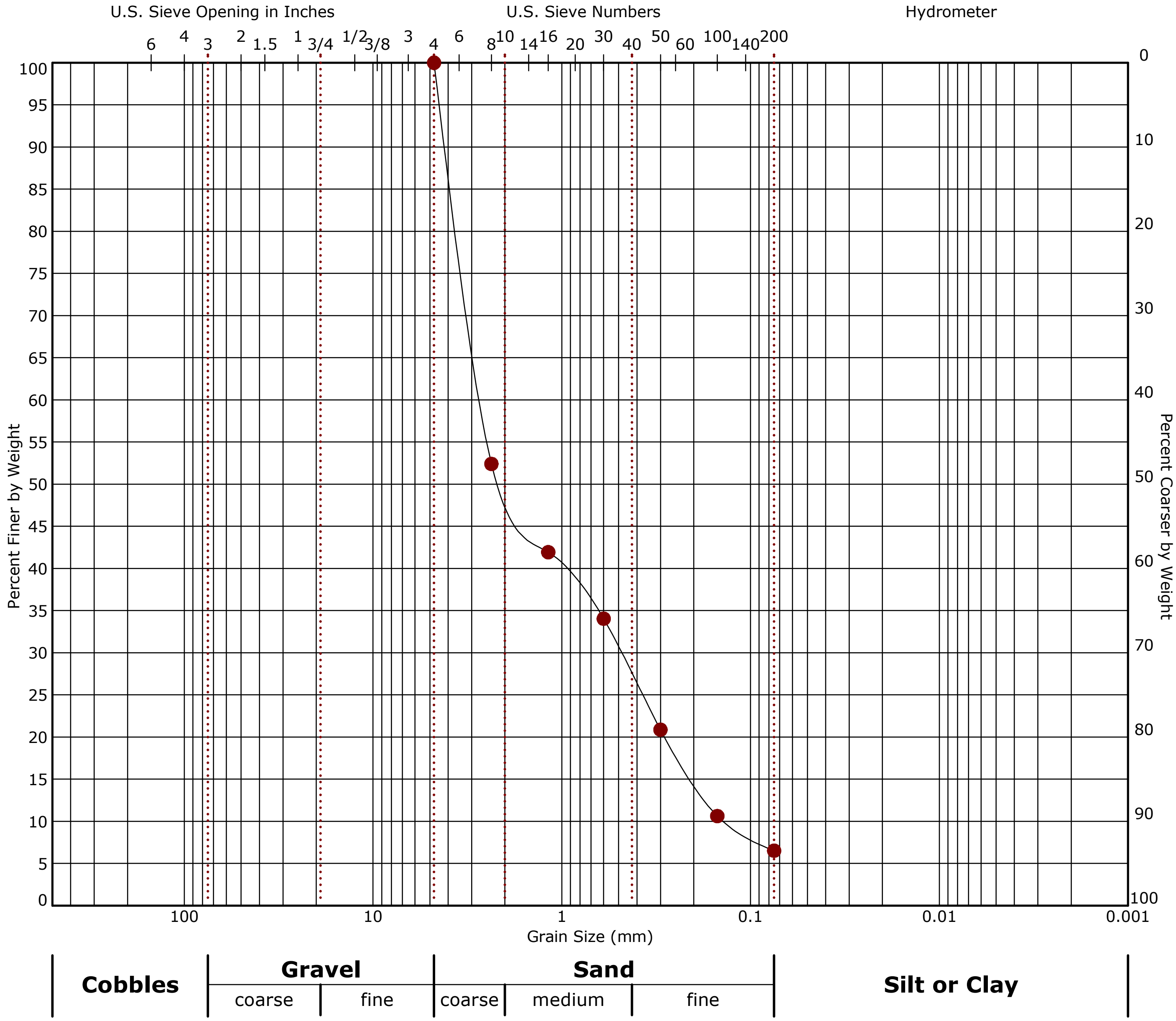


	Boring ID	Depth (Ft)	LL	PL	PI	Fines	USCS	Description
●	B-2	1 - 2.5	29	15	14	51.2	CL	SANDY LEAN CLAY
⊠	B-3	7.5 - 9	28	16	12	67.2	CL	SANDY LEAN CLAY
▲	B-4	7.5 - 9	34	16	18		CL	SANDY LEAN CLAY

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA

Grain Size Distribution

ASTM D422 / ASTM C136



	Boring ID	Depth (Ft)	Description	USCS	LL	PL	PI	Cc	Cu
●	B-2	5 - 6.5	POORLY GRADED SAND with CLAY	SP-SC				0.66	19.56

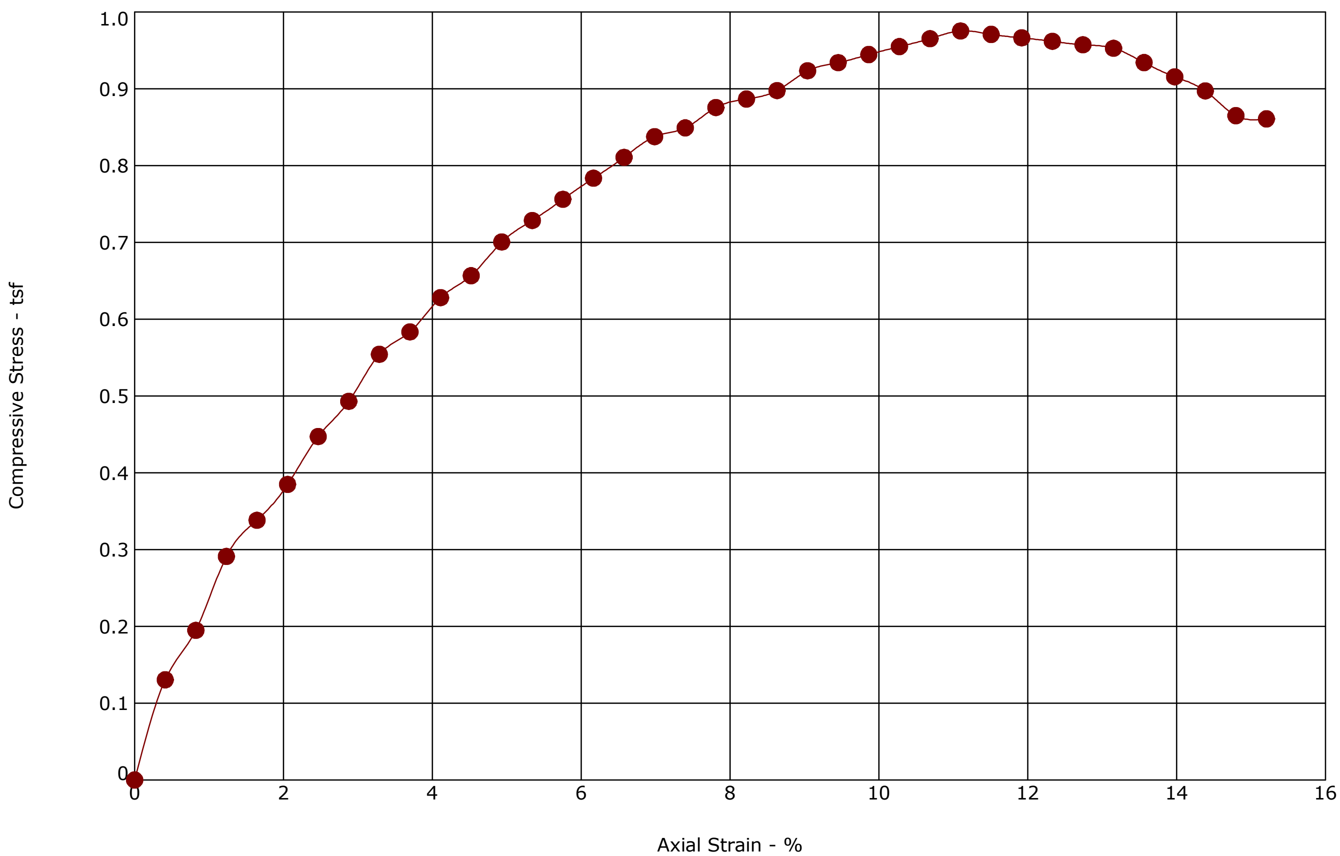
	Boring ID	Depth (Ft)	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay
●	B-2	5 - 6.5	4.75	2.639	0.485	0.135	0.0	0.0	93.5	6.5		

Pierce HS Pool Complex
960 Wildwood Road | Arbuckle, CA
Terracon Project No. NB225033

50 Golden Land Ct Ste 100
Sacramento, CA

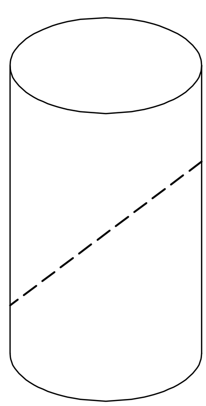
Unconfined Compression Test

ASTM D2166



Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B-2	1 - 2.5	DMRS	29	15	14	51.2	SANDY LEAN CLAY

Specimen Failure Mode



Failure Mode: Shear (dashed)

Specimen Test Data

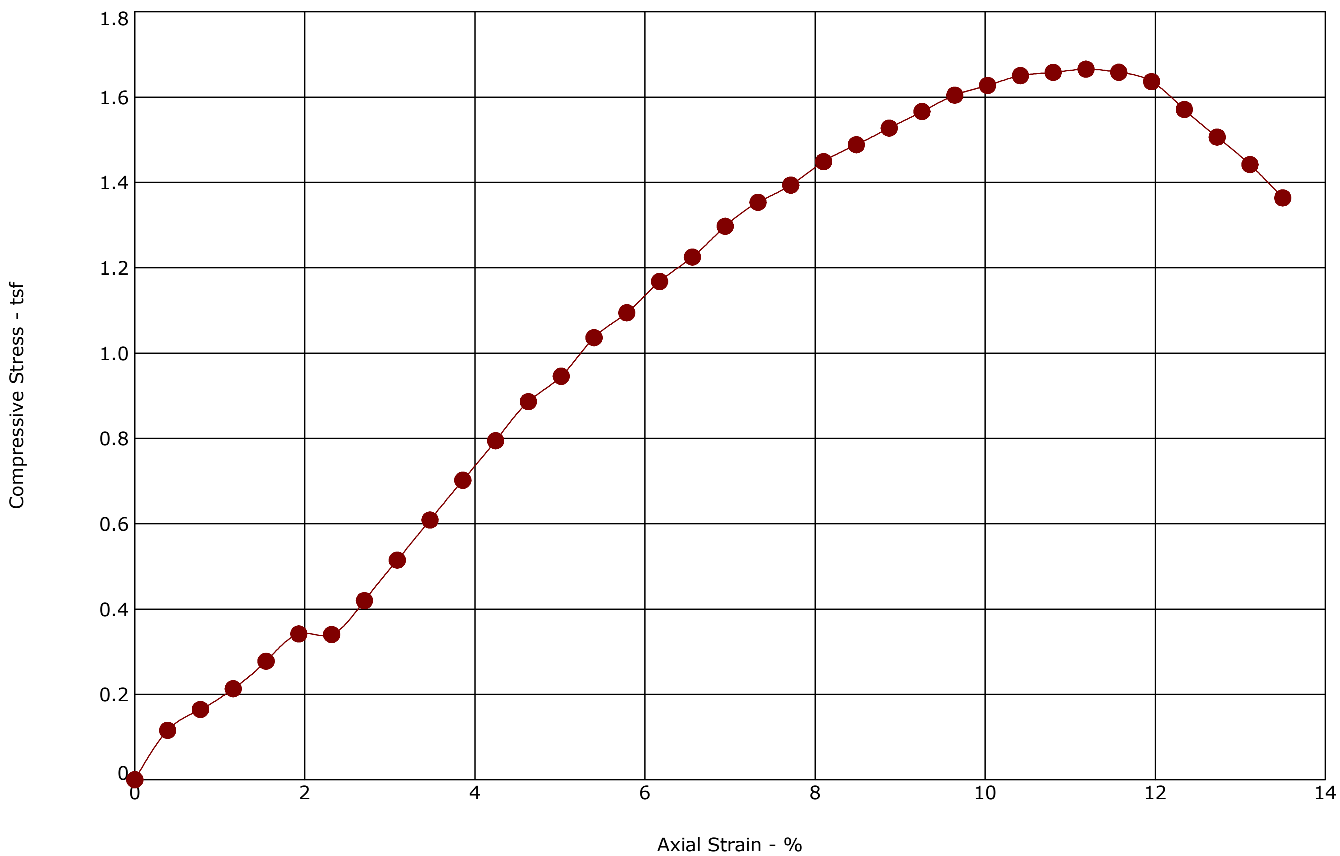
Moisture Content (%):	14.8
Dry Density (pcf):	109
Diameter (in.):	2.37
Height (in.):	4.87
Height / Diameter Ratio:	2.06
Calculated Saturation (%):	72.47
Calculated Void Ratio:	0.55
Assumed Specific Gravity:	2.7
Failure Strain (%):	11.10
Unconfined Compressive Strength (tsf):	0.98
Undrained Shear Strength (tsf):	0.49
Strain Rate (in/min):	
Remarks:	

Laboratory tests are not valid if separated from original report.

50 Golden Land Ct Ste 100
Sacramento, CA

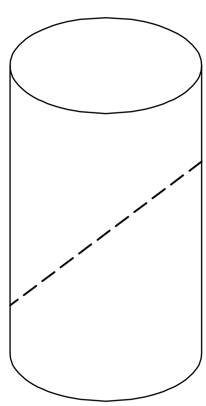
Unconfined Compression Test

ASTM D2166



Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B-2	7.5 - 9	DMRS					SANDY LEAN CLAY with GRAVEL

Specimen Failure Mode



Failure Mode: Shear (dashed)

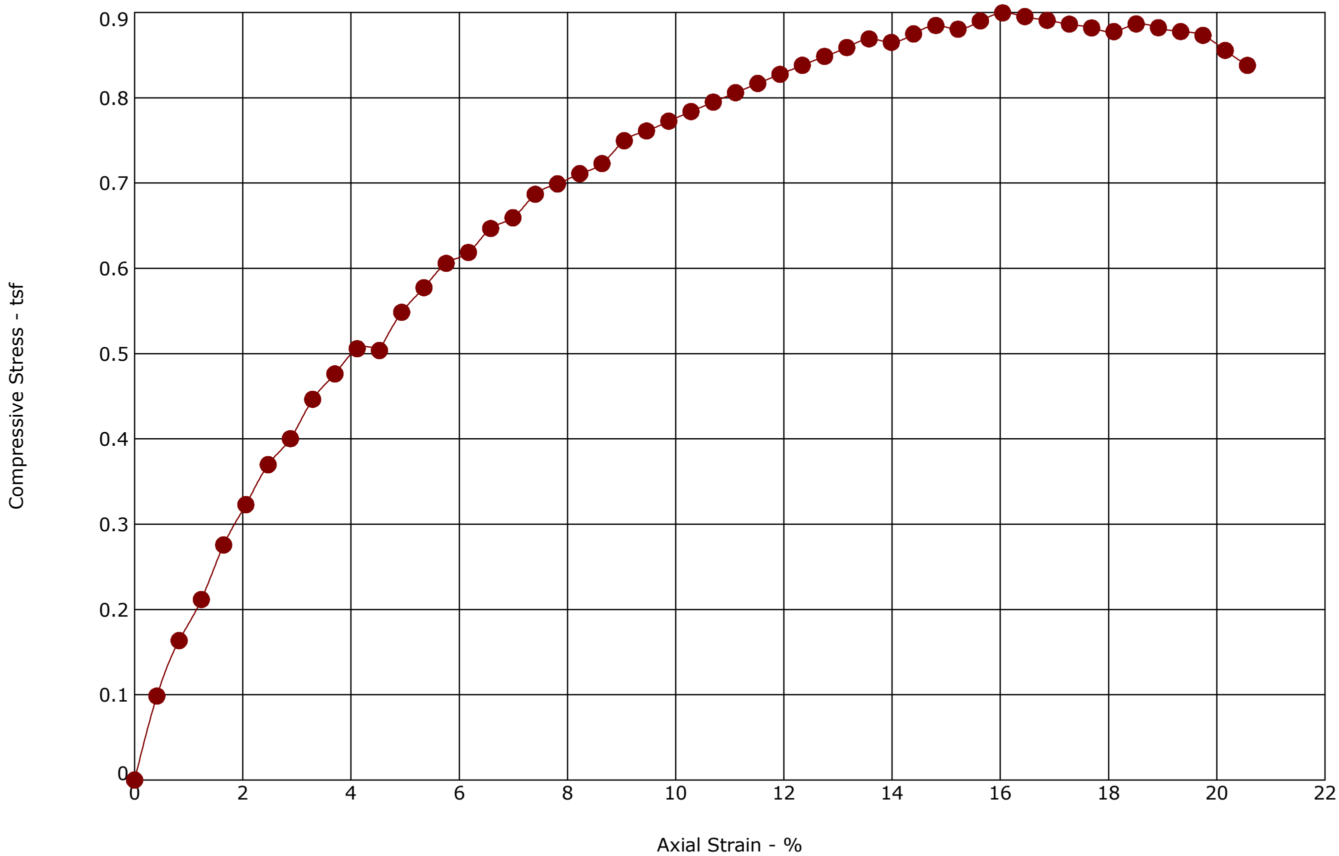
Specimen Test Data

Moisture Content (%):	12.7
Dry Density (pcf):	123
Diameter (in.):	2.35
Height (in.):	5.19
Height / Diameter Ratio:	2.21
Calculated Saturation (%):	92.73
Calculated Void Ratio:	0.37
Assumed Specific Gravity:	2.7
Failure Strain (%):	11.19
Unconfined Compressive Strength (tsf):	1.67
Undrained Shear Strength (tsf):	0.83
Strain Rate (in/min):	
Remarks:	

50 Golden Land Ct Ste 100
Sacramento, CA

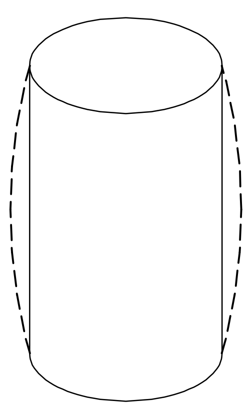
Unconfined Compression Test

ASTM D2166



Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B-4	7.5 - 9	DMRS	34	16	18		SANDY LEAN CLAY

Specimen Failure Mode



Failure Mode: Bulge (dashed)

Specimen Test Data

Moisture Content (%):	17.9
Dry Density (pcf):	111
Diameter (in.):	2.36
Height (in.):	4.86
Height / Diameter Ratio:	2.06
Calculated Saturation (%):	92.23
Calculated Void Ratio:	0.52
Assumed Specific Gravity:	2.7
Failure Strain (%):	14.81
Unconfined Compressive Strength (tsf):	0.88
Undrained Shear Strength (tsf):	0.44
Strain Rate (in/min):	
Remarks:	

CHEMICAL LABORATORY TEST REPORT

Project Number: NB225033
Service Date: 12/02/22
Report Date: 12/07/22

Terracon
10400 State Highway 191
Midland, Texas 79707
432-684-9600

Client

Pierce Joint Unified School District
540A 6th Street
Arbuckle, CA 95912

Project

Pierce HS Pool Complex
966 Wildwood Road
Arbuckle, CA

Sample Location	B-1
Sample Depth (ft.)	1-4
pH Analysis, ASTM - G51-18	6.7
Water Soluble Sulfate (SO4), ASTM C 1580 (%)	0.01
Sulfides, ASTM - D4658-15, (mg/kg)	nil
Chlorides, ASTM D 512 , (%)	0.05
RedOx, ASTM D-1498, (mV)	+540
Total Salts, ASTM D1125-14, (mg/kg)	1,885
Resistivity, ASTM G187, (ohm-cm)	3,098

Analyzed By: Zach Robertson

Zach Robertson
Engineering Technician III

Supporting Information

Contents:

Note: All attachments are one page unless noted above.

Terracon
50 Golden Land Ct Ste 100
Sacramento, CA

General Notes

Sampling	Water Level		Field Tests	
Modified Dames & Moore Ring Sampler		Water Initially Encountered	N	Standard Penetration Test Resistance (Blows/Ft.)
Grab Sample		Water Level After a Specified Period of Time	(HP)	Hand Penetrometer
		Water Level After a Specified Period of Time	(T)	Torvane
		Cave In Encountered	(DCP)	Dynamic Cone Penetrometer
			UC	Unconfined Compressive Strength
			(PID)	Photo-Ionization Detector
			(OVA)	Organic Vapor Analyzer

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

Descriptive Soil Classicification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance			**Consistency of Fine-Grained Soils** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Relative Density	**Standard Penetration or N-Value (Blows/Ft.)**	**Ring Sampler (Blows/Ft.)**	**Consistency**	**Unconfined Compressive Strength Qu (tsf)**	**Standard Penetration or N-Value (Blows/Ft.)**	**Ring Sampler (Blows/Ft.)**
Very Loose	0 - 3	0 - 6	Very Soft	less than 0.25	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	0.25 to 0.50	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium Stiff	0.50 to 1.00	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	1.00 to 2.00	8 - 15	10 - 18
Very Dense	> 50	> 99	Very Stiff	2.00 to 4.00	15 - 30	19 - 42
			Hard	> 4.00	> 30	> 42

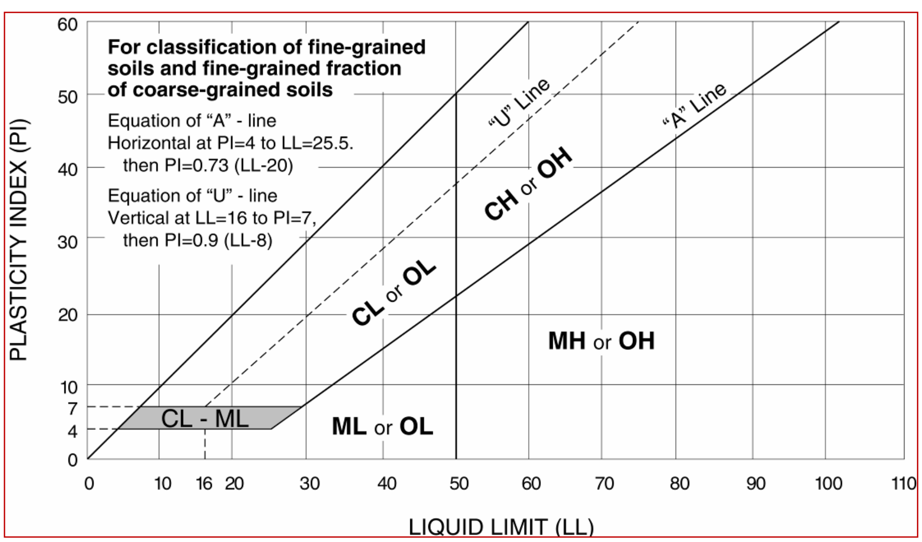
Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

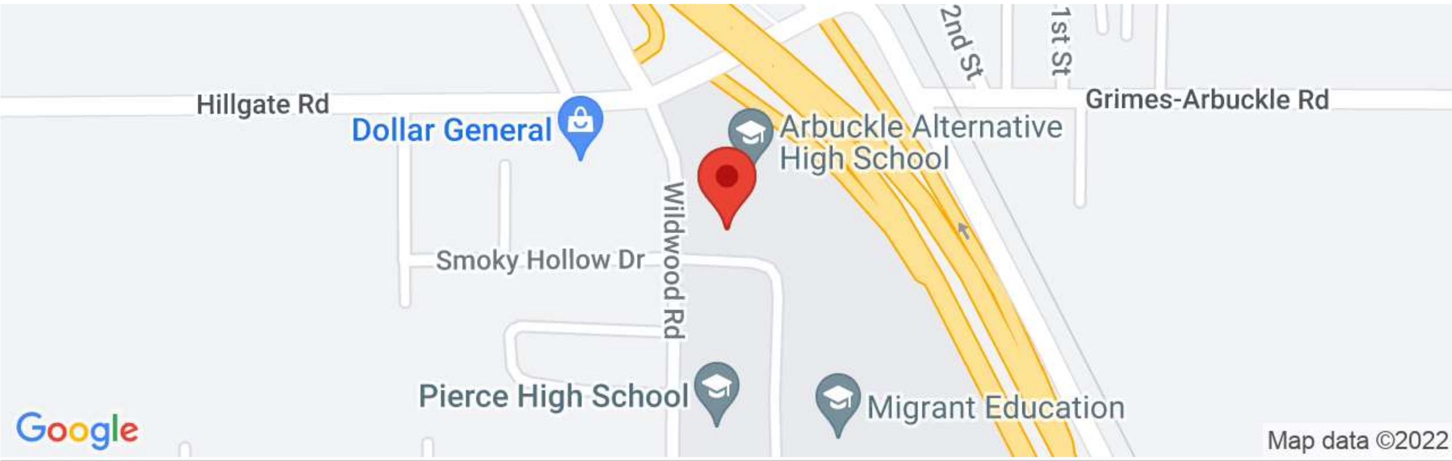
[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



OSHPD

Pierce HS Pool Complex

Latitude, Longitude: 39.0119, -122.0556



Date	12/30/2022, 12:17:16 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.937	MCE_R ground motion. (for 0.2 second period)
S_1	0.364	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.054	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.703	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.125	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.403	MCE_G peak ground acceleration
F_{PGA}	1.197	Site amplification factor at PGA
PGA_M	0.482	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.937	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.033	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.364	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.4	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.403	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.907	Mapped value of the risk coefficient at short periods
C_{R1}	0.911	Mapped value of the risk coefficient at a period of 1 s
C_V	1.268	Vertical coefficient

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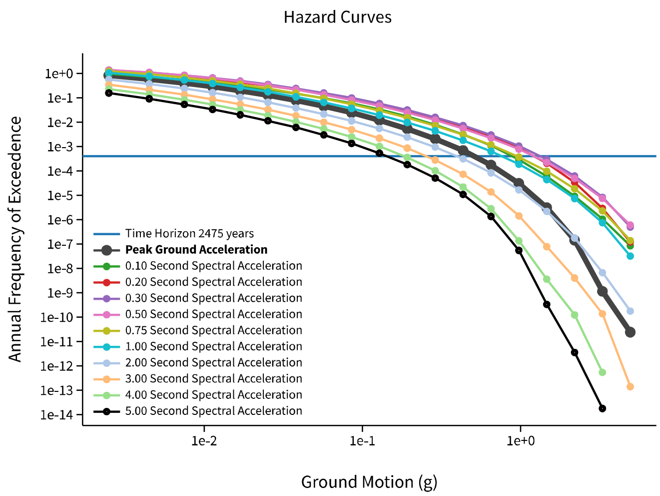
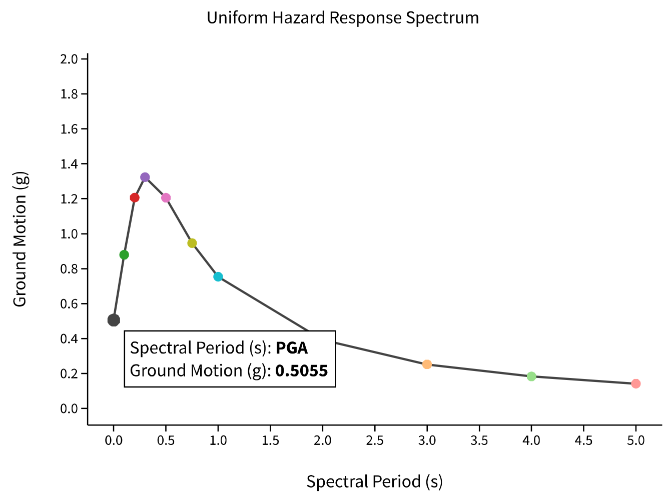
U.S. Geological Survey - Earthquake Hazards Program

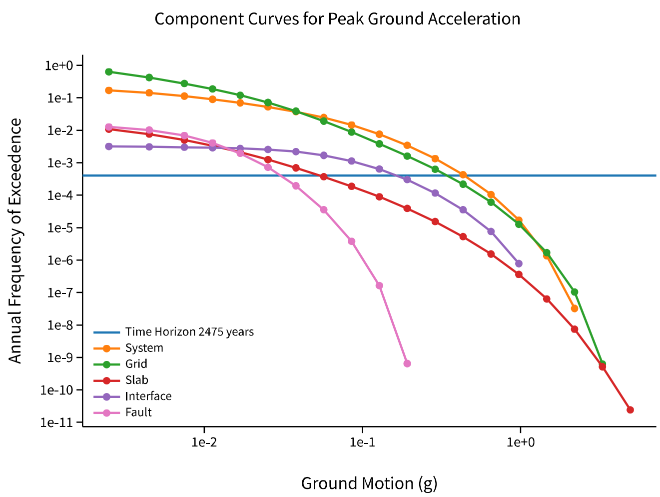
Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Hazard Curve





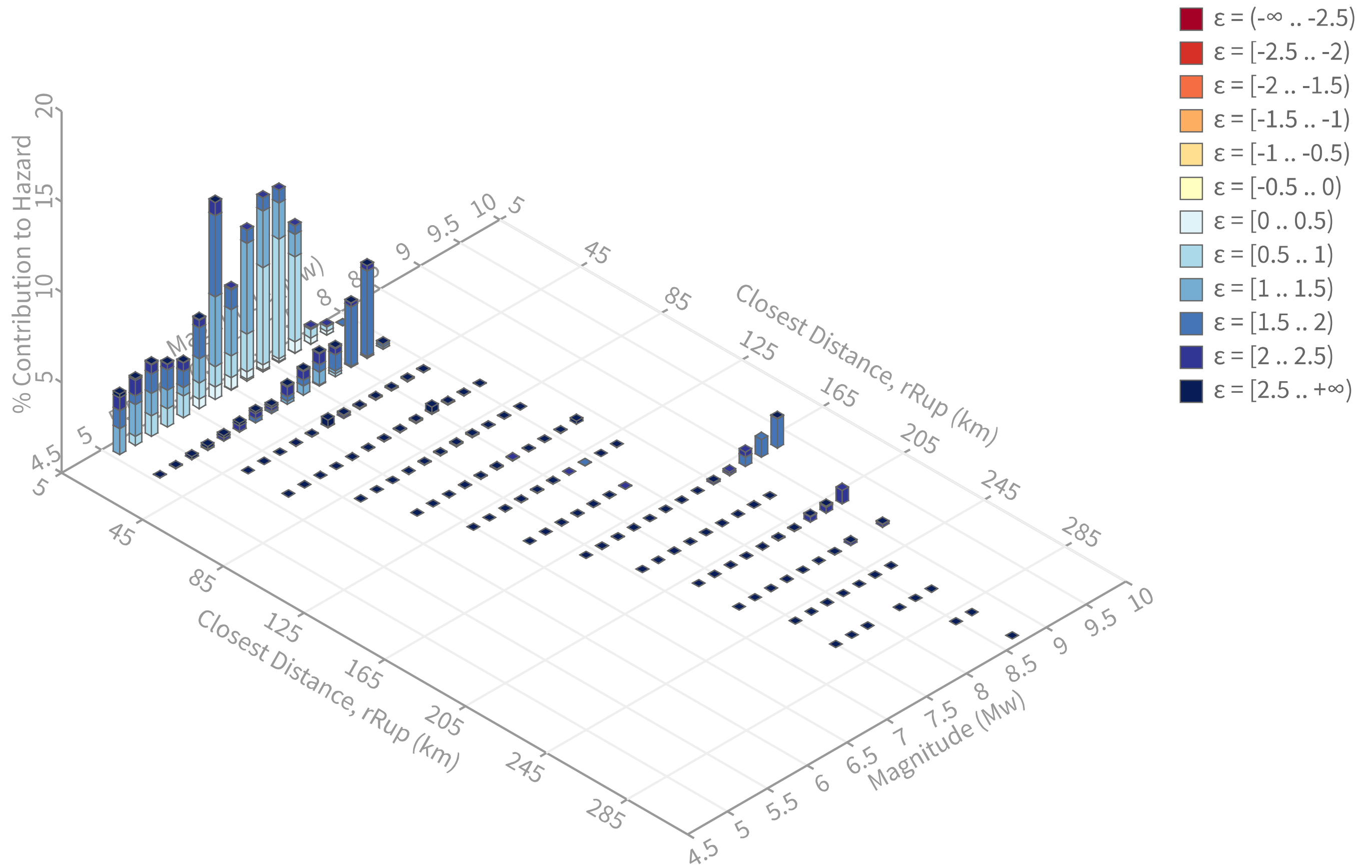


View Raw Data

Deaggregation

Component

Total



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ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)
% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.50547365 g

Recovered targets

Return period: 2717.2846 yrs
Exceedance rate: 0.00036801446 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.4 %

Mean (over all sources)

m: 6.75
r: 28.5 km
ε_0: 1.35 σ

Mode (largest m-r bin)

m: 6.3
r: 15.65 km
ε_0: 1.39 σ
Contribution: 10.89 %

Mode (largest m-r-ε_0 bin)

m: 7.11
r: 17.21 km
ε_0: 0.76 σ
Contribution: 6.62 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							30.29
Great Valley 03 Mysterious Ridge [2]		17.39	6.97	1.00	122.210°W	38.960°N	246.59	14.93
Hunting Creek - Bartlett Springs connector 2011 [4]		33.54	7.51	1.81	122.405°W	38.881°N	244.29	5.64
Great Valley 03 Mysterious Ridge [1]		18.18	6.31	1.59	122.235°W	39.005°N	267.42	3.39
Great Valley 03a Dunnigan Hills [6]		19.13	6.19	1.92	122.025°W	38.835°N	172.41	1.45
Great Valley 03 Mysterious Ridge [3]		17.42	7.02	0.97	122.206°W	38.952°N	242.98	1.33
Great Valley 03 Mysterious Ridge [4]		18.95	6.98	1.09	122.177°W	38.899°N	219.97	1.03
UC33brAvg_FM32	System							29.82
Great Valley 03 Mysterious Ridge [2]		17.39	6.98	1.00	122.210°W	38.960°N	246.59	14.57
Hunting Creek - Bartlett Springs connector 2011 [4]		33.54	7.50	1.81	122.405°W	38.881°N	244.29	5.48
Great Valley 03 Mysterious Ridge [1]		18.18	6.30	1.59	122.235°W	39.005°N	267.42	3.32
Great Valley 03a Dunnigan Hills [6]		19.13	6.19	1.92	122.025°W	38.835°N	172.41	1.44
Great Valley 03 Mysterious Ridge [3]		17.42	7.06	0.94	122.206°W	38.952°N	242.98	1.39
Great Valley 03 Mysterious Ridge [4]		18.95	7.04	1.06	122.177°W	38.899°N	219.97	1.07
UC33brAvg_FM31 (opt)	Grid							16.70
PointSourceFinite: -122.056, 39.052		6.60	5.76	0.89	122.056°W	39.052°N	0.00	3.90
PointSourceFinite: -122.056, 39.052		6.60	5.76	0.89	122.056°W	39.052°N	0.00	3.69
PointSourceFinite: -122.056, 39.106		10.39	5.97	1.29	122.056°W	39.106°N	0.00	1.29
PointSourceFinite: -122.056, 39.133		12.50	6.07	1.47	122.056°W	39.133°N	0.00	1.19
PointSourceFinite: -122.056, 39.106		10.39	5.97	1.29	122.056°W	39.106°N	0.00	1.17
PointSourceFinite: -122.056, 39.133		12.50	6.07	1.47	122.056°W	39.133°N	0.00	1.15
UC33brAvg_FM32 (opt)	Grid							16.68
PointSourceFinite: -122.056, 39.052		6.60	5.76	0.89	122.056°W	39.052°N	0.00	3.90
PointSourceFinite: -122.056, 39.052		6.60	5.76	0.89	122.056°W	39.052°N	0.00	3.69
PointSourceFinite: -122.056, 39.106		10.39	5.97	1.29	122.056°W	39.106°N	0.00	1.29
PointSourceFinite: -122.056, 39.133		12.51	6.07	1.47	122.056°W	39.133°N	0.00	1.19
PointSourceFinite: -122.056, 39.106		10.39	5.97	1.29	122.056°W	39.106°N	0.00	1.16
PointSourceFinite: -122.056, 39.133		12.51	6.07	1.47	122.056°W	39.133°N	0.00	1.15
sub0_ch_bot.in	Interface							3.25
Cascadia Megathrust - whole CSZ Characteristic		175.76	9.14	1.82	122.945°W	40.376°N	333.64	3.25
sub0_ch_mid.in	Interface							1.39
Cascadia Megathrust - whole CSZ Characteristic		214.16	8.95	2.24	123.829°W	40.347°N	314.93	1.39