GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS REPORT

MARGARET G. SCOTTEN SCHOOL
MODERNIZATION 2019 – MULTIPURPOSE ROOM

10821 SQUIRREL CREEK ROAD
GRASS VALLEY, CALIFORNIA
CGS Application No. 02-CGS3809

January 14, 2019 (Updated April 5, 2019)

Prepared For:

GRASS VALLEY ELEMENTARY SCHOOL DISTRICT
Mr. Eric Fredrickson, Superintendent
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PROJECT NO. 5219.00

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Mr. Eric Fredrickson, Superintendent
Grass Valley Elementary School District
10840 Gilmore Way
Grass Valley, California 95945
CGS Application No. 02-CGS3809

Reference: **Margaret G. Scotten School**
Modernization 2019 – Multipurpose Room
10821 Squirrel Creek Road
Grass Valley, Nevada County, California

Subject: **Geotechnical Engineering and Geologic Hazards Report**

Dear Mr. Fredrickson,

NV5 performed a geotechnical engineering investigation and conducted a geologic hazards evaluation for the proposed multipurpose room modernization located at Margaret G. Scotten School. NV5's geologic hazards and geotechnical engineering investigation of the site were performed consistent with the scope of services presented in our November 5, 2018 proposal (PN18220).

The findings, conclusions and recommendations presented in this report are based on the following relevant information collected and evaluated by NV5: literature review, surface observations, subsurface exploration, laboratory test results, and experience with similar projects, sites and conditions in the area. The proposed project will provide a new stage and bleacher addition to the existing multipurpose room utilizing conventional design and construction practices. There were no geologic hazards, seismic hazards or geotechnical engineering hazards identified on the site or in the immediate area that require design mitigation. It is NV5's opinion that the site is suitable for the proposed construction provided the geotechnical engineering recommendations presented in this report are incorporated into the earthwork and structural improvements. This report should not be relied upon without review by NV5 if a period of 24 months elapses between the issuance report date shown above and the date when construction commences.

NV5 appreciates the opportunity to provide geotechnical engineering services for this important project. If you have questions or need additional information, please do not hesitate to contact the undersigned at 530-478-1305.

Sincerely,
NV5

Daniel A. Vieira, G.I.T.
Project Geologist

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ACRONYMS

AB	aggregate base
AC	asphalt concrete
ACI	American Concrete Institute
amsl	above mean sea level
ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below ground surface
CalEPA	California Environmental Protection Agency
CAT	Caterpillar
CBC	California Building Code
CGS	California Geological Survey
CIDH	cast-in-drilled-hole
CQA	Construction Quality Assurance
CUSD	Chico Unified School District
DTSC	Department of Toxic Substances Control
DWR	California Department of Water Resources
EFP	equivalent fluid pressure
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FS	factor of safety
ft/s	feet per second
GBA	Geoprofessional Business Association
H:V	horizontal to vertical ratio
IBC	International Building Code
km	kilometer
MCE	maximum considered earthquake
M_L	local magnitude earthquake
mybp	million years before present
NEIC	National Earthquake Information Center
OSHA	Occupational Safety and Hazards Administration
P-wave	seismic compression wave
PCA	Portland Cement Association
pcf	pounds per cubic foot
PGA_M	peak ground acceleration
PI	plasticity index
psf	pounds per square foot
psf/ft	per square foot, per foot of depth
psi	pounds per square inch
S-wave	shear-wave
SFHA	Special Flood Hazard Area
SPT	standard penetration test
SRMS	seismic refraction microtremor survey
SSD	saturated surface dry
USCS	Unified Soils Classification System
USGS	United States Geological Survey

1.0 INTRODUCTION

NV5 performed a geotechnical engineering investigation, conducted a geologic hazards evaluation and prepared a Geotechnical Engineering and Geologic Hazards Report for the proposed new stage and bleacher addition to the existing multipurpose room at Margaret G. Scotten School in Grass Valley, California, consistent with the scope of services presented in NV5's *Proposal for Geologic Hazards and Geotechnical Engineering Investigation Services* (PN18220), dated November 5, 2018. NV5's findings, conclusions and recommendations are presented herein.

For your review, Appendix A presents a document prepared by the Geoprofessional Business Association (GBA) entitled *"Important Information about This Geotechnical Engineering Report."* This document summarizes project specific factors, limitations, content interpretation, responsibilities and other pertinent information.

1.1 SCOPE-OF-SERVICES

NV5 performed a specific scope-of-services to develop geotechnical engineering design recommendations for earthwork and structural improvements. Brief descriptions of each work scope task are presented below. A detailed description of each work scope task is presented in Section 2 ("Site Investigation") of this report.

- Task 1, Site Investigation: NV5 performed a site investigation to characterize the existing surface and subsurface soil, rock and groundwater conditions encountered to the maximum depth excavated. NV5's field engineer/geologist made observations, took representative soil samples, conducted seismic refraction surveys, and performed field tests at a limited number of subsurface exploratory locations. NV5 performed laboratory tests on selected soil samples to evaluate their engineering material properties.
- Task 2, Data Analysis and Engineering Design: NV5 evaluated the field and laboratory site data and the proposed site improvements and used this information to develop geotechnical engineering design recommendations for earthwork and structural improvements. NV5 used engineering judgment to extrapolate NV5's observations and conclusions regarding the field and laboratory data to other onsite areas located between and beyond the locations of NV5's subsurface exploratory excavations.
- Task 3, Report Preparation: NV5 prepared this report to present the findings, conclusions and recommendations for this geotechnical engineering investigation.

1.2 SITE LOCATION AND DESCRIPTION

The proposed new stage and bleacher addition to the existing multipurpose room is located at Margaret G. Scotten School in Grass Valley, California. The proposed building footprint is located in the northeastern portion of the existing Margaret G. Scotten School campus. The site is centered at about latitude 39.2223 North and longitude 121.0763 West on the United States Geological Survey's (USGS), 7.5 minute Grass Valley Quadrangle topographic map. The property elevation is approximately 2580 feet above mean sea level (amsl), based on review of the USGS 7.5-minute Grass Valley Quadrangle topographic map, and is generally moderately sloping towards the west. Figure 1 shows the site location and vicinity.

Figure 2 shows the existing property conditions and exploration work performed at the site. At the time the site investigation was performed on December 27, 2018, the following conditions were observed:

- The proposed new stage and bleacher addition area is located along the northwestern perimeter of the existing multipurpose room.
- The project site area is generally flat with very low topographic relief and currently supports exterior concrete flatwork with surrounding irrigated planter areas.
- Classrooms and school office buildings were located to the southwest and west of the multipurpose room. An exterior flatwork playground area is located to the southeast of the multipurpose room. An exterior flatwork parking area is located to the north and northeast of the multipurpose room.

1.3 PROPOSED IMPROVEMENTS

Based on preliminary project information provided by representatives of the Grass Valley Elementary School District (GVSD), the proposed 2019 campus modernization improvements include renovations to multiple site structures, including the new stage and bleacher addition to the existing multipurpose room, as well as, exterior hardscape improvements. The planned stage and bleacher addition to the existing multipurpose room is new construction consisting of a single-story structure, approximately 1,550 square feet in size, using wood framing on shallow perimeter and isolated spread foundations, concrete slab-on-grade floor, and concrete slab on grade sidewalks and landscape improvements. Earthwork grading will involve minor cuts and fills to meet the proposed building grades.

1.4 INVESTIGATION PURPOSE

The purpose of the geologic hazard evaluation and geotechnical investigation was to obtain sufficient on-site information about the soil, rock and groundwater conditions to facilitate the updated evaluation of potential geologic hazards described in the subsequent sections of this report and provide geotechnical engineering recommendations for the proposed earthwork and structural improvements. As part of this contract, NV5 did not evaluate the site for the presence of hazardous waste, mold, asbestos and radon gas. Therefore, the presence and removal of these materials are not discussed in this report.

2.0 SITE INVESTIGATION

NV5 performed a site investigation to characterize the existing surface and subsurface conditions beneath the proposed stage and bleacher addition to the existing multipurpose room. The site investigation included a literature review of published and unpublished geologic documents and maps, a surface reconnaissance investigation, and a subsurface exploratory investigation using seismic refraction survey equipment and a truck-mounted drill rig to excavate exploratory borings. Each component of the site investigation is presented below.

2.1 LITERATURE REVIEW

NV5 performed a limited review of available literature that was pertinent to the project site. The following summarizes NV5's findings:

2.1.1 Site Improvement Plans

Improvement plans were not available for review at the time this report was prepared.

2.1.2 Previous Site Investigation Reports

Previous site investigation reports were not available for review at the time this report was prepared.

2.2 REGIONAL GEOLOGY

The Margaret G. Scotten School site is located in the Sierra Nevada Foothills, on the western side of the Sierra Nevada geomorphic province. The Sierra Nevada province is an elongate, north-west trending structural block that is tilted upward so that it forms a steep scarp above the adjacent Basin and Range province to the east. The west slope of the Sierra Nevada dips gently westward, and the western portion of the block is buried beneath sediment of the Great Valley province. Sediment within the Great Valley is derived from continual uplift and erosion of the Sierra Nevada.

The regional geology of the western foothills of the Sierra Nevada is generally comprised of a complex assemblage of igneous and metamorphic rocks. The regional structure of the foothills is characterized by the north-northwest trending Foothills Fault System, a feature formed during the Mesozoic era (between approximately 65 million and 248 million years ago) in a compressional tectonic environment. A change to an extensional tectonic environment during the late Cenozoic (approximately within the last 30 million years), resulted in normal faulting which has occurred coincident with some segments of the older faults in the region.

2.3 SITE GEOLOGY

Based on review of the *Geologic Map of the Chico Quadrangle*, published by the California Division of Mines and Geology (Saucedo and Wagner, 1992), the geology underlying the subject site is comprised of volcanic rocks of the Miocene-Pliocene Epochs (23 to 2.59 million years before present). Saucedo and Wagner (1992) generalize the volcanic deposits underlying the subject site as andesitic pyroclastic rocks. The *Geologic Map of the Chico Quadrangle* also identifies a geologic unit to the south of the project site as massive diabase associated with the Paleozoic and Mesozoic (541 to 66 million years before present) Lake Combie complex. A regional geologic map including the site area is provided as Figure 3.

2.4 REGIONAL FAULTING AND SEISMIC SOURCES

Regional faulting is associated with the central area of the Foothill Fault System which includes the Spenceville Fault, Deadman Fault, Wolf Creek Fault Zone, Giant Gap Fault, Grass Valley Fault, Weimar Fault Zone, Foresthill Fault and the Ramshorn Fault. The Foothill Fault System is a broad zone of northwest trending east dipping normal faults formed along the margin of the Great Valley and the Sierra Nevada geologic provinces on the western flank of the Sierra Nevada and southern Cascade mountain ranges. The central part of the fault zone is split into branches: the Melones Fault Zone to the east, the Cleveland Hill Fault to the northwest, the Spenceville Fault to the west, the Wolf Creek Fault Zone to the south and the Grass Valley Fault Zone in the area of the subject site.

NV5 reviewed the Official Maps of Earthquake Fault Zones delineated by the California Geological Survey through December 2010, on the internet at http://www.quake.ca.gov/gmaps/WH/regulatorymaps.htm. These maps are updates to Special Publication 42, Interim Revision 2007 edition *Fault Rupture Hazard Zones in California*, which describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. Special Publication 42 and the 2010 on-line update indicate that the site is not located within an Alquist-Priolo active fault zone. There are currently no proposed earthquake fault zone maps in the immediate area of Grass Valley, California.

According to the *Fault Activity Map of California and Adjacent Areas* (Jennings, 1994), the closest known active fault which has surface displacement within Holocene time (about the last 11,000 years) is the Cleveland Hills Fault. The 2010 Fault Activity Map of California by the California Geological Survey, (*http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#*), Geologic Data Map No. 6 shows the nearest known active fault with surface displacement within Holocene time to be the Cleveland Hill Fault. The mapped fault zone is located approximately 27 miles northwest of the subject site and is associated with ground rupture during the Oroville earthquakes of 1975. The approximate location of the Margaret G. Scotten school campus (Site Location) identified on the *Fault Activity Map of California and Adjacent Areas* (Jennings, Charles W., 1994) is presented as Figure 4.

2.5 FIELD INVESTIGATION

NV5 performed a field investigation of the site on December 27, 2018. NV5's field engineer/geologist described the surface and subsurface soil, rock and groundwater conditions observed at the site using the procedures cited in the ASTM International (ASTM), Volume 04.08, *Soil and Rock; Dimension Stone; and Geosynthetics* as general guidelines. The field engineer/geologist described the soil color using the general guideline procedures presented in the Munsell® Soil-Color Chart. Engineering judgment was used to extrapolate the observed surface and subsurface soil, rock and groundwater conditions to areas located between and beyond the subsurface exploratory locations. The surface, subsurface and groundwater conditions observed during the field investigation are summarized below.

2.5.1 Surface Conditions

NV5 observed the following surface conditions during the field investigation of the property. Figure 2 shows the project site and the approximate existing building footprints and boundaries. A major portion of the proposed multipurpose room addition location supports an exterior sidewalk hardscape with surrounding irrigated planter areas. It was noted, that the root systems of the existing trees at the site were exposed at the ground surface. NV5 did not observe any surface conditions of concern during the site investigation.

2.5.2 Subsurface Conditions

The subsurface soil, rock and groundwater conditions were investigated by drilling exploratory borings and a seismic refraction survey performed along the western boundary of the existing multipurpose room. The subsurface information obtained from this investigation method is described in the following subsections.

2.5.2.1 Exploratory Boring Information

NV5 provided engineering oversight for the excavation of 2 exploratory soil borings at the project site. The borings were advanced with a truck-mounted GEFCO drill rig equipped with 6-inch diameter, continuous flight, solid stem augers. Figure 2 shows the approximate locations of the subsurface exploratory borings. Borings B-1 and B-2 were advanced until encountering practical refusal to drilling at the maximum depths of 3.25 to 8 feet below ground surface (bgs). Engineering judgment was used to extrapolate the observed soil, rock and groundwater conditions to areas located between and beyond the subsurface exploratory borings.

NV5's field engineer/geologist logged each exploratory boring using the ASTM D2487 Unified Soils Classification System (USCS) as guidelines for soil descriptions and the American Geophysical Union guidelines for rock descriptions. Relatively undisturbed soil samples were collected with a 2.5-inch inside diameter split-spoon sampler equipped with 6-inch long stainless steelliner sampler tubes. The sampler was driven into the soil using an automatic trip hammer weighing 140 pounds with a 30-inch free-fall. The sampler was driven 12 to 18 inches. The blows required to drive each 6-inch increment were recorded and the blows required to drive the last 12 inches, or portion thereof, were converted to equivalent Standard Penetration Test (SPT) blow counts using a conversion factor of 0.65 (Burmister, 1948) for correlation with empirical data. Disturbed samples were also obtained at selected depths by driving a 1.375-inch inside diameter (2-inch outside diameter) Standard Penetration Test (SPT) sampler, without liners or rings, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches, the blows to drive each 6-inch increment were recorded, and the blows required to drive the final 12 inches, or portion thereof, are provided on the boring logs. The stainless-steel liner and bulk samples were sealed and labeled. All of the representative soil samples were transported to the NV5 Nevada City office soil laboratory facility.

Detailed descriptions of the soil, rock and groundwater conditions that were encountered in each subsurface exploratory location are presented on the exploratory boring logs included in Appendix B. The soil and rock descriptions include: visual field estimates of the particle size percentages (by dry weight), color, relative density or consistency, moisture content and cementation that comprise each soil material encountered.

A generalized profile of the soil, rock and groundwater conditions encountered to the maximum depth explored (8 feet bgs) for the proposed building areas is presented below. The soil and/or rock units encountered in the subsurface exploratory excavations were generally stratigraphically continuous across the site with some variations in gradations and thicknesses. The units encountered in general stratigraphic sequence during the subsurface investigation of the site are described below.

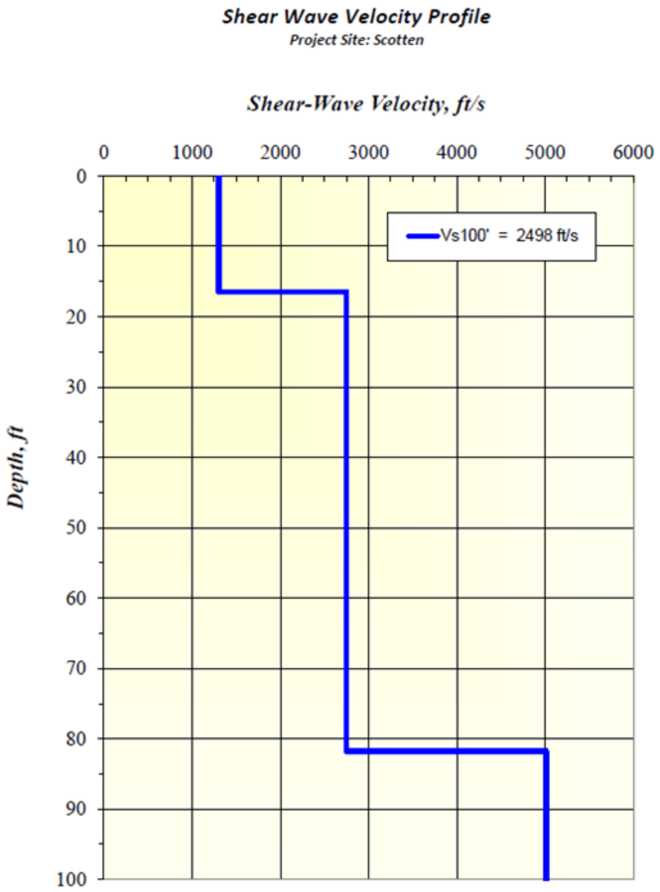
- **CL, Low Plasticity Sandy Clay Soil**: This soil is considered to be fill consisting of the following field estimated particle size percentages: 65-70 percent low to medium plasticity silt and clay, 30-35 percent fine to coarse sand. This soil varies from dark gray to yellowish red with Munsell® Soil-Color Chart designations of (5YR 4/1 & 5YR 4/6). This soil was medium stiff to stiff and moist to wet at the time of the subsurface investigation.
- **GM, Silty Gravel with Sand Soil**: This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 15-30 percent low to medium plasticity silt and clay, 20-35 percent fine to coarse sand, and 50 percent fine to coarse gravels. This soil varies from yellowish red with Munsell® Soil-Color Chart designations of (5YR 4/6). This soil was loose to medium dense and moist at the time of the subsurface investigation.
- **Rx, Massive Diabase Rock (Lake Combie complex):** This bedrock unit is colored dark bluish gray with a Munsell® Soil-Color Chart designation of (Gley 2 4/1) with white inclusions. The rock was described as strongly cemented, slightly to moderately weathered, very strong, massive and damp at the time of the subsurface investigation.

NV5 prepared a geologic cross section using the geologic boring logs from exploratory borings B-1 and B-2. The alignment of the geologic cross section is presented in Figure 2. The geologic cross section is presented in Figure 5.

2.5.2.2 Seismic Refraction Microtremor Survey

The Seismic Refraction Microtremor Survey (SRMS) performed at the site used the SeisOpt® ReMi™ Vs30 method to determine the in-situ shear-wave (S-wave) velocity profile (Vs Model) of the uppermost 100 feet (30 meters) of soil/rock beneath the site. The measured S-wave profile is used to determine the 2016 California Building Code (CBC) Site Class in accordance with Chapter 16A, Section 1613A.3.2 and Chapter 20 of ASCE 7-10.

The SRMS method is performed at the surface using a conventional seismograph equipped with geophones that record both seismic compression waves (P-waves) and S-waves. The P-wave and S-wave sources consist of ambient seismic microtremors which are constantly being generated by cultural activities and natural noise in the area. NV5 recorded the seismic vibrations generated by local pedestrian activity for some of the data acquisition recordings. The data was collected in a series of twelve, 30-second-long, continuous recording periods. The *Shear Wave Velocity Profile* below shows the Vs Model subsurface shear-wave velocity profile for the site that was developed from the SeisOpt®ReMi™ data.



The Vs Model developed for the site indicates that the harmonic mean seismic shear wave velocity for the upper 100 feet of the subsurface is approximately 2,498 feet per second (ft/s). This weighted shear wave velocity corresponds to the higher range of Site Class C (very dense soil and soft rock profile 1,200 – 2,500 ft/s), as described in Chapter 20, Table 20.3-1 Site Classification of ASCE 7-10.

2.5.2.3 Groundwater Conditions

The groundwater table was not encountered within the two exploratory borings to the maximum depths of approximately 3.25 to 8 feet bgs. The moisture content of each soil unit described on the exploratory boring logs is considered the natural moisture within the vadose soil zone (soil situated above the groundwater table).

NV5 used the California Department of Water Resources (DWR) Well Completion Report Map Application (https://dwr.maps.arcgis.com/apps/webappviewer/index.html?id=181078580a214c0986e2da28f8623b37) to review historical groundwater depth data in the immediate area. Based on review of groundwater data generated from a domestic water supply well approximately 450-feet north of the project site, the depth to the groundwater table was approximately 100 feet bgs (approximately 2470 feet amsl) in 1991.

3.0 LABORATORY TESTING

NV5 performed laboratory tests on selected soil samples taken from the subsurface exploratory excavations to determine their geotechnical engineering material properties. These engineering material properties were used to develop geotechnical engineering design recommendations for earthwork and structural improvements. The following laboratory tests were performed using the cited ASTM guideline procedures:

- ASTM D1140 Particle Size analysis (No. 200 Mesh Wash)
- ASTM D2216 Soil Moisture Content
- ASTM D2487 Soil Classification by the USCS
- ASTM D2166 Unconfined Compression
- ASTM D2937 In Place Density of Soil
- ASTM D4318 Atterberg Indices (Dry Method)

Table 3.0-1 presents a summary of the geotechnical engineering laboratory test results. Appendix C presents the laboratory test data sheets.

Table 3.0-1, Laboratory Test Results

Boring	**Sample**		**ASTM Test Results**$^{(1)}$									
			D2487/D2488	**D2216**	**D2937**	**D422**		**D4318**		**D2166**	**D3080**	
No.	***No.***	***Depth (ft)***	***USCS (sym)***	***Moisture Content (%)***	***Dry Density (pcf)***	***Passing No. 4 Mesh Sieve (%)***	***Passing No. 200 Mesh Sieve (%)***	***Plasticity Index (%)***	***Liquid Limit (%)***	***UC Compressive Strength (psf)***	***DS Friction Angle (degrees)***	***Cohesion (psf)***
B-1	B1-L1-3	2.5	GM	33.2	84.7	--	--	--	--	--	--	--
B-2	B2-L2-2	2.5	CL	38.1	79.5	--	--	--	--	2623.8	--	1311.9*
B-2	B2-L3-2	5.5	GM	49.6	72.9	84	16	NP	NP	--	--	--

Notes: (1) Laboratory test forms are presented in Appendix C
ASTM = ASTM International
USCS = Unified Soils Classification System
UC = Unconfined Compression
DS = Direct Shear
No. = number
% = percent
ft = feet
Sym = symbol
Psf = pounds per square foot
NP = Non Plastic
* = Half of unconfined compressive strength.

4.0 HISTORICAL SEISMICITY

The regional geology and faulting are discussed in Section 2 of this report. NV5 used the USGS National Earthquake Information Center (NEIC) Earthquake Search Results on-line database (https://earthquake.usgs.gov/earthquakes/search/) to identify historical seismic activity within a 100 kilometer (km) (62 miles) radial distance of the subject site. The database includes several moderate size earthquakes (greater than magnitude 5.4 local magnitude [M_L]) that occurred in the Sacramento Valley and Sierra Nevada transition areas since 1836. These earthquakes include the following events:

- The September 12, 1966,5.9 ML shock occurred approximately within 1 mile (1.6 km) north of the town of Truckee, which is located approximately 44.75 miles (72 km) east-north-east from the subject site. This earthquake was accompanied by ground breakage over a 10 mile long zone indicative of subsurface fault movement. Minor damage occurred to the Prosser and Boca dams, interstate 80 bridges along with several landslides, and the initial shock was felt as far west as San Francisco and east as Salt Lake City (Kachadoorian, Yerkes & Waananen 1967).

- The August 1, 1975 ML Oroville Earthquake main shock occurred on the Cleveland Hill Fault with the hypocentre located approximately 0.6 miles (1 km) east-northeast of Palermo. This is approximately 29.5 miles (47.5 km) northwest of the subject site. The earthquake was accompanied by linear surface faulting extending for approximately 4.3 miles (7km) eastward of the main shock center (DWR 1979). The earthquake sequence consisted of five foreshocks (ML 3 or greater), the main shock, and numerous aftershocks (Toppozada and Cramer, 1984).

5.0 LIQUEFACTION AND SEISMIC SETTLEMENT

NV5 evaluated the potential for liquefaction occurring at this site on the basis of the geologic units encountered in exploratory borings, standard penetration test (SPT) blow count data, probabilistic expected seismic ground acceleration analysis, and literature review.

5.1 LIQUEFACTION

Soil liquefaction results when the shear strength of a saturated soil decreases to zero during cyclic loading that is generally caused by machine vibrations or earthquake shaking. Generally, saturated, clean, loose, uniformly graded sand and loose, silty sand soils of Holocene age are the most prone to undergo liquefaction, however, saturated, gravelly soil, and some clay-rich soil may be prone to liquefaction under certain conditions. The onsite soil composed of two to three feet of medium stiff to stiff, cohesive soil (sandy clay fill) overlying approximately 1 to 4 feet of loose to medium dense, moist, silty gravel with sand. Groundwater was not encountered in the borings at depths ranging from approximately 3.25 to 8 feet bgs. Practical refusal to drilling was encountered in very strong, massive diabase bedrock at 3.25 and 8 feet bgs in Borings B-1 and B-2. Groundwater data collected from nearby domestic wells indicate the groundwater table was encountered at approximately 100 feet bgs in the area. The site soil conditions and recent groundwater depth make the probability of liquefaction occurring during ground shaking caused by a maximum considered earthquake (MCE) to be very low at the property. Therefore, based on this information, NV5 believes that the results of the seismic refraction survey and site soil and rock conditions make the probability of liquefaction occurring during a nearby earthquake to be very low.

5.2 SEISMIC SETTLEMENT AND LATERAL SPREADING

Because the potential for liquefaction of the soil is considered low, the onsite soils encountered beneath the site and the relatively gently sloping terrain of the site and surrounding areas, NV5 considers there to be a low probability for the occurrence of post-liquefaction settlement and lateral spreading that would be detrimental to the proposed site improvements.

6.0 OTHER GEOLOGIC HAZARDS

NV5's evaluation of geologic hazards for the site was based on review of geologic maps and literature, regional aerial photographs, a site reconnaissance, and analysis of the soil and rock conditions encountered during the December 27, 2018 site investigation. This section provides additional information to meet the conditions of the 2016 CBC and California Geological Survey Note 48 (October 2013). The existing school campus site is not located within special geologic hazard zones designated by the California Geologic Survey or local building departments for liquefaction and landslides. The following presents NV5's evaluation of pertinent geologic hazards and their potential to negatively impact the site.

6.1 EXPANSIVE SOIL

The site soil conditions observed during the surface reconnaissance and during the subsurface geotechnical investigation are characterized as fine grained fill (sandy clay) and coarse grain (silty gravel with sand) size soils. Atterberg Limits (ASTM D4318) testing was performed on a representative near-surface soil sample collected during the subsurface investigation. The Atterberg Limits test results indicate the fine soil material to be non-plastic (ML) soil. Based on the results of the Atterberg Limits testing and our experience with similar soils in the area, the potential for expansive soil hazards to affect the proposed buildings is considered to be very low.

6.2 SOIL CORROSION POTENTIAL

The site soil corrosion potential was evaluated by Sunland Analytical. The soil was found to be moderately corrosive.Any buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion. The soil sample tested was collected at a depth of approximately 3 feet bgs from Boring B-2. The test results are summarized in Table 6.2-1 below.

Table 6.2.-1, Summary of Corrosion Potential Lab Test Data

Boring No.	Sample No.	Sample Depth (ft)	Test No.	Description	Test Results
B-2	B2-L2-3	3	ASTM G-200m	Redox	273 mV
B-2	B2-L2-3	3	CA DOT Test #422m	Chloride	37.3 ppm
B-2	B2-L2-3	3	CA DOT Test #417	Sulfate	0.4 ppm
B-2	B2-L2-3	3	CA DOT Test #643 mod. (Sm. Cell)	PH	5.64
B-2	B2-L2-3	3	CA DOT Test #643 mod. (Sm. Cell)	Min. Resistivity	5,900 ohms-cm

Notes:	ASTM	ASTM International
	CA DOT	California Department of Transportation
	ft	feet
	ppm	parts per million
	mV	millivolts
	ohms- c	ohms-centimeters

The chloride and sulfate concentrations less than 500 ppm and 2000 ppm, respectively, are not considered corrosive to reinforced concrete structures and cement mortar-coated steel. Typical concrete mix designs from this area contain Type II/V cement.

Based on these limited tests (i.e., Redox, pH, resistivity, chloride, sulfate, and sulfide) the soil is considered moderately corrosive to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. All buried metallic piping should be protected against corrosion in accordance with the pipe manufacture recommendations. The laboratory report is included in Appendix E.

We reviewed the Online Soil Survey prepared by the USDA Soil Conservation Service (https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx). Based on review of soil survey information the native soil conditions onsite possess a moderate corrosion potential for concrete and a high corrosion potential for uncoated steel. To reduce the likelihood of corrosion problems, materials used for underground utilities, permanent subsurface drainage improvements, and foundation systems should be selected based on local experience and practice. If alternative or new construction methods or materials are being proposed, it may be appropriate to have the selected materials evaluated by a corrosion engineer for compatibility with the onsite soil and groundwater conditions.

6.3 VOLCANIC HAZARDS

According to the USGS Map of Potential Areas of Volcanic Hazards (Miller, 1989), the property is not situated within a recognized active volcanic area. The nearest known active volcanic zone is the Mono Lake Volcanic Field, approximately 125 miles southeast of the site. The most recent volcanic eruptions occurring at the Mono Lake Volcanic Field were from 300 years ago. In summary, NV5’s opinion is that there is very low potential for encountering a volcanic hazard within the proposed building footprint area.

6.4 FLOODING

The subject property is not located within any 100-year flood zone, a Special Flood Hazard Area (SFHA) as designated by the Federal Emergency Management Agency (FEMA). FEMA is required by federal law to compile Flood Insurance Rate Maps identifying areas of potential flooding. Property located within a SFHA is subject to a one percent (1%) or greater chance of complete or partial flooding in any given year. FEMA defines this type of flood as the "base flood" which is more commonly known as a "100-year-flood". A 100-year-flood has a 26 percent chance of occurring during any 30-year period. Based on review of the Nevada County Unincorporated Areas FEMA Flood Insurance Rate Map (FIRM) 060210, dated February 3, 2010, the site is located within Zone X, defined as areas outside the 0.2 percent annual chance floodplain. Therefore, there is minimal flood hazard at the site. NV5’s opinion is that the potential for stream induced-flooding and earthquake-induced flooding hazards that would negatively impact the proposed building footprint areas are extremely low.

6.5 TSUNAMIS AND SEICHES

There are no bodies of water with the potential for tsunamis and or sieches located near the subject property. In summary, we believe that the potential for encountering tsunami and/or seiches hazards within the proposed building foot-print area is not probable.

6.6 SLUMPS AND LAND SUBSIDENCE

NV5 did not observe slumps or hummocky surface feature depressions that indicate the occurrence of land subsidence. Generally, the site is underlain by medium stiff to stiff sandy clay, over loose to medium dense silty gravels with sand. NV5's opinion is that the potential for slumping and land subsidence hazards to occur within the native soil or rock sections encountered within the proposed building footprint areas is low.

6.7 LANDSLIDES

The existing topography at the site and near vicinity consists of gently to moderately sloping terrain. The site is not located in an area of known historical landslides and there is no indication of historic landslides, including rock falls, debris flows or deep and shallow failure. Therefore, the potential for the occurrence of a landslide hazard at the proposed building footprint area is considered to be remote.

6.8 MINING RELICS

NV5 did not observe any evidence of past mining activities during site reconnaissance. Review of available geologic maps and mine related literature did not show any past mining activities at the site or immediately surrounding area. If any evidence of mining activity is encountered during grading then additional geotechnical engineering or environmental assessment may be warranted. In summary, NV5 considers the potential for encountering past mining related hazards within the proposed building footprint areas to be low.

7.0 CONCLUSIONS

The conclusions presented in this section are based on information developed from the field and laboratory investigations.

1. It is NV5's opinion that the site is suitable for the proposed improvements provided that the geotechnical engineering design recommendations presented in this report are incorporated into the earthwork and structural improvement project plans. Prior to construction, NV5 should be allowed to review the proposed final earthwork grading plan and structural improvement plans to determine if the geotechnical engineering recommendations have been properly incorporated, are still applicable or need modifications.
2. Based on the site geology, the observations within the exploratory borings, and the SeisOpt ReMi Vs30 shear-wave profile analysis, the site soil profile can be modeled, according to the 2016 CBC, Chapter 16*A*, and ASCE 10-7, Chapter 20, as a Site Class C (very dense soil and soft rock) designation for the purposes of establishing seismic design loads for the proposed improvements.
3. Based on the subsurface exploratory boring blow counts, other field data, and literature review, NV5 believes that the site soil and groundwater conditions make the probability of liquefaction occurring during a nearby earthquake to be low.
4. The soil conditions observed to a maximum depth of 8 feet below the existing ground surface in our subsurface exploratory excavations (described relative to the existing ground surface) generally consisted of medium stiff to stiff, sandy clay (CL) fill underlain by loose to medium dense, silty gravel with sand (GM), underlain by strongly cemented, slightly to moderately weathered, very strong, massive diabase bedrock.
5. NV5's field and laboratory test data indicates that the sandy clay (CL) fill and silty gravel with sand (GM) soil units encountered beneath the site has the following general geotechnical engineering properties: medium stiff to stiff/loose to medium dense with a moderate bearing capacity that is suitable for supporting shallow foundations, provided on-site soil conditions are confirmed during construction by a representative of NV5.
6. Groundwater was not encountered in the exploratory borings to the maximum depths explored of 3.25 to 8 feet bgs at the time of this subsurface investigation. Based on the above average rainfall, subsurface geologic conditions and review of domestic well data near the site, NV5 assumes that for design and evaluation purposes, the historically high groundwater table will probably be located at a depth greater than approximately 50 feet bgs.

8.0 RECOMMENDATIONS

NV5 developed geotechnical engineering design recommendations for earthwork and structural improvements from the field and laboratory investigation data. Subsequent to earthwork and site preparation, it is anticipated that the structure may be founded on conventional continuous and/or spread footings founded in properly compacted fill. NV5's recommendations are presented below.

8.1 EARTHWORK GRADING

NV5's earthwork grading recommendations include: import fill soil, temporary excavations, stripping and grubbing, native soil preparation for engineered fill placement, engineered fill construction with testable earth materials, cut-fill transitions, cut and fill slope grading, erosion controls, underground utility trenches, construction de-dewatering, soil corrosion potential, subsurface groundwater drainage, surface water drainage, grading plan review and construction monitoring.

8.1.1 Import Fill Soil

Import fill soil should meet the geotechnical engineering material properties described in Section 8.1.5.1 (Engineered Fill Construction with Non-Expansive Soil) of this report. Prior to importation to the site, the source generator should document that the import fill meets the guidelines set forth by the California Environmental Protection Agency (CalEPA) Department of Toxic Substances Control (DTSC) in their 2001 "Information Advisory, Clean Imported Fill Material." This advisory represents the best practice for characterization of soil prior to import for use as engineered fill. The project engineer should approve all proposed import fill soil for use in constructing engineered fills at the site.

8.1.2 Temporary Excavations

All temporary excavations must comply with applicable local, state and federal safety regulations, including the current Occupational Safety and Hazards Administration (OSHA) excavation and trench safety standards. Construction site safety is the responsibility of the contractor, who is solely responsible for the means, methods and sequencing of construction operations. Under no circumstances should the findings, conclusions and recommendations presented herein be inferred to mean that NV5 is assuming any responsibility for temporary excavations, or for the design, installation, maintenance and performance of any temporary shoring, bracing, underpinning or other similar systems. NV5 could provide temporary cut slope gradients, if required.

8.1.3 Stripping and Grubbing

The site should be stripped and grubbed of vegetation and other deleterious materials, as described below.

1. Following demolition of existing exterior hardscape in the proposed improvement area, strip and remove the underslab sand and top 2 to 4 inches of sandy clay fill and other deleterious materials from the proposed improvement area. Grub the underlying 6 to 8 inches of soil to remove any large vegetation roots or other deleterious material while leaving the soil in place. The project geotechnical engineer or his/her representative should approve the use of any soil materials generated from the clearing and grubbing activities.

2. Remove all existing underground utilities extending through the proposed building pad. Excavate the remaining cavities or holes to a sufficient width so that an approved backfill soil can be placed and compacted in the cavities or holes. Enough backfill soil should be placed and compacted in order to match the surrounding elevations and grades. The project engineer or his/her representative should observe and approve the preparation of the cavities and holes prior to placing and compacting engineered fill soil in the cavities and holes.
3. If encountered, excessively large amounts of vegetation, other deleterious materials and oversized rock materials should be removed from the site.

8.1.4 Native Soil Preparation for Engineered Fill Placement

After completing site stripping and grubbing activities, the exposed native soil in proposed fill areas should be prepared for placement and compaction of engineered fills, as described below.

1. The native soil should be scarified to a minimum depth of 8 inches below the existing land surface, or stripped and grubbed surface, and then uniformly moisture conditioned. If the soil is classified as a coarse-grained soil by the USCS (i.e., GP, GW, GC, GM, SP, SW, SC or SM) then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified as a low plasticity fine-grained soil by the USCS (i.e., CL, ML), then it should be moisture conditioned to between 2 and 4 percentage points greater than the ASTM D1557 optimum moisture content. If soil is classified as a high plasticity fine-grained soil by the USCS (i.e., CH, MH), the soil should be removed from the building pad area or contact NV5 for further recommendations.
2. The native soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry unit weight (density). The moisture content, density and relative percent compaction should be tested by the project engineer or his/her field representative to evaluate whether the compacted soil meets or exceeds the minimum percent compaction and moisture content requirements. The earthwork contractor shall assist the project engineer or his/her field representative by excavating test pads with the on-site earth moving equipment. Native soil preparation beneath concrete slab-on-grade structures (i.e., floors, sidewalks, patios, etc.) should be prepared as specified in Section 8.2 (Structural Improvements).
3. The prepared native soil surface should be proof-rolled with a fully-loaded 4,000-gallon-capacity water truck with the rear of the truck supported on a double-axle, tandem-wheel undercarriage or approved equivalent. The proof-rolled surface should be visually observed by the project geotechnical engineer or his/her field representative to be firm, competent and relatively unyielding. The project engineer or his/her field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe, however, this evaluation method should not be performed in place of proof rolling as described above.
4. Construction Quality Assurance (CQA) tests should be performed using the minimum testing frequencies presented in Table 8.1.4-1 or as modified by the project engineer to better suit the site conditions.
5. The native soil surface should be graded to minimize ponding of water and to drain surface water away from the building foundations and associated structures. Where possible, surface water should be collected, conveyed and discharged into natural drainage courses, storm sewer inlet structures, permanent engineered storm water runoff percolation/evaporation basins or engineered infiltration subdrain systems.

Table 8.1.4-1, Minimum Testing Frequencies

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 1,500 CY or Material Change(2)
D6938	Nuclear Density and Nuclear Moisture Content	1 per 250 CY

Notes:
(1) These are minimum testing frequencies that may be increased or decreased at the project geotechnical engineer's discretion on the basis of the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.
ASTM = ASTM International
CY = cubic yards
No. = number

8.1.5 Engineered Fill Construction with Testable Earth Materials

Engineered fills are constructed to support structural improvements. Engineered fills should be constructed using non-expansive soil as described in Section 8.1.5.1. If possible, the use of expansive soil for constructing engineered fills should be avoided. If the use of expansive soil cannot be avoided, then engineered fills should be constructed as described in Section 8.1.5.2 or as modified by the project engineer. If soil is to be imported to the site for constructing engineered fills, then NV5 should be allowed to evaluate the suitability of the borrowed soil source by taking representative soil samples for laboratory testing. Testable earth materials are generally considered to be soils with gravel and larger particle sizes retained on the No. 4 mesh sieve that make up less than 30 percent by dry weight of the total mass. The relative percent compaction of testable earth materials can readily be determined by the following ASTM test procedures: laboratory compaction curve (D1557), field moisture and density (D6938). Construction of engineered fills with non-expansive and expansive testable earth materials is described below.

8.1.5.1 Engineered Fill Construction with Non-Expansive Soil

Construction of engineered fills with non-expansive soil should be performed as described below.

1. Non-expansive soil used to construct engineered fills should consist predominantly of materials less than ½-inch in greatest dimension and should not contain rocks greater than 3 inches in greatest dimension (oversized material). Non-expansive soil should have a plasticity index (PI) of less than or equal to 15, as determined by ASTM D4318 Atterberg Indices testing. Oversized materials should be spread apart to prevent clustering so that void spaces are not created. The project engineer or his/her field representative should approve the use of oversized materials for constructing engineered fills.
2. Non-expansive soil used to construct engineered fills should be uniformly moisture conditioned. If the soil is classified by the USCS as coarse grained (i.e., GP, GW, GC, GM, SP, SW, SC or SM), then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified by the USCS as fine grained (i.e., CL, ML), then it should be moisture conditioned to between 2 and 4 percentage points greater than the ASTM D1557 optimum moisture content.
3. Engineered fills should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose lifts (layers) prior to compacting.

4. The soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
5. The earthwork contractor should compact each loose soil lift with a tamping foot compactor such as a Caterpillar (CAT) 815 Compactor or equivalent as approved by NV5's project engineer or his/her field representative. A smooth steel drum roller compactor should not be used to compact loose soil lifts for construction of engineered fills.
6. The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 8.1.5.1-1 or as modified by the project engineer to better suit the site conditions.

Table 8.1.5.1-1, Minimum Testing Frequencies for Non-Expansive Soil

ASTM No.	Test Description	Minimum Test Frequency(1)
D1557	Modified Proctor Compaction Curve	1 per 1,500 CY or Material Change (2)
D6983	Nuclear Moisture and Density	1 per 250 CY

Notes: (1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.
ASTM = ASTM International
CY = cubic yards
No. = number

7. The moisture content, density and relative percent compaction of all engineered fills should be tested by the project engineer's field representative during construction to evaluate whether the compacted soil meets or exceeds the minimum compaction and moisture content requirements. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth-moving equipment.
8. The prepared finished grade or finished subgrade soil surface should be proof-rolled, as mentioned above in Section 8.1.4, Paragraph 3.

8.1.5.2 Engineered Fill Construction with Expansive Soil

NV5 did not encounter highly expansive soil within the shallow soil or zone that would be influenced by the foundation loads at the site during the subsurface investigation. If expansive soils are encountered during grading of the site, and if the property owner desires to use expansive soil to construct engineered fills, then NV5 should be notified to prepare recommendation options for constructing fills with potentially expansive soil.

8.1.6 Cut-Fill Transitions

NV5 did not review any grading plan at the time that this report was prepared; however, we do not anticipate that site conditions during construction will generate a cut-fill transition with fills greater than 3 feet thick. Care should be taken when removing existing foundations and re-routing underground utilities so that large excavations are not opened which could inadvertently result in differing soil conditions between native soil and utility backfill that could be subject to differential settlement. If fills greater than 3 feet are planned, or demolition requires deep and wide excavations, NV5 should be notified so that additional recommendations to properly construct the fill pad beneath the project location can be provided to ensure that a cut-fill transition is not constructed that may be subject to differential settlement in the future.

8.1.7 Cut and Fill Slope Grading

NV5 does not anticipate that grading of cut and fill slopes will have vertical heights greater than 3 feet at the site. In general, both cut and fill slopes should be graded at a maximum slope gradient of 2H:1V (horizontal to vertical slope ratio). Surface water should not be allowed to flow over the cut and fill slopes graded at the site. If steeper cut and/or fill slopes are designed, then NV5 should be allowed to review the proposed cuts and provide additional recommendations as appropriate.

8.1.8 Erosion Controls

Erosion controls should be installed as described below.

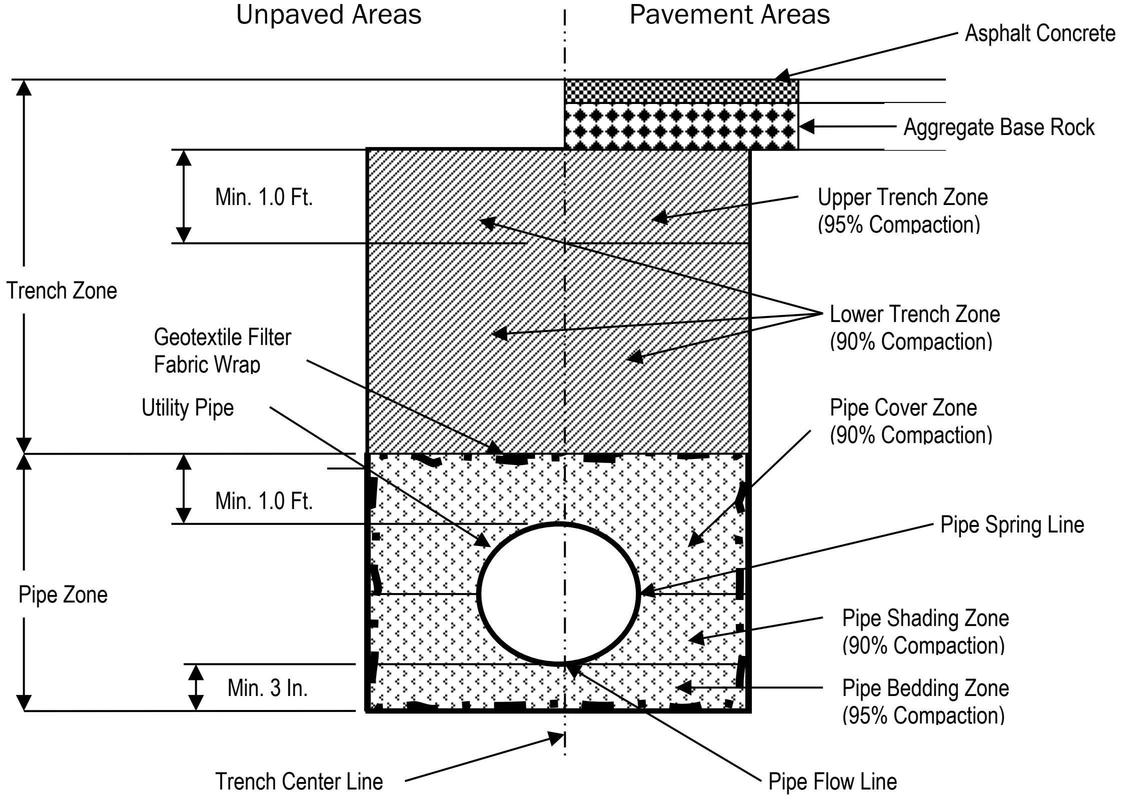
1. Erosion controls should be installed on all cut and fill slopes to minimize erosion caused by surface water runoff.
2. Install on all slopes either an appropriate hydroseed mixture compatible with the soil and climate conditions of the site, as determined by the local United States Soil Conservation District, or apply an appropriate manufactured erosion control mat.
3. Install surface water drainage ditches at the top of cut and fill slopes (as necessary) to collect and convey both sheet flow and concentrated flow away from the slope face.
4. The intercepted surface water should be discharged into a natural drainage course or into other collection and disposal structures.

8.1.9 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below for each trench zone shown in the figure below.

1. Trench Excavation Equipment: NV5 anticipates that the contractor will be able to excavate all underground utility trenches to depths of 3 feet bgs with a Case 580 Backhoe or equivalent.
2. Trench Shoring: All utility trenches that are excavated deeper than 4 feet bgs are required by California OSHA to be shored with bracing equipment or sloped back to an appropriate slope gradient prior to being entered by any individuals.
3. Trench Dewatering: NV5 does not anticipate that the proposed underground utility trenches will encounter shallow groundwater. However, if the utility trenches are excavated during the winter rainy season, then shallow or perched groundwater may be encountered. The earthwork contractor may need to employ de-watering methods as discussed in Section 8.1.11 in order to excavate, place and compact the trench backfill materials.
4. Pipe Zone Backfill Type and Compaction Requirements: The backfill material type and compaction requirements for the pipe zone, which includes the bedding zone, the shading zone and the cover zone, are described in Detail 8.1.9-1 below.

Detail 8.1.9-1 TYPICAL TRENCH BACKFILL ZONES



Not to Scale

- Pipe Zone Backfill Material Type: Trench backfill used within the pipe zone, which includes the bedding zone, the shading zone and the cover zone, should consist of ¾-inch-minus, washed, crushed rock. The crushed rock particle size gradation should meet the following requirements (percentages are expressed as dry weights using ASTM D422 test method): 100 percent passing the ¾-inch sieve, 80 to 100 percent passing the ½-inch sieve, 60 to 100 percent passing the 3/8-inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve. If groundwater is encountered within the trench during construction, or if groundwater is expected to rise during the rainy season to an elevation that will infiltrate the pipe zone within the trench, then the pipe zone material should be wrapped with a minimum 6 ounce per square yard, non-woven geotextile filter fabric such as TenCate® Mirifi N140 or an approved equivalent. The geotextile seam should be located along the trench centerline and have a minimum 1-foot overlap. If the utility pipes are coated with a corrosion protection material, then the pipes should be wrapped with a minimum 6 ounce per square yard, non-woven, geotextile cushion fabric such as TenCate® Mirifi N140 or an approved equivalent. The geotextile cushion fabric should have a minimum 6-inch seam overlap. The geotextile cushion fabric will protect the pipe from being scratched by the crushed rock backfill material.

- Pipe Bedding Zone Compaction: Trench backfill soil placed in the pipe bedding zone (beneath the utilities) should be a minimum of 3 inches thick, moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density. Crushed rock should be mechanically consolidated under the observation of NV5.
- Pipe Shading Zone Compaction: Trench backfill soil placed within the pipe shading zone (above the bedding zone and to a height of one pipe radius above the pipe spring line) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. Crushed rock should be mechanically consolidated under the observation of NV5. The pipe shading zone backfill material should be shovel-sliced to remove voids and to promote compaction.
- Pipe Cover Zone Compaction: Trench backfill soil placed within the pipe cover zone (above the pipe shading zone to 1 foot over the pipe top surface) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. Crushed rock should be mechanically consolidated under the observation of NV5.

5. Trench Zone Backfill and Compaction Requirements: The trench zone backfill materials consist of both lower and upper zones, as discussed below.
    - Trench Zone Backfill Material Type: Soil used as trench backfill within the lower and upper intermediate zones, as shown on the preceding figure, should consist of non-expansive soil with a PI of less than or equal to 15 (based on ASTM D4318) and should not contain rocks greater than 3 inches in greatest dimension.
    - Lower Trench Zone Compaction: Soil used to construct the lower trench zone backfills should be uniformly moisture conditioned to within 0 and 4 percentage points of the ASTM D1557 optimum moisture content, placed in maximum 12-inch-thick loose lifts prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
    - Upper Trench Zone Compaction (Road and Parking Lot Areas): Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 and 4 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 8-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
    - Upper Trench Zone Compaction (Non-Road and Non-Parking Lot Areas): Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 and 2 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 6-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
6. CQA Testing and Observation Engineering Services: The moisture content, dry density and relative percent compaction of all engineered utility trench backfills should be tested by the project engineer’s field representative during construction to evaluate whether the compacted trench backfill materials meet or exceed the minimum compaction and moisture content requirements presented in this report. The earthwork contractor shall assist the project engineer’s field representative by excavating test pads with the on-site earth moving equipment.

- Compaction Testing Frequencies: The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 8.1.9-1 or as modified by the project engineer to better suit the site conditions.

Table 8.1.9-1, Minimum Testing Frequencies for Utility Trench Backfill

ASTM No.	Test Description	Minimum Test Frequency⁽¹⁾
D1557	Modified Proctor Compaction Curve	1 per 500 CY (2) Or Material Change
D6983	Nuclear Moisture and Density	1 per 100 LF per 24-Inch-Thick Compacted Backfill Layer (2) The maximum loose lift thickness shall not exceed 12-inches prior to compacting.

Notes: (1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion on the basis of the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.
ASTM = ASTM International
CY = cubic yards
No. = number

- Final Proof Rolling: The prepared finished grade AB rock surface and/or finished subgrade soil surface of utility trench backfills should be proof-rolled, as mentioned above in Section 8.1.4, Paragraph 3.

8.1.10 Construction De-watering

NV5 does not anticipate the need to perform de-watering of the site during earthwork grading, however, the earthwork contractor should be prepared to de-water the utility trench excavations and any other excavations if perched water or the groundwater table is encountered during winter or spring grading. The following recommendations are preliminary and are not based on performing a groundwater flow analysis. A detailed de-watering analysis was not a part of the proposed work scope. It should be understood that it is the earthwork contractor's sole responsibility to select and employ a satisfactory de-watering method for each excavation.

1. NV5 anticipates that de-watering of utility trenches can be performed by constructing sumps to depths below the trench bottom and removing the water with sump pumps.
2. Additional sump excavations and pumps should be added as necessary to keep the excavation bottom free of standing water and relatively dry when placing and compacting the trench backfill materials.
3. If groundwater enters the trench faster than it can be removed by the de-watering system, thereby allowing the underlying compacted soil to become unstable while compacting successive soil lifts, then it may be necessary to remove the unstable soil and replace it with free-draining, granular drain rock. Native backfill soil can again be used after placing the granular rock to an elevation that is higher than the groundwater table.

4. If granular rock is used, it should be wrapped in a non-woven geotextile fabric, such as TenCate® Mirifi® N140 or an approved equivalent. The geotextile filter fabric should have minimum 1-foot overlapped seams. The granular rock should meet or exceed the following gradation specifications (all percentages are expressed as dry weights using ASTM D422 test method): 100 percent passing the 3/4-inch sieve, 80 to 100 percent passing the 1/2-inch sieve, 60 to 100 percent passing the 3/8-inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve.
5. NV5 recommends that the utility trench excavations be performed as late in the summer months as possible to allow the groundwater table to reach its lowest seasonal elevation.

8.1.11 Subsurface Groundwater Drainage

NV5 does not anticipate encountering perched groundwater or a shallow local groundwater table during the wet weather construction season. If groundwater is encountered during grading, then NV5 should be allowed to observe the conditions and provide site-specific de-watering recommendations.

8.1.12 Surface Water Drainage

NV5 recommends the following surface water drainage mitigation measures:

1. Grade all slopes to drain away from building areas with a minimum 4 percent slope for a distance of not less than 10 feet from the building foundations.
2. Grade all landscape areas near and adjacent to buildings to prevent ponding of water.
3. Direct all building downspouts to solid pipe collectors which discharge to natural drainage courses, storm sewers, catchment basins, infiltration subdrains or other drainage facilities.

8.1.13 Grading Plan Review and Construction Monitoring

CQA includes review of plans and specifications and performing construction monitoring, as described below.

1. NV5 should be allowed to review the final earthwork grading improvement plans prior to commencement of construction to determine whether the recommendations have been implemented and, if necessary, to provide additional and/or modified recommendations.
2. NV5 should be allowed to perform CQA monitoring of all earthwork grading performed by the contractor to determine whether the recommendations have been implemented and, if necessary, to provide additional and/or modified recommendations.
3. NV5's experience, and that of the engineering profession, clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining a design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering observation and CQA testing services. Upon your request we will prepare a CQA geotechnical engineering services proposal that will present a work scope, a tentative schedule and a fee estimate for your consideration and authorization. If NV5 is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then NV5 will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party's expectations in the future.

8.2 STRUCTURAL IMPROVEMENTS

NV5's structural improvement design criteria recommendations include: seismic design parameters, shallow continuous strip and isolated foundations for buildings, shallow rectangular or square foundations, and concrete slab-on-grade interior floors, and sidewalks. These recommendations are presented hereafter.

8.2.1 Seismic Design Parameters

NV5 developed the code-based seismic design parameters in accordance with Section 1613 of the 2016 CBC and the California Office of Statewide Health Planning and Development (OSHPD) seismic design maps, formerly facilitated by the USGS, *U.S. Seismic "DesignMaps" Web Application, Version 3.1.0.* The internet based application (https://seismicmaps.org/) is used for determining seismic design values from the 2016 ASCE-7 Standard, and the 2015 International Building Code (2015 IBC) in accordance with the 2016 CBC. The spectral acceleration, site class, site coefficients and adjusted maximum considered earthquake spectral response acceleration, and design spectral acceleration parameters are presented in Table 8.2.1-1. The Seismic Design Parameter detailed report from the OSHPD analysis is provided in Appendix D.

8.2.2 Seismic Design Category

Based on the short period response acceleration ground motion parameters above (S_{DS} = 0.454) and the Risk Category of III, the Seismic Design Category is C. Based on the 1-S period response acceleration ground motion parameters above (S_{D1} = 0.253) and the Risk Category of III, the Seismic Design Category is D. Therefore, the Seismic Design Category for the site is D.

Table 8.2.1-1 2016 CBC Seismic Design Parameters

Description	**Value**	**Reference**
Latitude North (degree)	39.2223	Google Earth
Longitude West (degree)	-121.0763	Google Earth
Site Coefficient, F_A	1.166	*2016 CBC, Table 1613A.3.3(1), OSHPD, ASCE 7-16*
Site Coefficient, F_V	1.557	*2016 CBC, Table 1613A.3.3(2), OSHPD, ASCE 7-16*
Site Class	C = Very Dense Soil and Soft Rock	*ASCE 7-10 Chapter 20, Table 20.3-1*
Short (0.2 sec) Spectral Response, S_S (g)	0.584	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Long (1.0 sec) Spectral Response, S_1 *(g)*	0.243	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Short (0.2 sec) MCE Spectral Response, S_{MS} *(g)*	0.681	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Long (1.0 sec) MCE Spectral Response, S_{M1} *(g)*	0.379	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Short (0.2 sec) Design Spectral Response, S_{DS} *(g)*	0.454	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Long (1.0 sec) Design Spectral Response, S_{D1} *(g)*	0.253	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Seismic Design Category (Risk Category I, II or II)	D	*ASCE 7-10, Section 11.4.3, OSHPD, ASCE 7-16*
Geometric Mean Peak Ground Acceleration (PGA_M) (g)	0.258	*ASCE 7-10, Section 11.8.3, OSHPD, ASCE 7-16*

deg = degrees
CBC = California Building Code
MCE = *Maximum Considered Earthquake*
sec = second
g = gravitational acceleration (9.81 meters per second^2 = 32.2 feet per second^2)
OSHPD = *California Office of Statewide Health Planning and Development seismic design maps.*
USGS = United States Geological Survey

8.2.3 Geometric Mean Peak Ground Acceleration

NV5 used the OSHPD *Seismic Design Maps Web Application* to determine the seismic design parameters for the site, including the geometric mean peak ground acceleration (PGA_M). The PGA_M is calculated by using the Site Coefficient ($F_{PGA} = F_a$) multiplied by the PGA mapped values found on Figure 22-7 from ASCE 7-10. The PGA_M was calculated using the following equation:

$$PGA_M = F_{PGA}PGA = 1.181 \text{ x } 0.219 = 0.258 \text{ g}$$

The Seismic Design Parameters detailed report from the OSHPD analysis is provided in Appendix D.

8.2.4 Shallow Foundations

Shallow continuous and isolated spread foundations that will support load bearing walls shall be designed as follows:

1. The base of all shallow foundations should bear on firm, competent non-expansive native soil, or non-expansive engineered fill compacted consistent with the earthwork recommendations of Section 8.1.
2. Continuous strip foundations should be constructed with the following dimensions:
    - Minimum Width = 12 Inches
    - Minimum Embedment Depth below the lowest adjacent exterior surface grade as shown in Table 8.2.4-1.
3. The bearing capacities to be used for structural design of shallow foundations embedded in either non-expansive native soil or non-expansive engineered fill are presented in Table 8.2.4-1.
    - The calculated factor of safety (FS) for allowable bearing pressures including live plus dead loads is 3.0 for all foundation embedment depths.
    - The allowable bearing pressure capacities were increased by a factor of 1.33 to include wind or seismic short-term loads.
    - The project structural engineer of record should review the factor of safety and confirm that it is not less than the over-strength factor for this structure.

Table 8.2.4-1, Foundation Bearing Pressures for Shallow Continuous Strip and Isolated Spread Foundations

Minimum Foundation Embedment Depth (in.)	**Maximum Ultimate Bearing Pressures For Live + Dead Loads (psf)**	**Maximum Allowable Bearing Pressures For Live + Dead Loads (psf)**	**Maximum Allowable Bearing Pressures For Live + Dead + Wind or Seismic Loads (psf)**	**Allowable Safety Factor (Ultimate/Total) (dim.)**
12	5,000	1,650	2,200	3.0
18	6,000	2,000	2,650	3.0
24	7,500	2,500	3,330	3.0

Notes:
psf = pounds per square foot
in. = inches
dim. = dimensionless

4. Foundation lateral resistance may be computed from passive pressure along the side of the foundation and sliding friction/cohesion resistance along the foundation base, however, the larger of the two resistance forces should be reduced by 50 percent when combining these two forces. The passive pressure can be assumed to be equal to an equivalent fluid pressure (EFP) per foot of depth. The passive pressure force and sliding friction coefficient for computing lateral resistance are as follows:

- Passive pressure = 300 (H), pounds per square foot (psf), where H = foundation embedment depth (feet) below lowest adjacent soil surface.
- Foundation bottom sliding friction coefficient = 0.4 (dimensionless).

5. Minimum steel reinforcement for continuous strip foundations should consist of two No. 4 bars with one bar placed near the top and one bar placed near the bottom of each foundation or as designated by a California licensed structural engineer.
6. The concrete should have a minimum 3,000 pounds per square inch compressive break strength after 28 days of curing, have a water-to-cement ratio from 0.40 to 0.50, and should be placed with minimum and maximum slumps of 4 and 6 inches, respectively. Since water is often added to uncured concrete to increase workability, it is important that strict quality control measures be employed during placement of the foundation concrete to ensure that the water-to-cement ratio is not altered prior to or during placement.
7. Concrete coverage over steel reinforcements should be a minimum of 3 inches as recommended by the American Concrete Institute (ACI).
8. Prior to placing concrete in any foundation excavations, the contractor shall remove all loose soil, rock, wood debris or other deleterious materials from the foundation excavations.
9. Foundation excavations should be saturated prior to placing concrete to aid the concrete curing process; however, concrete should not be placed in standing water.
10. Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loading. Based on the anticipated foundation dimensions and loads, we estimate that the total post-construction settlement of foundations designed and constructed in accordance with the recommendations will be on the order of 1/2 inch. Differential settlement between similarly loaded, adjacent foundations is expected to be about 1/4 inch, provided the foundations are founded into similar materials (e.g., all on competent and firm engineered fill, native soil or rock).
11. Prior to placing concrete in any foundation excavation, the project geotechnical engineer or his/her field representative should observe the excavations to document that the following requirements have been achieved: minimum foundation dimensions, minimum reinforcement steel placement and dimensions, removal of all loose soil, rock, wood debris or other deleterious materials, and that firm and competent native or engineered fill soil is exposed along the entire foundation excavation bottom. Strict adherence to these requirements is paramount to the satisfactory behavior of a building foundation. Minor deviations from these requirements can cause the foundations to undergo minor to severe amounts of settlement which can result in cracks developing in the foundation and adjacent structural members, such as concrete slab-on-grade floors.

8.2.5 Concrete Slab-On-Grade Interior, Sidewalk and Patio Construction

In general, NV5 recommends that subgrade elevations on which the concrete slab-on-grade floors are constructed be a minimum of 6 inches above the elevation of the surrounding parking lots, driveways and landscaped areas. Elevating the building will reduce the potential for subsurface water to enter beneath the concrete slab-on-grade floors and exterior surfaces and underground utility trenches.

The concrete slab-on-grade building floors, patios, sidewalks and driveway areas should be evaluated by a California-licensed civil engineer for expected live and dead loads to determine if the minimum slab thickness and steel reinforcement recommendations presented in this report should be increased or redesigned.

NV5 recommends using the guideline procedures, methods and material properties that are presented in the following ASTM and ACI documents for construction of concrete slab-on-grade floors:

- ACI 302.1R-04, Guide for Concrete Floor and Slab Construction, reported by ACI Committee 302.
- ASTM E1643-98 (Reapproved 2005), Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.
- ASTM E1745-97 (Reapproved 2004), Standard Specifications for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs.
- ASTM F710-5, Standard Practice for Preparing Concrete Floors to Receive Resilient Flooring.

The interior building concrete slab-on-grade floor and exterior shop, sidewalk and patio concrete slab-on-grade floor components are described below from top to bottom. If static or intermittent live floor loads greater than 250 psf are anticipated, then a California-licensed structural engineer should design the necessary concrete slab-on-grade floor thickness and steel reinforcements.

1. Minimum 4-Inch-Thick Concrete Slab: The concrete slab should be installed with a minimum 3,000 pounds per square inch (psi) compressive strength after 28 days of curing. NV5 recommends that the concrete design use a water-to-cement ratio between 0.40 and 0.45 and should be placed with minimum and maximum slumps of 3 and 5 inches, respectively. The concrete mix design is the responsibility of the concrete supplier.
2. Steel Reinforcement: Reinforcement should be used to improve the load-carrying capacity, to reduce cracking caused by shrinkage during curing and from both differential and repeated loadings. It should be understood that it is nearly impossible to prevent all cracks from development in concrete slabs; in other words, it should be expected that some cracking will occur in all concrete slabs no matter how well they are reinforced. Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California-licensed structural engineer.

   Rebar: As a minimum, use No. 3 rebar (ASTM A615/A 615M-04 Grade 60), tied and placed with 18-inch centers in both directions (perpendicular) and supported on concrete "dobies" to position the rebar in the center of the slab during concrete pouring. NV5 does not recommend that the steel reinforcements of the concrete slab-on-grade floor be tied into the perimeter or interior continuous strip foundations or interior isolated column foundations. In other words, we recommend that the concrete slab-on-grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures.
3. Underslab Vapor-Moisture Retarder Membrane: The underslab retarder membrane should be placed in areas with moisture sensitive floor coverings as a floor component that will minimize transmission of both liquid water and water vapor transmission through the concrete slab-on-grade floor. NV5 recommends using at a minimum a Class A (ASTM E1745-97 [Reapproved 2004]), minimum 10-mil-thick, plastic, vapor-moisture, retarder membrane material such as Stego Wrap® underslab vapor retarder membranes or equivalents. Additionally, the following materials are recommended: Stego® Tape and Stego® Mastic or equivalents to seal membrane joints and any utility penetrations.

Regardless of the type of moisture-vapor retarder membrane used, moisture can wick up through a concrete slab-on-grade floor. Excessive moisture transmission through a concrete slab floor can cause adhesion loss, warping and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor and both fungi and mold growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. Commercial sealants, polymer additives to the concrete at the batch plant, entrained air, flyash, and a reduced water-to-content ratio can be incorporated into the concrete slab-on-grade floor mix design to reduce its permeability and water-vapor transmissivity properties. A waterproofing consultant should be contacted to provide detailed recommendations if moisture sensitive flooring materials will be installed on the concrete slab-on-grade floors.

4. Minimum 4-Inch-Thick Crushed Rock or Class II Aggregate Base Rock Layer: Interior floors should be underlain by clean crushed rock, while exterior floors should use either crushed rock or Class II AB rock. Crushed rock should be mechanically consolidated under the observation of NV5. AB rock layers should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. The crushed rock should be washed to produce a particle size distribution of 100 percent (by dry weight) passing the ¾ inch sieve and 5 percent passing the No. 4 sieve and 0 to 3 percent passing the No. 200 sieve. An alternative rock material for external slab-on-grade concrete surfaces would include AB rock meeting the specification of Caltrans Class II AB. Just prior to pouring the concrete slab, the rock layer should be moistened to a saturated surface dry (SSD) condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks.

   If the current property owner elects to eliminate the crushed rock or AB rock layer beneath the interior and exterior concrete slabs-on-grade for economic reasons, then there will be an inherent greater risk assumed by the developer for the development of both shrinkage and bearing-related cracks in the associated slabs.

5. Subgrade Soil Preparation: The subgrade soil should be prepared and compacted consistent with the recommendations of Section 8.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 90 percent of the ASTM D1557 dry density with relatively uniform moisture content within ± 3 percentage points of the ASTM D1557 optimum moisture content.
6. Crack Control Grooves: Crack control grooves should be installed during placement or saw cuts should be made in accordance with the ACI and Portland Cement Association (PCA) specifications. Generally, NV5 recommends that expansion joints be provided between the slab and perimeter footings, and that crack control grooves or saw cuts are installed on 10-foot-centers in both directions (perpendicular).
7. Field Observations: Field observations should be made by an NV5 construction monitor of all concrete slab-on-grade surfaces and installed steel reinforcements prior to pouring concrete.

9.0 REFERENCES

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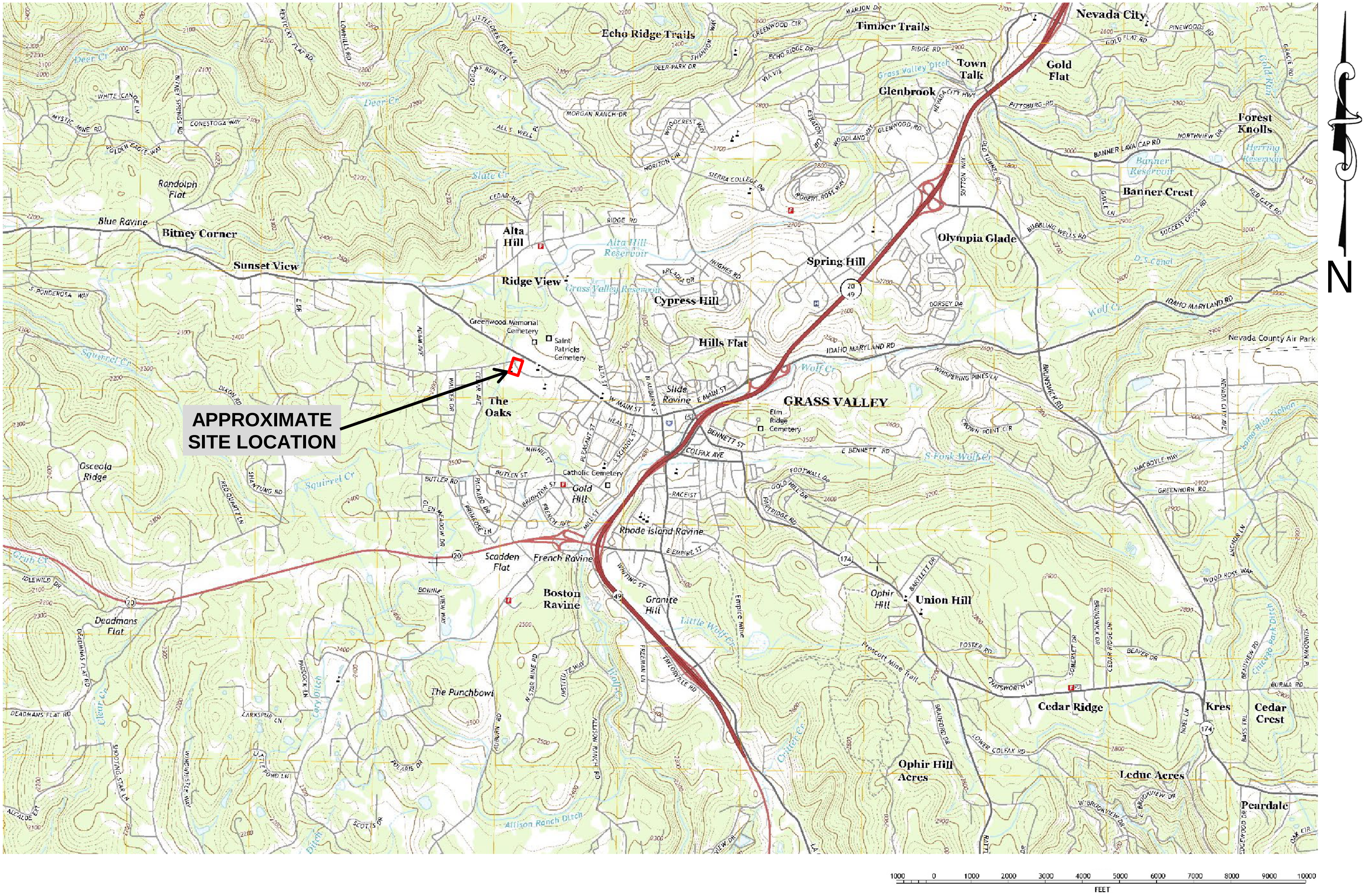
10.0 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. This report should not be relied upon without review by NV5 if a period of 24 months elapses between the issuance report date shown above and the date when construction commences.
2. NV5's professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in Northern California. No warranties are either expressed or implied.
3. NV5 provided engineering services for the site project consistent with the work scope and contract agreement presented in the proposal and agreed to by the client. The findings, conclusions and recommendations presented in this report apply to the conditions existing when NV5 performed the services and are intended only for the client, purposes, locations, timeframes and project parameters described herein. NV5 is not responsible for the impacts of any changes in environmental standards, practices or regulations subsequent to completing the services. NV5 does not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of the client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.
4. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. The validity of the conclusions and recommendations presented in this report can only be made by NV5; therefore, NV5 should be allowed to review all project changes and prepare written responses with regards to their impacts on the conclusions and recommendations. Additional fieldwork and laboratory testing may be required for NV5 to develop any modifications to the recommendations. The cost to review project changes and perform additional fieldwork and laboratory testing necessary to modify the recommendations is beyond the scope-of-services presented in this report. Any additional work will be performed only after receipt of an approved scope-of-work, budget and written authorization to proceed.
5. The analyses, conclusions and recommendations presented in this report are based on the site conditions as they existed at the time NV5 performed the surface and subsurface field investigations. NV5 has assumed that the subsurface soil and groundwater conditions encountered at the location of the exploratory borings are generally representative of the subsurface conditions throughout the entire project site; however, if the actual subsurface conditions encountered during construction are different than those described in this report, then NV5 should be notified immediately so that we can review these differences and, if necessary, modify the recommendations.
6. The elevation or depth to the groundwater table underlying the project site may differ with time and location; therefore, the depth to the groundwater table encountered in the exploratory borings is only representative of the specific time and location where it was observed.
7. The project site map shows approximate exploratory excavation locations as determined by pacing distances from identifiable site features; therefore, their locations should not be relied upon as being exact nor located with the accuracy of a California-licensed land surveyor.

8. NV5’s geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of hazardous materials. Although NV5 did not observe the presence of hazardous materials at the time of the field investigation, all project personnel should be careful and take the necessary precautions in the event hazardous materials are encountered during construction.
9. NV5’s geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of mold nor for the future potential development of mold at the project site. If an evaluation of the presence of mold and/or for the future potential development of mold at the site is desired, then the property owner should contact a consulting firm specializing in these types of investigations. NV5 does not perform mold evaluation investigations.
10. NV5’s experience and that of the civil engineering profession clearly indicates that during the construction phase of a project the risks of costly design, construction and maintenance problems can be significantly reduced by retaining a design geotechnical engineering firm to review the project plans and specifications and to provide geotechnical engineering CQA observation and testing services. Upon your request NV5 will prepare a CQA geotechnical engineering services proposal that will present a work scope, a tentative schedule and fee estimate for your consideration and authorization. If NV5 is not retained to provide geotechnical engineering CQA services during the construction phase of the project, then NV5 will not be responsible for geotechnical engineering CQA services provided by others nor any aspect of the project that fails to meet your or a third party’s expectations in the future.

FIGURES



APPROXIMATE SITE LOCATION

Base Map: Grass Valley Quadrangle, California-Nevada Co., 7.5 Minute Series

N|V|5

SITE LOCATION MAP

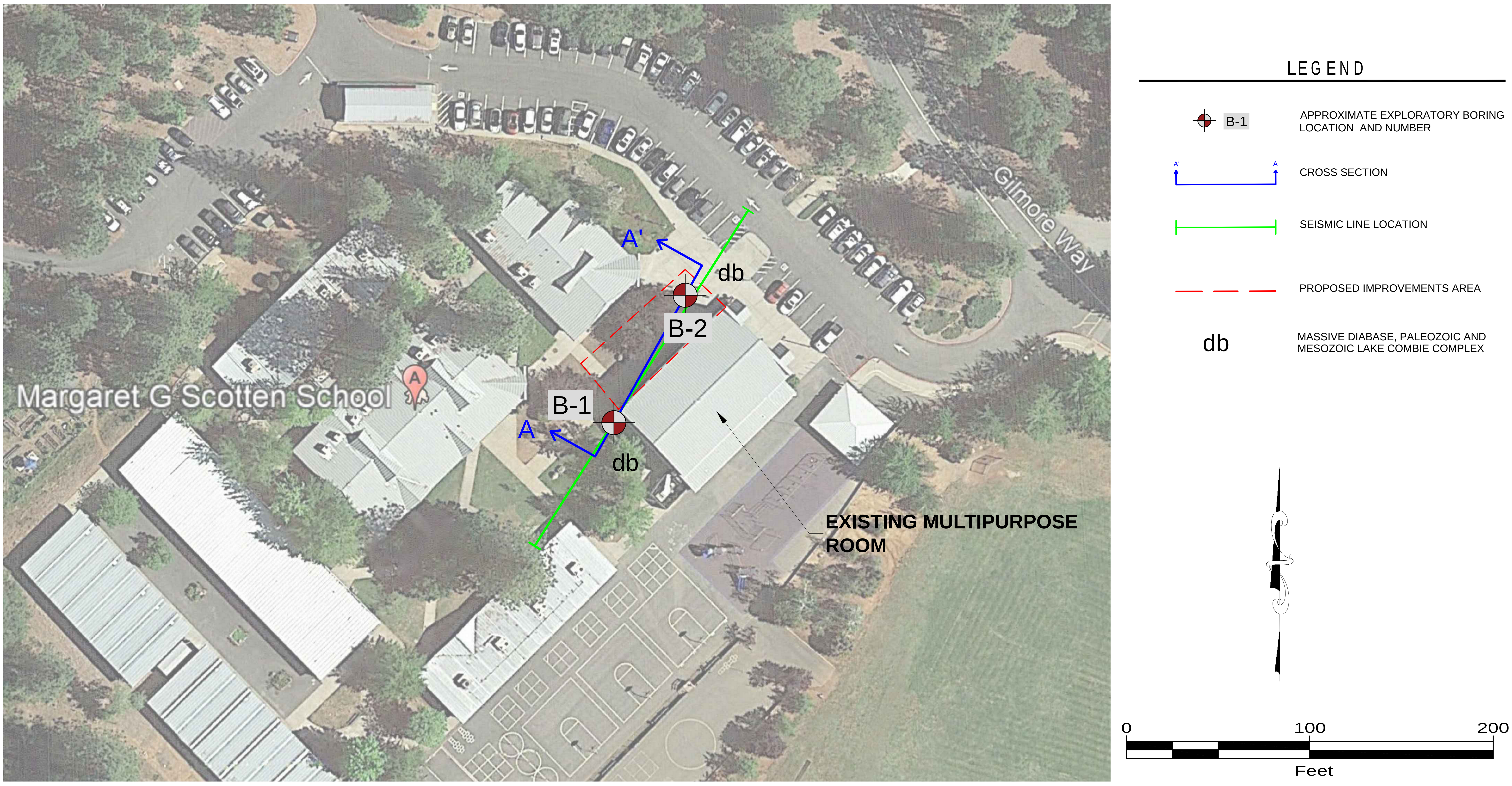
SCOTTEN SCHOOL IMPROVEMENTS

GRASS VALLEY, CALIFORNIA 95945

PROJECT NO. 5219.00

JANUARY 2019

FIGURE 1



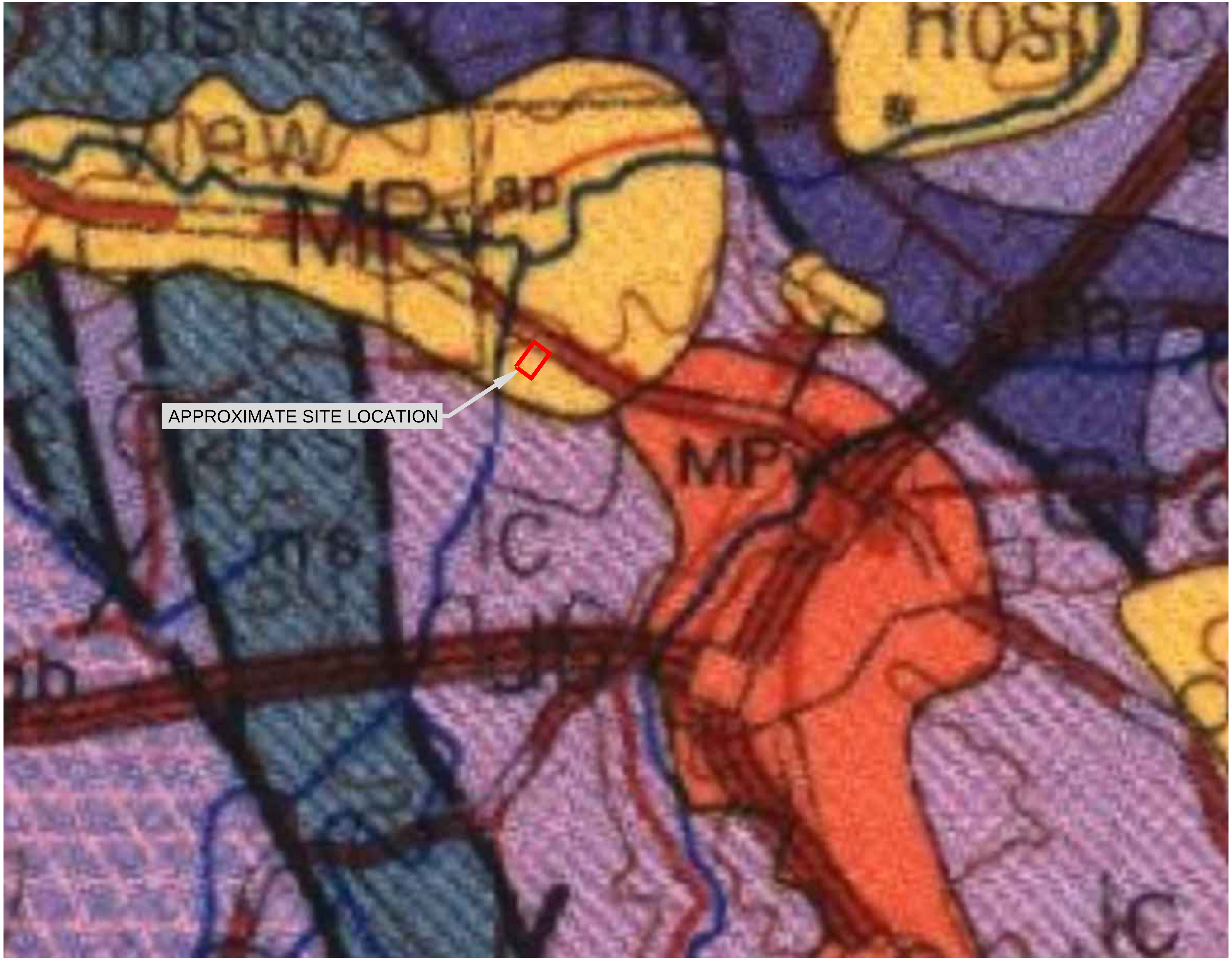
ASE MAP: GOOGLE EARTH 5/17/18, ACCESSED 1/11/19

SITE EXPLORATION & GEOLOGIC MAP

SCOTTEN SCHOOL IMPROVEMENTS
GRASS VALLEY, CLAIFORNIA

DRAWN BY: TMK
CHECKED BY: DAV
NV5 PROJECT: 5219.00
DATE: APRIL, 2019

FIGURE
2



LEGEND

MPv	**Miocene-Pliocene volcanic rocks** *(b-basalt; a-andesite; af-andesite flows; ap-andesite pyroclastic rocks; t-dacitic tuff-breccia)*

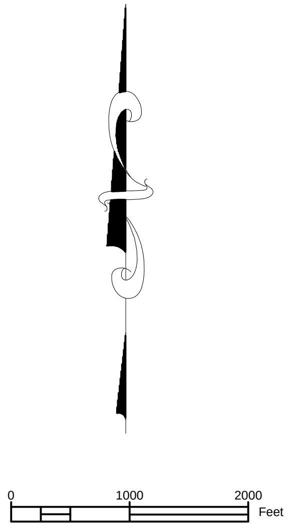
MESOZOIC PLUTONIC ROCKS

KJqd	QUARTZ DIORITE, TONALITE, TRONDHJEMITE, QUARTZ MONZONITE

PALEOZOIC AND
MESOZOIC ROCKS

lc- Lake Combie complex

gb	**Gabbroic rocks**
db	**Massive diabase**
um	**Ultramafic rocks**
qd	**Quartz diorite and tonalite**
ms	ms - **metasedimentary rocks**



Base Map: The Geology of the Chico Quadrangle by G.J. Saucedo and D.L. Wagner, 1992.

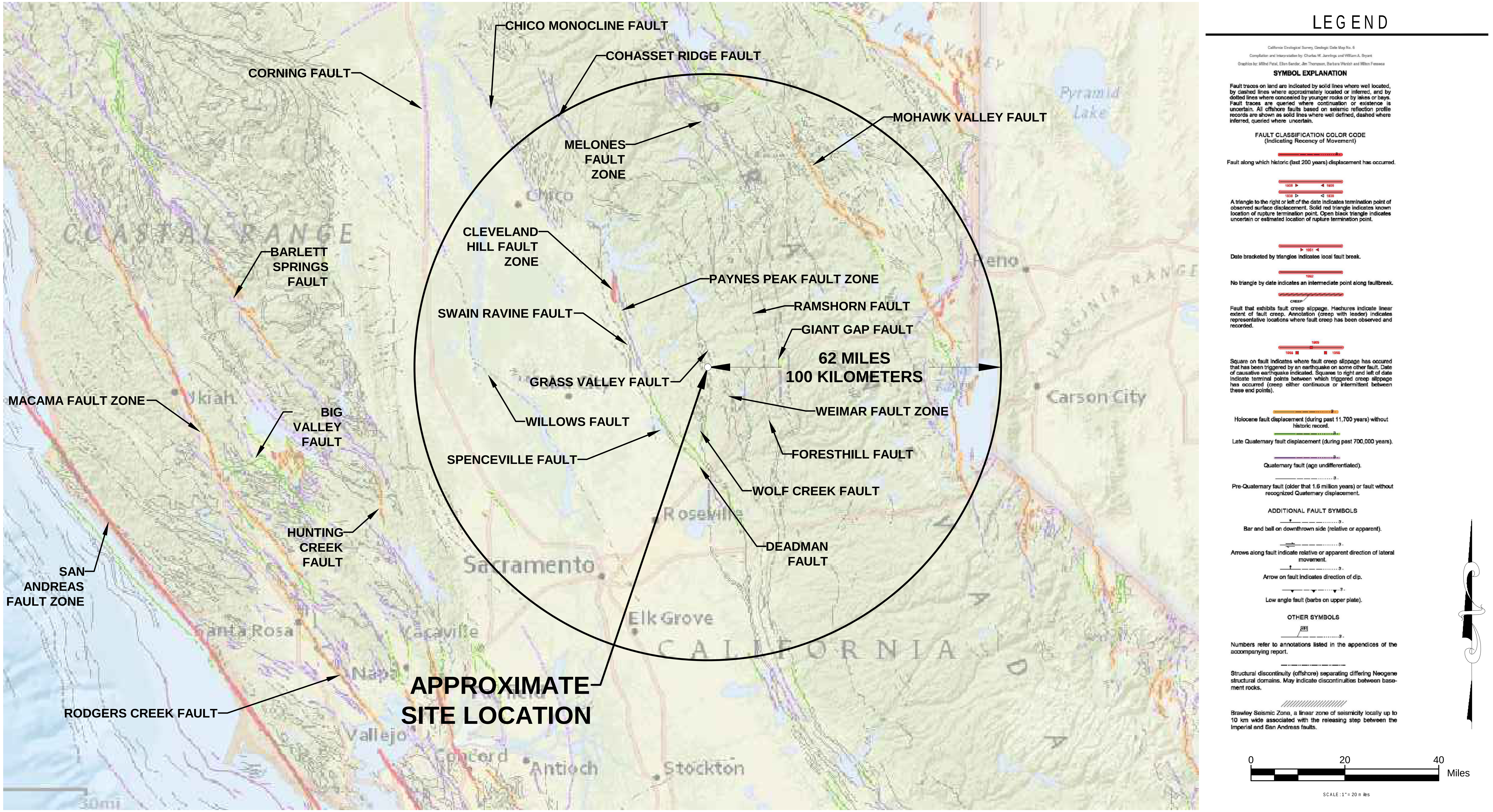
NV5

REGIONAL GEOLOGIC MAP

SCOTTEN SCHOOL IMPROVEMENTS
GRASS VALLEY, CLAIFORNIA

DRAWN BY: TMK
CHECKED BY: DAV
NV5 PROJECT: 5219.00
DATE: JANUARY, 2019

FIGURE
3



LEGEND

California Geological Survey, Geologic Data Map No. 6

Compilation and Interpretation by: Charles W. Jennings and William A. Bryant

Graphics by: Milind Patel, Ellen Sander, Jim Thompson, Barbara Wanish and Milton Fonseca

SYMBOL EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

?

Fault along which historic (last 200 years) displacement has occurred.

1906 ▶ ◀ 1906

1838 ▷ ◁ 1838

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

▶ 1951 ◀

Date bracketed by triangles indicates local fault break.

1992

No triangle by date indicates an intermediate point along faultbreak.

CREEP

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

1969

1966 ■ ■ 1968

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

?

Holocene fault displacement (during past 11,700 years) without historic record.

?

Late Quaternary fault displacement (during past 700,000 years).

?

Quaternary fault (age undifferentiated).

?

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

?

Bar and ball on downthrown side (relative or apparent).

?

Arrows along fault indicate relative or apparent direction of lateral movement.

?

Arrow on fault indicates direction of dip.

?

Low angle fault (barbs on upper plate).

OTHER SYMBOLS

491

?

Numbers refer to annotations listed in the appendices of the accompanying report.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

SCALE: 1" = 20 miles

Base Map: California Geological Survey 2010 Interactive Fault Activity Map.

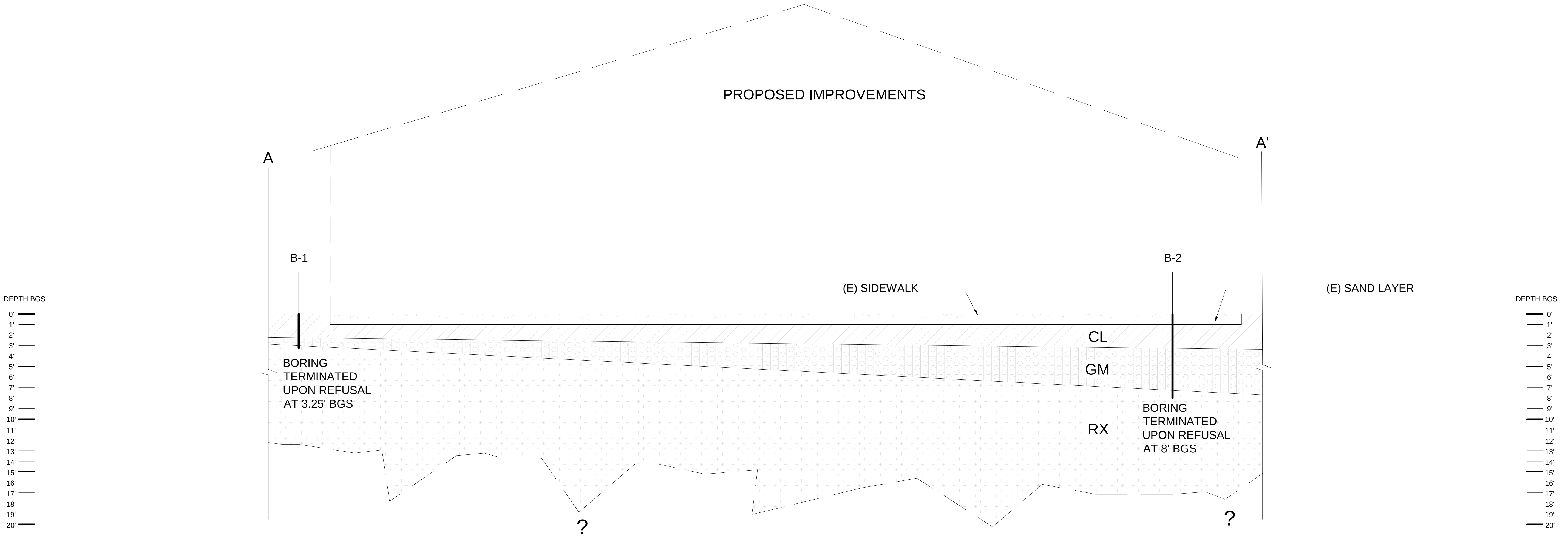
NV5

FAULT ACTIVITY MAP

SCOTTEN SCHOOL IMPROVEMENTS
GRASS VALLEY, CLAIFORNIA

DRAWN BY: TMK
CHECKED BY: DAV
NV5 PROJECT: 5219.00
DATE: JANUARY, 2019

FIGURE
4



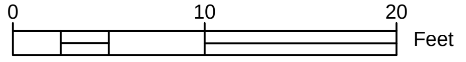
GENERALIZED GEOLOGIC/USCS UNITS

▨ (CL) SANDY CLAY

▦ (GM) SILTY GRAVEL WITH SAND

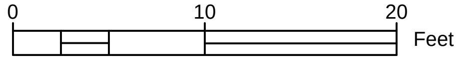
⬚ (RX) MASSIVE DIABASE

VERTICAL SCALE



SCALE: 1" = 10'

HORIZONTAL SCALE



SCALE: 1" = 10'

NV5

GEOLOGIC CROSS-SECTION

SCOTTEN SCHOOL IMPROVEMENTS
GRASS VALLEY, CLAIFORNIA

DRAWN BY:	TMK
CHECKED BY:	DAV
NV5 PROJECT:	5219.00
DATE:	JANUARY, 2019

FIGURE
5

APPENDIX A

Important Information about This Geotechnical Engineering Report
(Included with permission of GBA, Copyright 2016)

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B

Exploratory Boring Logs

EXPLORATORY BORING LOG

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

Boring No. B-1

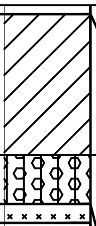
Project Name: GV SCHOOL DISTRICT IMPRVMNTS
Project No.: 5219.00
Task: -
Start: 12-27-18
Location: SCOTTEN SCHOOL
Ground Elev. (Ft. MSL): 2600'
Finish: 12-27-18
Sheet: 1 of 1
Logged By: DAV
Drilling Company: P.C. EXPLORATION
Drill Rig Type: GEFCO
Driller: SCOTT FLEMMING
Drilling Method: FLIGHT AUGER
Hammer Type: 140LB AUTO HAMMER
Boring Dia. (In.): 6
Total Depth (Ft.): 3.25
Backfill or Well Casing: BACKFILL WITH DRILL CUTTINGS

Ground Water Information

Date	NFWE			
Time	-			
Depth (ft)	-			

Soil and/or Rock Descriptions
(USCS Symbol; USCS Name; Field Estimated Particle Size Gradation (%); Munsel Color; Density/Consistency; Moisture; Fill Material; Dilatancy; Plasticity Toughness; Dry Strength; Structure; Cementation; Organics; Odor; Other)

Time (H:M)	Pocket Penetrometer (TSF)	SPT Blow Counts (Blows / Foot)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Well Construction Detail	Graphic Log	Soil and/or Rock Descriptions
07:40			SSA							GRASS LAWN SURFACE
			↓			1				(CL); SANDY CLAY; FIELD ESTIMATE: 70% CLAY, 30% FINE TO COARSE SAND; DARK GRAY (5YR 4/1); MEDIUM STIFF; WET; TRACE ORGANICS. [FILL].
07:50	0.75-1	7	MC	6"	B1-L1-1	2	■			
		11	↓	6"	B1-L1-2		■			(GM); SILTY GRAVEL WITH SAND; FIELD ESTIMATE: 30% SILT, 20% FINE TO COARSE SAND, 50% FINE TO COARSE GRAVEL; YELLOWISH RED (5YR 4/6); MEDIUM DENSE; MOIST.
		50/4.5"		6"	B1-L1-3	3	■			
08:05		50/1"	SPT	0						(RX); MASSIVE DIABASE; DARK BLUISH GRAY (GLEY 2 $\frac{4}{1}$) WITH WHITE INCLUSIONS; STRONGLY CEMENTED; SLIGHTLY WEATHERED; VERY STRONG; MASSIVE; DAMP
08:10		50/0.75"		0		4				
						5				BORING TERMINATED UPON DRILLING REFUSAL @ 3.25 FT BGS. NFWE
						6				
						7				
						8				
						9				
						10				
						11				
						12				
						13				
						14				
						15				
						16				
						17				
						18				
						19				
						20				



NOTES: BOH = BOTTOM OF HOLE
BGS = BELOW GROUND SURFACE
NFWE = NO FREE WATER ENCOUNTERED
MC = MODIFIED CALIFORNIA (2.5") SAMPLER
HA = HAND AUGER
SPT = STANDARD PENETRATION TEST
HSA = HOLLOW STEM AUGER

N|V|5

EXPLORATORY BORING LOG

792 Searls Avenue, Nevada City, California, 95959
PHONE: 530-478-1305, FAX: 530-478-1019

Boring No. B-2

Sheet: 1 **of** 1

Project Name: GV SCHOOL DISTRICT IMPRVMNTS
Project No.: 5219.00
Task: -
Start: 12-27-18
Location: SCOTTEN SCHOOL
Ground Elev. (Ft. MSL): 2600'
Finish: 12-27-18
Logged By: DAV
Drilling Company: P.C. EXPLORATION
Drill Rig Type: GEFCO
Driller: SCOTT FLEMMING
Drilling Method: FLIGHT AUGER
Hammer Type: 140LB AUTO HAMMER
Boring Dia. (In.): 6
Total Depth (Ft.): 8
Backfill or Well Casing: DRILL CUTTINGS, UPPER 2' CAPPED W/ GROUT

Ground Water Information

Date	NFWE			
Time	-			
Depth (ft)	-			

Time (H:M)	Pocket Penetrometer (TSF)	SPT Blow Counts (Blows / Foot)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Sample Interval And Symbol	Well Construction Detail	Graphic Log
09:00			CORE						
09:05		3	MC			1			
		7		14"					
		10			B2-L1-1	2			
09:10		9	MC	4"	B2-L2-1				
	2.25-2.75	11		6"	B2-L2-2	3			
		15		6"	B2-L2-3				
09:15		4	SPT			4			
		6		13"	B2-B1-1				
		6				5			
09:20		8	MC	5"	B2-L3-1				
		16		6"	B2-L3-2	6			
		24		6"	B2-L3-3				
09:25		22	MC	5"	B2-L4-1	7			
		50/5"		6"	B2-L4-2				
09:30		50/3"	SPT	6"	B2-B2-1	8			BOH
						9			
						10			
						11			
						12			
						13			
						14			
						15			
						16			
						17			
						18			
						19			
						20			

Soil and/or Rock Descriptions

(USCS Symbol; USCS Name; Field Estimated Particle Size Gradation (%); Munsel Color; Density/Consistency; Moisture; Fill Material; Dilatancy; Plasticity Toughness; Dry Strength; Structure; Cementation; Organics; Odor; Other)

5.5 INCH CONCRETE SIDEWALK

8 INCHES OF SAND (SP)

(CL); SANDY CLAY; FIELD ESTIMATE: 65% CLAY, 35% FINE TO COARSE SAND; YELLOWISH RED (5YR 4/6); STIFF; MOIST. [FILL]

(GM); SILTY GRAVEL WITH SAND; LAB DATA: 84% SAND AND GRAVEL, 16% SILT; YELLOWISH RED (5YR 4/6); LOOSE TO MEDIUM DENSE; MOIST.

(RX); MASSIVE DIABASE; DARK BLUISH GRAY (GLEY 2 4/1) WITH WHITE INCLUSIONS; STRONGLY CEMENTED; SLIGHTLY TO MODERATELY WEATHERED; VERY STRONG; MASSIVE; DAMP.

BORING TERMINATED UPON DRILLING REFUSAL @ 8.0 FT BGS. NFWE

NOTES: BOH = BOTTOM OF HOLE
BGS = BELOW GROUND SURFACE
NFWE = NO FREE WATER ENCOUNTERED
MC = MODIFIED CALIFORNIA (2.5") SAMPLER
HA = HAND AUGER
SPT = STANDARD PENETRATION TEST
HSA = HOLLOW STEM AUGER

APPENDIX C

Soil Laboratory Test Results

UNCONFINED COMPRESSION

ASTM D2166

DSA File #:

DSA Appl #:

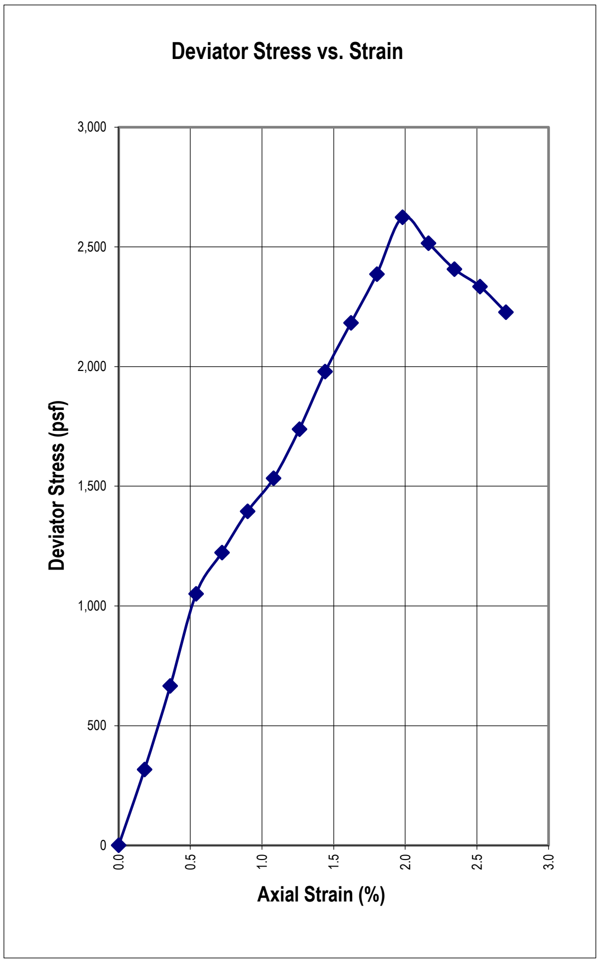
Project No.:	**5219.00**	Project Name:	**Scotten School**			Date:	**1/3/2019**
Sample No.:	**B2-L2-2**	Boring/Trench No.:	**B-2**	Depth (ft.)	**2.5**	Tested By:	**MLH**
Soil Description:	**Yellowish Red (5YR 4/6) Silty Gravel with Sand**					Check By:	**MLH**
Sample Location:						Lab No.:	**15-19-001**

Sample Data		
Tare Tube Number	I.D.	H
Tare Weight	(gm)	330.60
Wet Soil + Tare	(gm)	1052.70
Dry Soil + Tare	(gm)	853.60
Weight of Water	(gm)	199.10
Dry Soil Weight	(gm)	523.00
Moisture Content	(%)	38.07
Soil Height	(cm)	14.10
Sample Diameter	(cm)	6.10
Wet Unit Weight	(pcf)	109.41
Dry Unit Weight	(pcf)	79.24
Specific Gravity	(dim)	2.70
Saturation	(%)	91.27
Strain Rate	(%)	1.17
Proving Ring Constant	(lbs/unit)	1.108

Sample Sketch At Failure

Unconfined Shear Strength = 1,311.9 psf

Elapsed Time (Minutes)	Strain Units (0.001in/unit)	Strain Percent (%)	Area (cm^2)	Load Dial (units)	Load Force (lbs)	Deviator Stress (psf)
12:00:00	0	0.00	0.00	0	0.00	0.00
12:00:10	10	0.18	29.28	9	9.97	316.43
12:00:20	20	0.36	29.33	19	21.05	666.82
12:00:30	30	0.54	29.38	30	33.24	1050.96
12:00:40	40	0.72	29.44	35	38.78	1223.90
12:00:50	50	0.90	29.49	40	44.32	1396.21
12:01:00	60	1.08	29.54	44	48.75	1533.04
12:01:10	70	1.26	29.60	50	55.40	1738.92
12:01:20	80	1.44	29.65	57	63.16	1978.75
12:01:30	90	1.62	29.71	63	69.80	2183.04
12:01:40	100	1.80	29.76	69	76.45	2386.57
12:01:50	110	1.98	29.82	76	84.21	2623.87
12:02:00	120	2.16	29.87	73	80.88	2515.66
12:02:10	130	2.34	29.93	70	77.56	2407.83
12:02:20	140	2.52	29.98	68	75.34	2334.72493
12:02:30	150	2.70	30.04	65	72.02	2227.59808



PARTICLE SIZE DISTRIBUTION

ASTM D1140

DSA File #:

DSA Appl #:

Project No.:	**5219.00**	Project Name:	**Scotten School**			Date:	**1/3/2019**
Sample No.:	**B2-L3-2**	Boring/Trench:	**B-2**	Depth, (ft.):	**5.5**	Tested By:	**MLH**
Description:	**Yellowish Red (5YR 4/6) Silty Gravel with Sand**					Checked By:	**MLH**
Sample Location:						Lab. No.:	**15-19-001**

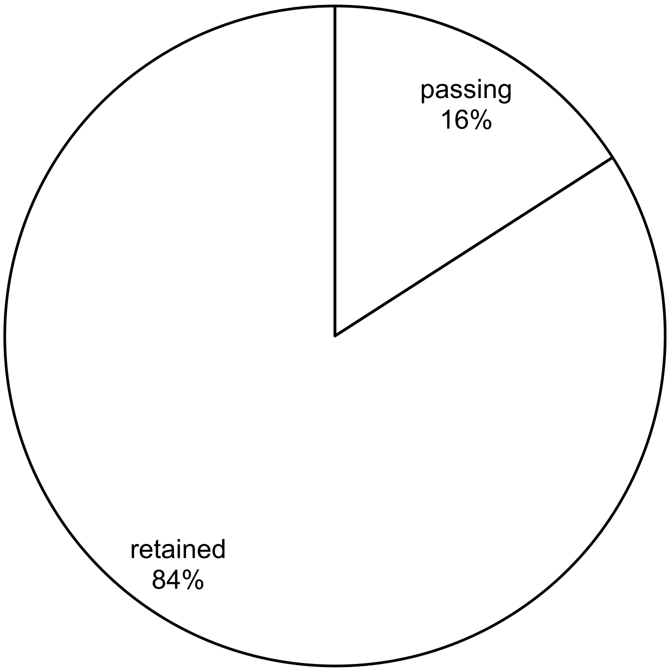
Moisture Content Data:

Pan ID	MC	
Pan Weight	50.69	(gm)
Wet Soil + Pan	92.84	(gm)
Dry Soil + Pan	90.44	(gm)
Water Weight	2.40	(gm)
Dry Soil Weight	39.75	(gm)
Moisture Content	6.0	(%)

Total Material Sample Data:

Pan ID	CTP	
Pan Weight	545.63	(gm)
Wet Soil + Pan Wt.	1,056.90	(gm)
Wet Weight	511.27	(gm)
Dry Weight	482.16	(gm)
Dry Wt.> #200 Sieve & Pan	951.12	(gm)
Dry Wt.> #200 Sieve	405.49	
Total Percent <#200 Sieve	15.90	(%)

Percent Passing/Retained
 # 200 Sieve



MOISTURE & DENSITY

ASTM D2216 & D2937

DSA File #:
DSA Appl #:

Project No.: **5219.00**
Project Name: **Scotten School**
Date: **1/3/2019**
Lab No.: **15-19-001**
Performed By: **MLH/NGH**
Checked By: **MLH**

SAMPLE LOCATION DATA

Boring/Trench No.	Units	B-1	B-2	B-2						
Sample No.		B1-L1-3	B2-L2-2	B2-L3-2						
Depth Interval	(ft.)	2.5	2.5	5.5						
Sample Description		Yellowish Red (5YR 4/6) Silty Gravel with Sand	Yellowish Red (5YR 4/6) Sandy Clay	Yellowish Red (5YR 4/6) Silty Gravel with Sand						
USCS Symbol										

SAMPLE DIMENSION AND WEIGHT DATA

Sample Length	(in)	5.550	5.550	6.180						
Sample Diameter	(in)	2.380	2.400	2.360						
Sample Volume	(cf)	0.0143	0.0145	0.0156						
Wet Soil + Tube Wt.	(gr)	1011.80	723.18	1050.40						
Tube Wt.	(gr)	280.26	0.00	275.90						
Wet Soil Wt.	(gr)	731.54	723.18	774.50						

MOISTURE CONTENT DATA

Tare No.		IJ	H	EJ						
Tare Wt.	(gr)	279.50	330.60	190.60						
Wet Soil + Tare Wt.	(gr)	947.30	1052.70	304.30						
Dry Soil + Tare Wt.	(gr)	780.70	853.60	266.59						
Water Wt.	(gr)	166.60	199.10	37.71						
Dry Soil Wt.	(gr)	501.20	523.00	75.99						
Moisture Content	(%)	33.2	38.1	49.6						

TEST RESULTS

Wet Unit Wt.	(pcf)	112.9	109.7	109.1						
Moisture Content	(%)	33.2	38.1	49.6						
Dry Unit Wt.	(pcf)	84.7	79.5	72.9						

MOISTURE CORRECTION DATA

Gauge Moisture	(%)									
K Value Correction Factor										

COMPACTION CURVE DATA (ASTM D698, ASTM D1557, or CAL216)

Test Method										
Curve No.										
Max Wet Unit Wt.	(pcf)									
Max Dry Unit Wt.	(pcf)									
Optimum Moisture	(%)									
Wet Relative Comp.	(%)									
Dry Relative Comp.	(%)									

NV5

ATTERBERG INDICES

ASTM D4318

DSA File #:

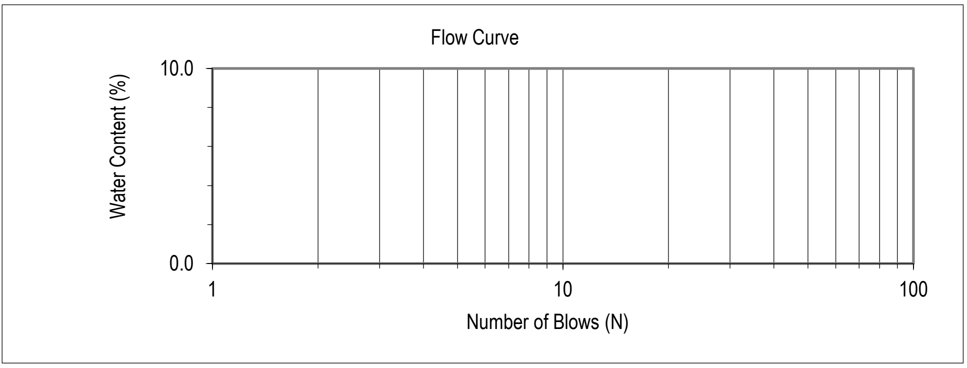
DSA Appl #:

Project No.: **5219.00** Project Name: **Scotten School** Date: **1/3/2019**
Sample No.: **B2-L3-2** Boring/Trench: **B-2** Depth, (ft.): **5.5** Tested By: **MLH**
Description: **Yellowish Red (5YR 4/6) Silty Gravel with Sand** Checked By: **MLH**
Sample Location: Lab. No.: **15-19-001**

Estimated % of Sample Retained on No. 40 Sieve: 75 Sample Air Dried: yes
Test Method A or B: A

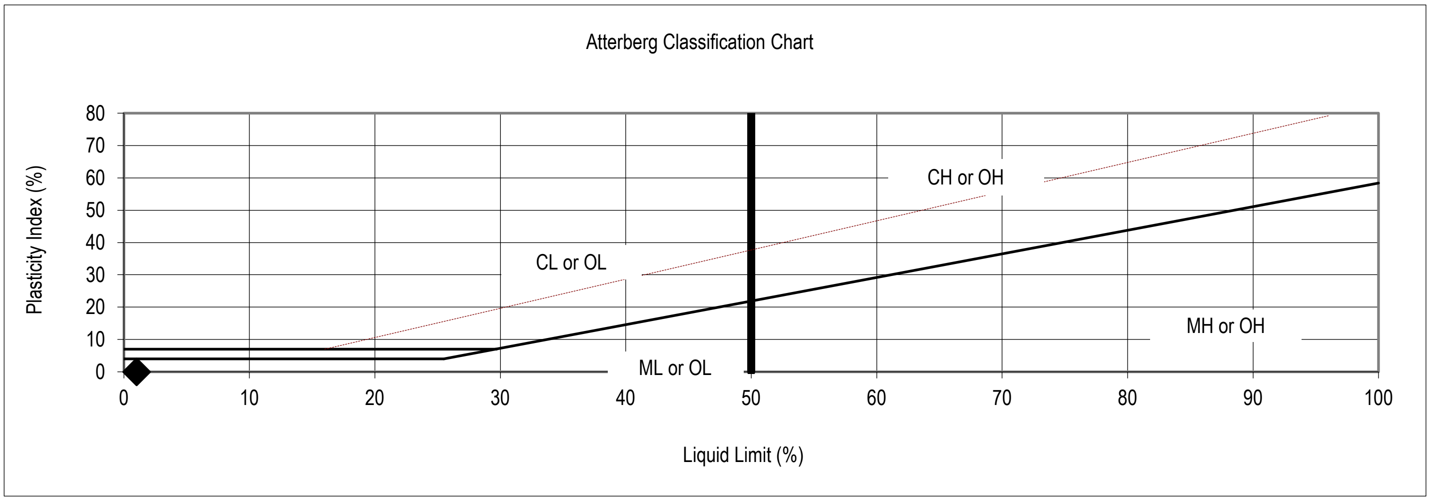
	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	LL	LC	LV			LK	LG	
Wt. Pan (gr)	10.56	11.25	10.99			11.03	10.84	
Wt. Wet Soil + Pan (gr)								
Wt. Dry Soil + Pan (gr)								
Wt. Water (gr)	0.00	0.00	0.00			0.00	0.00	
Wt. Dry Soil (gr)	-10.56	-11.25	-10.99			-11.03	-10.84	
Water Content (%)	0.0	0.0	0.0			0.0	0.0	
Number of Blows, N								

LIQUID LIMIT = NP PLASTIC LIMIT = NP



Plasticity Index = NP

Group Symbol = ML



APPENDIX D

Seismic Design Parameters

Scotten School

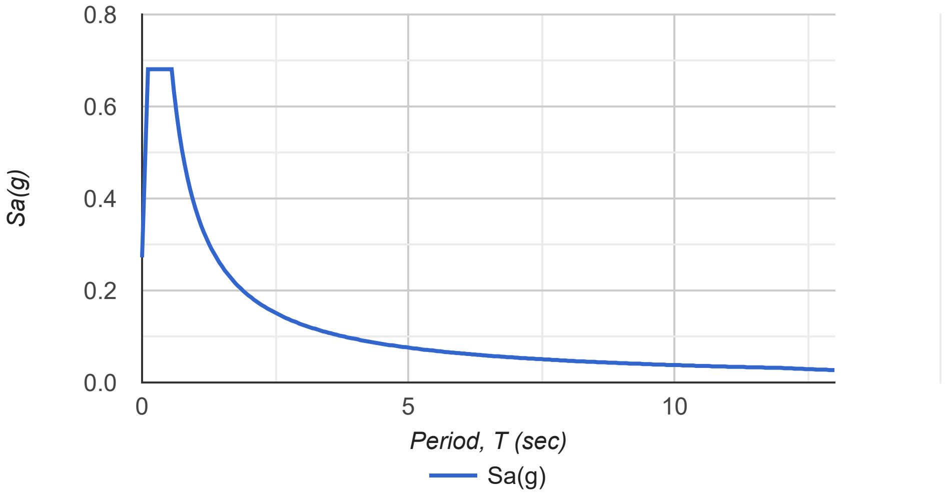
Latitude, Longitude: 39.2223, -121.0763

Date	4/5/2019, 11:30:27 AM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

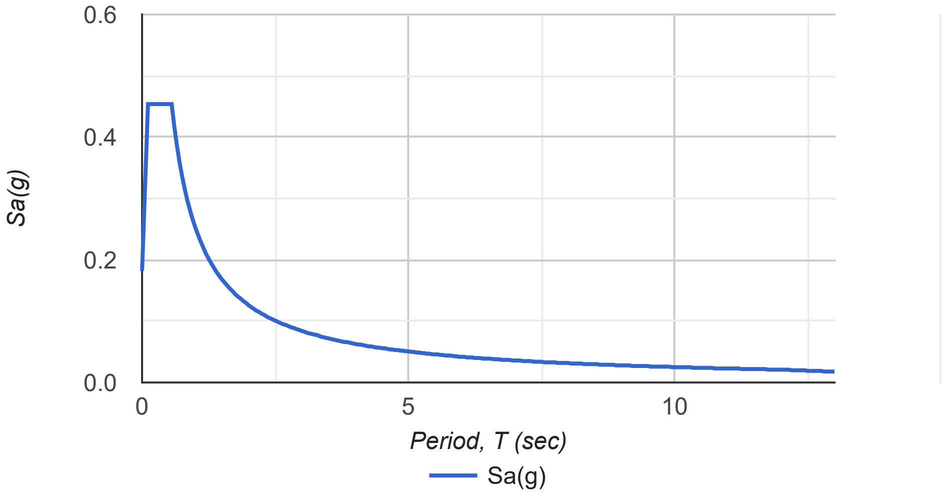
Type	Value	Description
S_S	0.584	MCE_R ground motion. (for 0.2 second period)
S_1	0.243	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.681	Site-modified spectral acceleration value
S_{M1}	0.379	Site-modified spectral acceleration value
S_{DS}	0.454	Numeric seismic design value at 0.2 second SA
S_{D1}	0.253	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.166	Site amplification factor at 0.2 second
F_v	1.557	Site amplification factor at 1.0 second
PGA	0.219	MCE_G peak ground acceleration
F_{PGA}	1.181	Site amplification factor at PGA
PGA_M	0.258	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	0.584	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.571	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.243	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.227	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)
C_{RS}	1.022	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	1.073	Mapped value of the risk coefficient at a period of 1 s



MCER Response Spectrum



Design Response Spectrum

APPENDIX E

Soil Corrosion Potential Test Results

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 01/11/2019
Date Submitted 01/08/2019

To: Michelle Holub
Holdrege & Kull
792 Searls Ave.
Nevada City, CA 95959

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 5219.000 Site ID : B2-L2-3@3.0 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 78780-164732.

EVALUATION FOR SOIL CORROSION

Soil pH	5.64	
Moisture	28.8 %	
Minimum Resistivity	5.90 ohm-cm (x1000)	
Chloride	37.3 ppm	00.00373 %
Sulfate	0.4 ppm	00.00004 %
Redox Potential	(+) 273 mv	
Sulfides	Presence - NEGATIVE	

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m
Redox Potential ASTM G-200m, Sulfides AWWA C105/A25.5