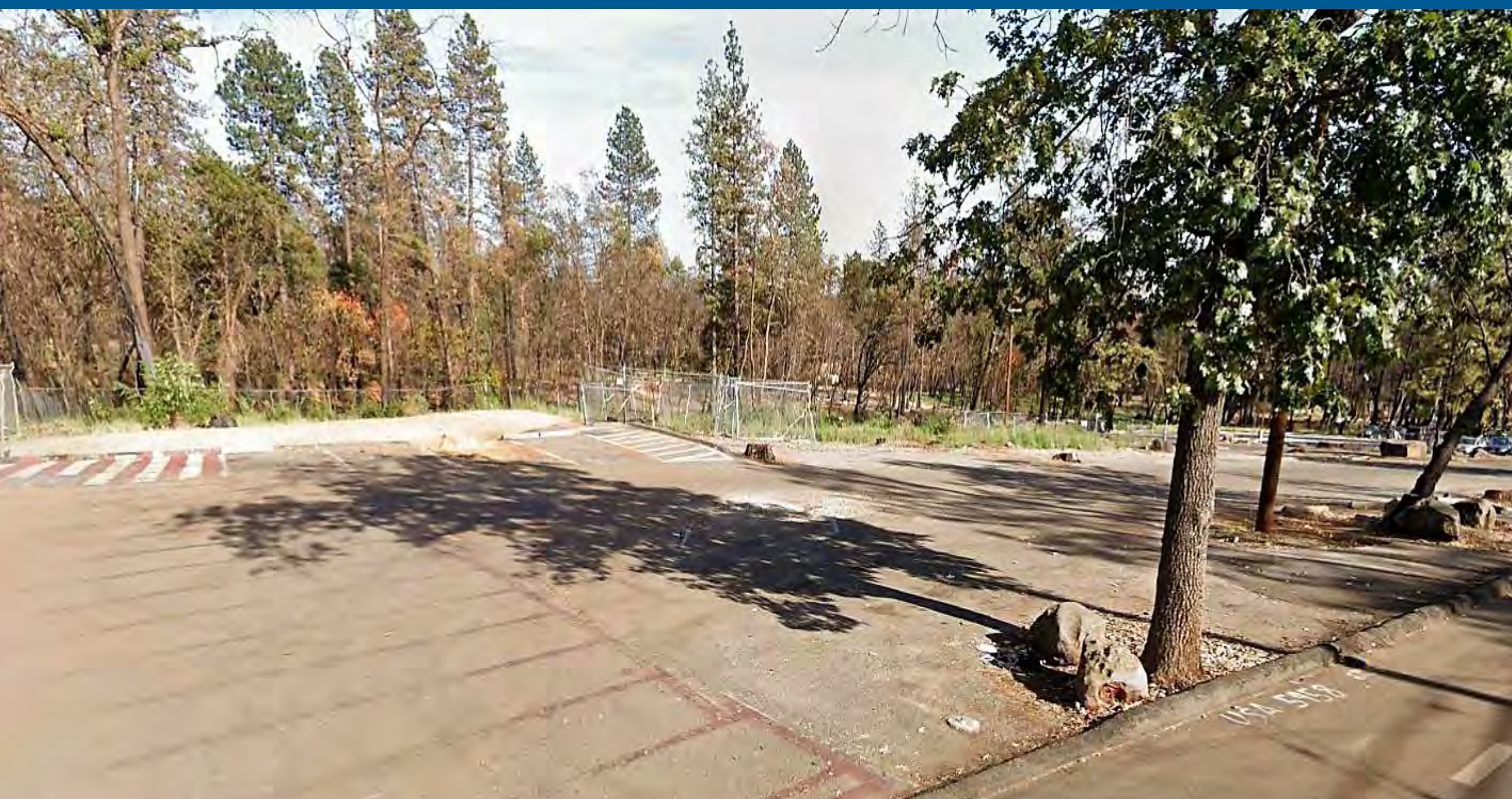
GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS REPORT
PROPOSED RIDGEVIEW HIGH SCHOOL
MAXWELL DRIVE
PARADISE, CALIFORNIA

AUGUST 17, 2020

Prepared For:

PARADISE UNIFIED SCHOOL DISTRICT
6696 Clark Road
Paradise, California 95969

Mr. David McCready, Assistant Superintendent, Business Services



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125620-0070994.00.001

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August 17, 2020
Project No. 70994.00.001

Mr. David McCready
Paradise Unified School District
6696 Clark Road
Paradise, California 95969

Via PDF: dmccready@pusdk12.org

Reference: **Geotechnical Engineering and Geologic Hazards Report**
Proposed Ridgeview High School
Maxwell Drive
Paradise, Butte County, California

Dear Mr. McCready,

NV5 performed a geotechnical engineering investigation and conducted a geologic hazards evaluation for the proposed Ridgeview High School at the northeast corner of Maxwell Drive and Pleasant Lane in Paradise, California. NV5's geotechnical engineering investigation and geologic hazards evaluation of the site was performed consistent with the scope of services presented in the May 27, 2020 proposal (PC20.109).

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 single-story continuation high school structure utilizing conventional design and construction practices. There were no geologic, seismic or geotechnical engineering hazards identified at the site that would require mitigation during construction. 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 530-894-2487.

Sincerely,
NV5

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A Important Information about This Geotechnical Engineering Report (Included with permission of GBA, Copyright 2019)
B Site Data Report
C Exploratory Boring Logs
D Soil Laboratory Test Results
E Seismic Design Parameters

ACRONYMS

AB	aggregate base
AC	asphalt concrete
ACI	American Concrete Institute
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
CQA	Construction Quality Assurance
DTSC	Department of Toxic Substances Control
FEMA	Federal Emergency Management Agency
FS	factor of safety
ft/s	feet per second
GAMA	Groundwater Ambient Monitoring and Assessment Program
GBA	Geoprofessional Business Association
km	kilometer
M_L	local magnitude
MHA	maximum horizontal acceleration
MCE	maximum considered earthquake
mg/kg	milligrams per kilograms
msl	mean sea level
mybp	million years before present
NEIC	National Earthquake Information Center
NOA	naturally occurring asbestos
OSHA	Occupational Safety and Hazards Administration
oz/sy	ounce per square yard
P-waves	seismic compression waves
PCA	Portland Cement Association
pcf	pounds per cubic foot
pCi/L	picoCuries per liter
PGA_M	peak ground acceleration
PI	plasticity index
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinylchloride
RHS	Ridgeview High School
S-wave	shear-wave
SEAOC	Structural Engineers Association of California
SFHA	Special Flood Hazard Area
SPT	standard penetration test
SRMS	Seismic Refraction Microtremor Survey
USCS	Unified Soils Classification System
USGS	United States Geological Survey
V_s Model	velocity profile

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 Ridgeview High School (RHS) at the northeast corner of Maxwell Drive and Pleasant Lane in Paradise, California, consistent with the scope of services presented in NV5's *Proposal for Geologic and Geotechnical Engineering Services* (PC20.109), dated May 27, 2020. The scope of services was based on the 2019 California Building Code (CBC) and current *Checklist for Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings* (Note 48) available at the time the proposal was prepared. 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 evaluate potential geologic hazards located within the site and its immediate vicinity and 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, 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 evaluate potential geologic hazards that may negatively impact the proposed site improvements and 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. NV5 reviewed geologic and seismic literature, maps, aerial photos, and on-line sources for information about site soil and rock conditions, and potential geologic and seismic hazards. A probabilistic seismic hazard analysis was performed in accordance with American Society of Civil Engineers (ASCE) 7-16 to estimate potential ground motion at the site.
- **Task 3 Report Preparation:** NV5 prepared this report to present the findings, conclusions and recommendations for this geologic hazards evaluation and geotechnical engineering investigation. The report followed the guidelines presented in California Geological Survey (CGS) Note 48, *Checklist for Review of Engineering Geology and Seismology Reports for California*

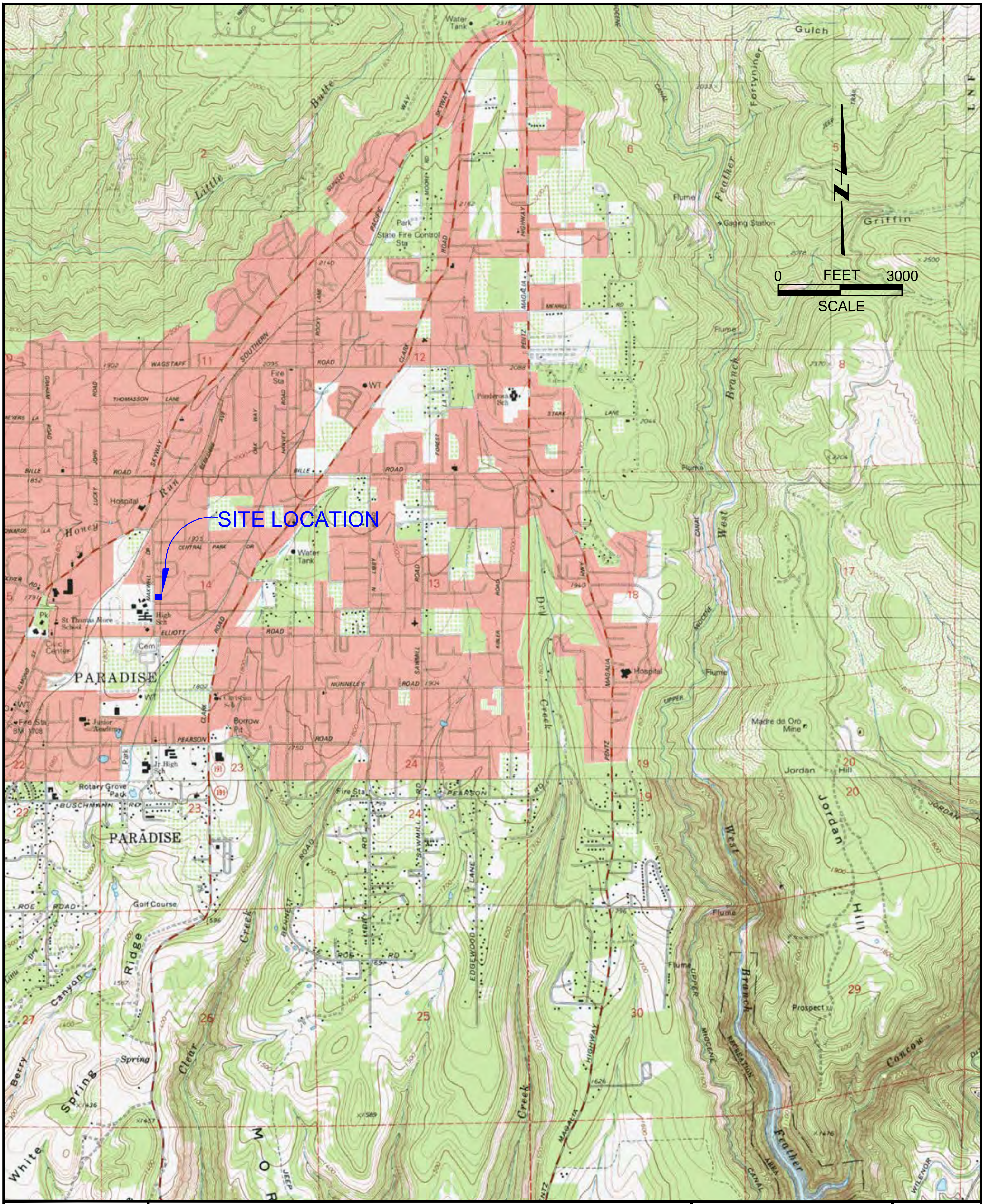
Public Schools, Hospitals, and Essential Services Buildings, dated November 2019, and the 2019 CBC.

1.2 SITE LOCATION AND DESCRIPTION

The proposed RHS project is located at the northeastern corner of Maxwell Drive and Pleasant Lane in Paradise, California. The RHS site is centered at about latitude 39.7622 north and longitude -121.6121 west on the United States Geological Survey's (USGS), 7.5 minute Paradise East Quadrangle topographic map. The property elevation is approximately 1880 feet above mean sea level (msl), based on review of the USGS 7.5-minute Paradise East Quadrangle 2018 topographic map. Figure 1 shows the site location and vicinity.

At the time the site investigation was performed on June 30, 2020, the following conditions were observed and are shown in the inset image above:

- The area of the proposed RHS currently supports asphalt concrete (AC) pavements in a major portion of the site. The northeastern portion of the site is undeveloped land supporting volunteer weeds, grasses, and brush. Several tree stumps were observed in the eastern portion of the site from trees removed following the 2018 Camp Fire.



N V 5	SITE LOCATION PARADISE USD - RIDGEVIEW HIGH SCHOOL PARADISE, BUTTE COUNTY, CALIFORNIA	DRAWN BY:	SC	FIGURE 1
		CHECKED BY:	DJP	
		PROJECT:	70994.00	
		DATE:	AUGUST 2020	

1.3 PROPOSED IMPROVEMENTS

Based on preliminary project information provided by representatives of BCA Architects, the Architect of Record, and review of the undated *Facilities Master Plan Priorities – Paradise/Ridgeview HS* prepared by BCA Architects, NV5 understands the proposed improvements include construction of a new RHS continuation high school. Appendix B presents the Site Data Report, dated August 5, 2020, prepared by BCA Architects in accordance with 2019 CBC, Section 1603A.2. Based on the Site Data Report information, NV5 anticipates the new RHS continuation high school building will consist of one tall, single-story structure approximately 12,514 square feet in size using wood or light metal framing supported on shallow perimeter and isolated spread foundations and concrete slab-on-grade floor. Associated development will likely include construction of underground utilities, concrete slab-on-grade sidewalks and landscape improvements. Earthwork grading will involve minor cuts and fills to meet the proposed building grade. Figure 2 shows the proposed building location and the approximate exploratory boring locations.

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.

Notes: Adapted from the Revised Master Plan, Facilities Master Plan Priorities - Paradise/Ridgeview HS, prepared by BCA Architects.

NV5

SITE PLAN AND EXPLORATORY BORING LOCATIONS MAP

PARADISE USD - RIDGEVIEW HIGH SCHOOL

PARADISE, BUTTE COUNTY, CALIFORNIA

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FIGURE 2

2.0 SITE INVESTIGATION

NV5 performed a site investigation to characterize the existing surface and subsurface conditions beneath the proposed RHS and associated improvements. 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 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

NV5 has prepared multiple geotechnical engineering reports and geologic hazard evaluation reports for the neighboring Paradise High School. As such, NV5 reviewed the following specific reports associated with the surrounding site area. The following identifies each report and summarizes the findings, conclusions and recommendations presented in each report:

- Holdrege & Kull (H&K), 2012, Geologic Hazards Evaluation and Geotechnical Engineering Investigation Report for the Paradise High School, New Gymnasium Building, 5911 Maxwell Drive, Paradise, California, Project Number 70400-01, May 25.

NV5's review of this document revealed the surface and near-surface soil conditions generally consist of stiff to very stiff sandy clay underlain by slightly weathered, Olivine Basalt volcanic rock. This geotechnical and geologic hazard report was prepared for a proposed gymnasium to be built at the location of the existing tennis courts along Maxwell Drive across from the RHS site.

- H&K, 2017, Geologic Hazards Evaluation and Geotechnical Engineering Investigation Report for the Paradise High School, New Gymnasium Building, 5911 Maxwell Drive, Paradise, California, Project Number 70400-05, January 18.

NV5's review of this document revealed the surface and near-surface soil conditions generally consist of competent rock encountered at shallow depths consisting of Olivine Basalt of Paradise and also weathered Tuscan Formation which degrades into stiff to very stiff sandy clay/silt and medium dense to dense clayey sand, eventually grading to competent lahar. This investigations at Paradise High School were performed for the Gymnasium currently under construction located between the existing softball field and eastern tennis courts in the northern portion of the Paradise High School campus located across the street from the RHS site. No geological hazards were identified in the evaluations.

2.2 REGIONAL GEOLOGY

The proposed RHS is situated within the Cascade geologic province near the northwest boundary of the Sierra Nevada geologic province of California. The Sierra Nevada geologic province is characterized by uplifted granitic batholiths. In the northwest portion of the province, several individual plutons of granitic rock are separated from the main batholith by a wide belt of metamorphic rocks and the Foothills Fault System.

The Cascade geologic province is characterized by Pliocene age volcanic sequences of lahars and lava flows that cover the pre-Cenozoic (66.0 million years before present [mybp] and older) metamorphic and plutonic rocks of the northern Sierra Nevada (Wakabayashi and Sawyer, 2001). The metamorphic basement rocks and plutonic rocks are visible along the deep stream incision of the Feather River to the northeast and east of the Town of Paradise. The Cascade volcanic rocks are composed of a coalescing sequence of Pliocene age andesitic lahars, and andesitic and basaltic lava flows. The volcanogenic deposits in the subject area are part of the Olivine Basalt of Paradise and the Tuscan Formation.

2.3 SITE GEOLOGY

According to the *Geologic Map of the Chico Monocline and Northeastern Part of the Sacramento Valley, California* (Harwood, et al. 1981) the site is underlain by the Olivine Basalt of Paradise. These Pliocene volcanic rocks are composed of grey, slightly weathered, vesicular, glomeroporphoritic olivine basalt with phenocrysts of clinopyroxene, olivine, and plagioclase.

Beneath the basalt is the Tuscan Formation, a series of Pliocene lahars and interbedded tuffs, overlying Paleozoic (240 to 570 mybp) age metamorphic, auriferous channel deposits, and marine sedimentary rocks. The Tuscan Formation is a wedge-shaped mass, which tilts and thins southwestward. Superimposed on this wedge-shaped mass are several folds and numerous factures with small to negligible offset (Lydon, 1967). A geologic map of the site area provided from the *Geologic Map of the Chico Quadrangle* (Saucedo and Wagner, 1992) is provided as Figure 3.

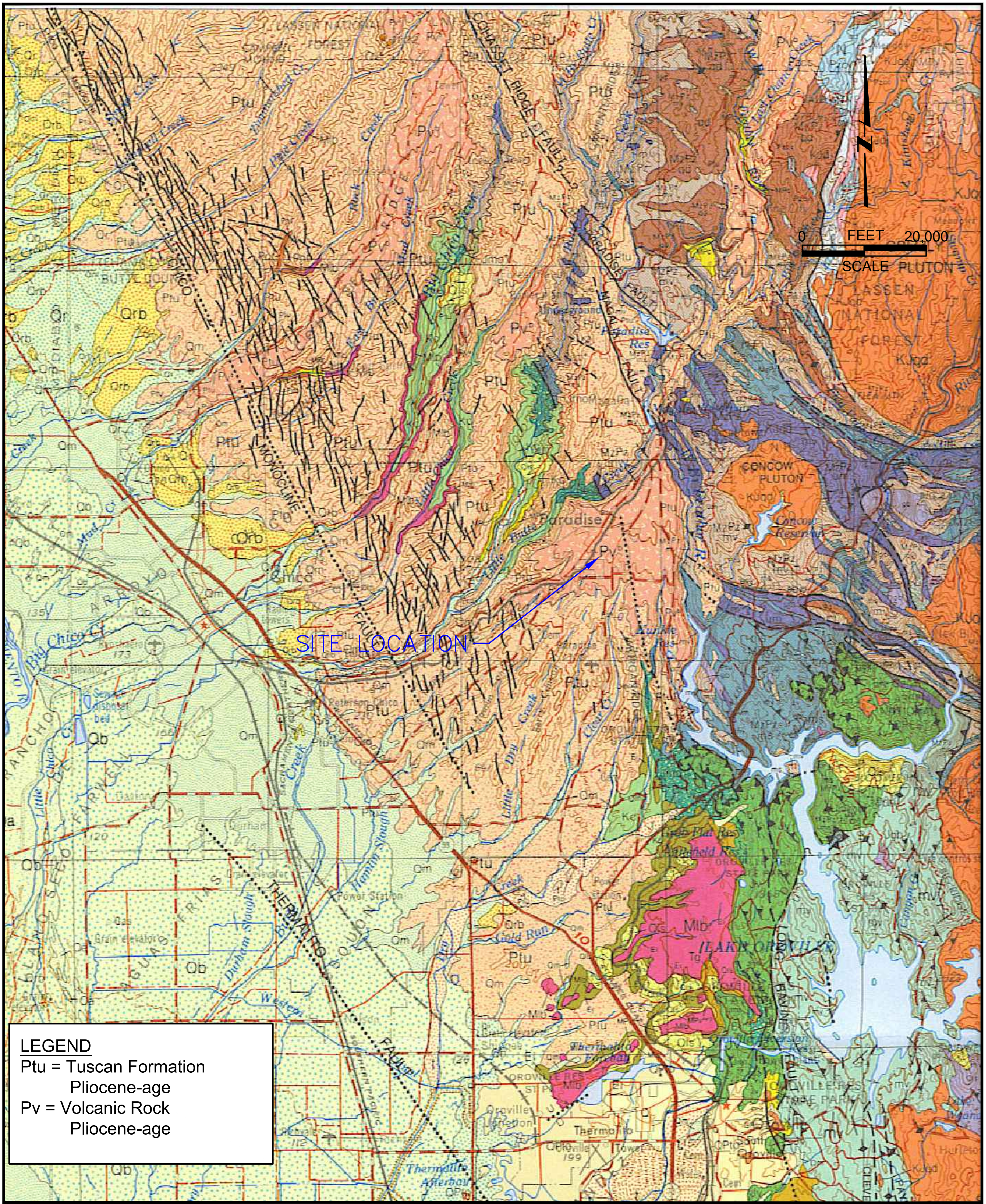
2.4 REGIONAL FAULTING AND SEISMIC SOURCES

Regional faulting is associated with the northern extent of the Foothill Fault System which includes the Chico Monocline, Cohasset Ridge Fault, Paradise Fault, Magalia Fault, and the Cleveland Hill 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 northern part of the fault zone is split into three branches: the Melones fault zone to the east, the Cleveland Hills fault to the south, and Chico Monocline to the west. The Chico Monocline Fault is identified as a major tectonic boundary with late Cenozoic displacement responsible for the formation of the Chico monocline. The fault is listed as Quaternary age and may have experienced anomalous aftershocks after the 1975 Oroville earthquake (Harwood and Helley, 1985).

NV5 reviewed the CGS California Earthquake Hazard Zone Application (EQ Zapp) on the internet at (https://maps.conservation.ca.gov/cgs/EQZApp/app/). 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. Review of the available maps referenced in EQ Zapp (updated April 4, 2019) indicates 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 RHS.

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 CGS Fault Activity Map of California (2010), (https://maps.conservation.ca.gov/cgs/fam/) also shows the nearest known active faults with surface displacement within Holocene time to be the Cleveland Hill Fault. The mapped fault zone is located approximately 20 miles south of the subject site and is associated with ground rupture during the Oroville earthquakes of 1975. The approximate location of the RHS site identified on the *Fault Activity Map of California and Adjacent Areas* is presented as Figure 4.

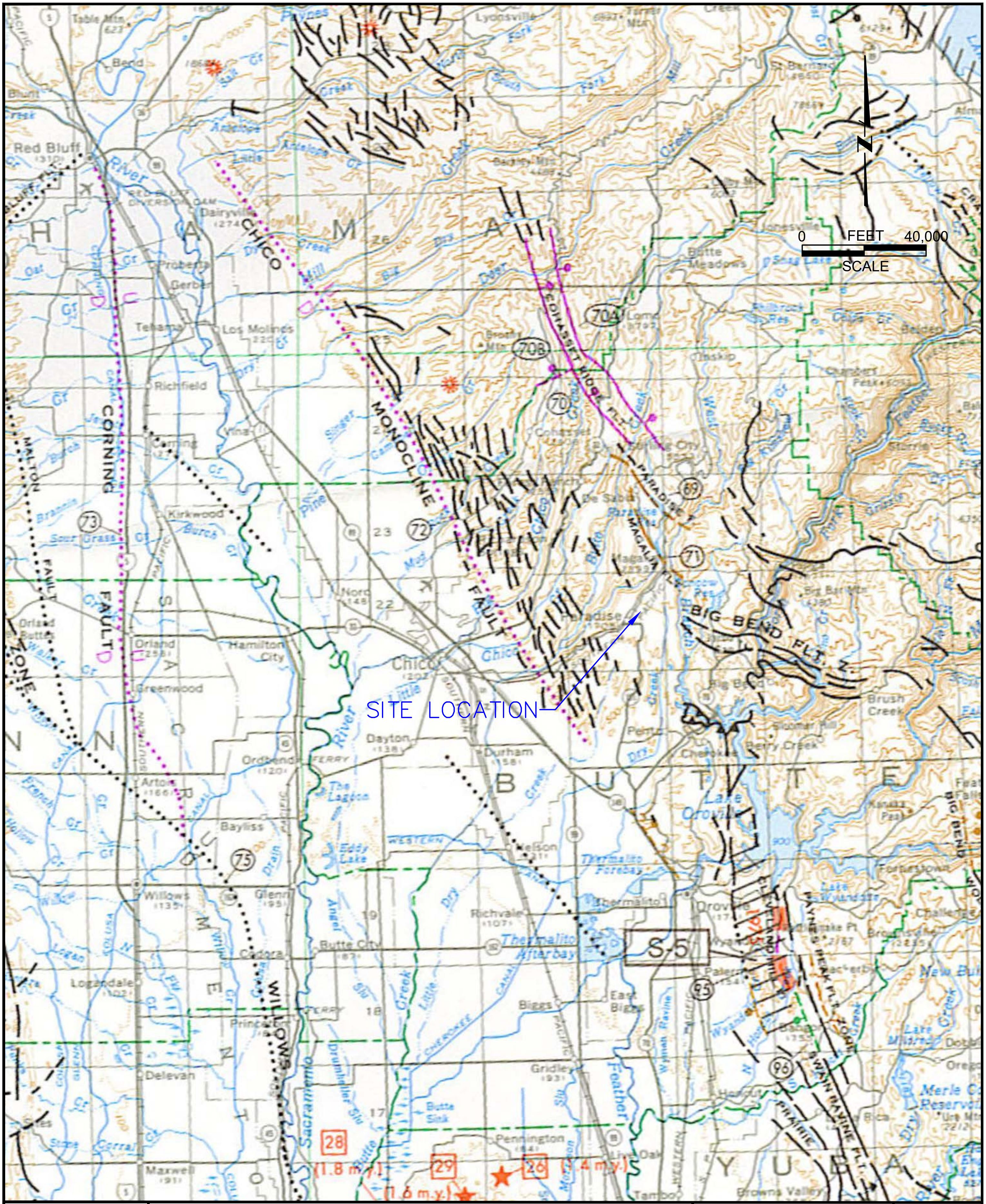


LEGEND

Ptu = Tuscan Formation
Pliocene-age

Pv = Volcanic Rock
Pliocene-age

NV5	REGIONAL GEOLOGIC MAP PARADISE USD – RIDGEVIEW HIGH SCHOOL PARADISE, BUTTE COUNTY, CALIFORNIA	DRAWN BY: SC CHECKED BY: DJP PROJECT: 70994.00.001 DATE: AUGUST 2020	FIGURE 3



NV5

FAULT ACTIVITY MAP
PARADISE USD – RIDGEVIEW HIGH SCHOOL
PARADISE, BUTTE COUNTY, CALIFORNIA

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FIGURE
4

2.5 FIELD INVESTIGATION

NV5 performed a field investigation of the site on June 30, 2020. 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 (I)* 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 proposed building footprint and approximate exploratory boring locations.

The area of the proposed RHS currently supports AC pavements in a major portion of the site. The northeastern portion of the site is undeveloped land supporting volunteer weeds, grasses, and brush. Several tree stumps were observed in the eastern portion of the site from trees removed following the 2018 Camp Fire.

2.5.2 Subsurface Conditions

The subsurface soil and groundwater conditions were investigated by excavating exploratory borings in the vicinity of the proposed RHS building. The subsurface information obtained from this investigation method is described in the following subsections.

2.5.2.1 Exploratory Boring Information

NV5 excavated a total of 3 exploratory soil borings at the project site. The borings were advanced with a CME-75 truck-mounted drill rig equipped with 6-inch diameter hollow stem augers. Figure 2 shows the approximate locations of the subsurface exploratory excavations. The borings were excavated to maximum depths of 19.0 to 47.5 feet below ground surface (bgs). Boring B20-1 encountered practical refusal to excavation on bedrock at a depth of 47.5 feet bgs. Engineering judgment was used to extrapolate the observed soil, rock and groundwater conditions to areas located between and beyond the subsurface exploratory excavations. 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.

NV5's field engineer/geologist logged each exploratory boring using the ASTM D2487 USCS as guidelines for soil descriptions and the American Geophysical Union guidelines for rock descriptions. Relatively undisturbed soil samples were collected with an unlined standard penetration test (SPT) split-spoon sampler and 2.5-inch-inside-diameter, split-spoon sampler equipped with stainless steel liner sampler tubes. The samplers were driven into the soil using an automatic trip hammer weighing 140 pounds with a 30-inch free-fall. The stainless-steel liner samples were sealed with labeled plastic caps. The samples collected with the SPT sampler were sealed in labeled plastic bags.

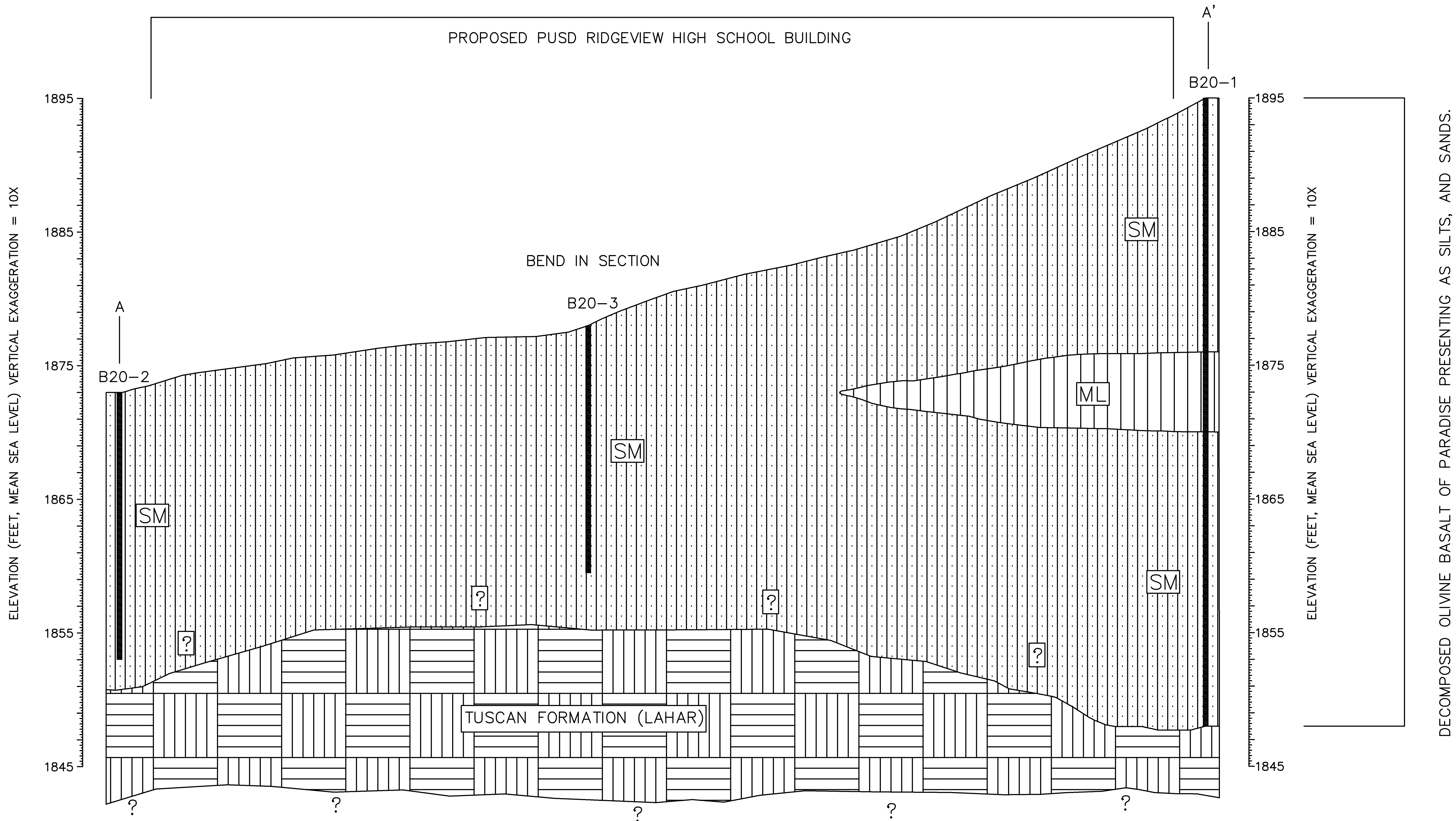
Representative bulk samples of the near-surface soil materials generated from drilling the exploratory borings also were collected and placed in labeled sample bags. The soil samples collected in the exploratory borings were transported to NV5's Chico soil laboratory facility.

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 C. 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 (47.5 feet) below the proposed building area 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.

- **SM, Silty Sand Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 60 percent fine sand and 40 percent low to high plasticity silt and clay fines. This soil is predominantly dark reddish brown with a Munsell® Soil-Color Chart designation of (2.5Y 3/4). This soil was loose to medium dense and moist at the time of the subsurface investigation.
- **ML, Low Plasticity Silt Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 60 percent low to high plasticity silt and clay fines and 40 percent fine sand. This soil is predominantly yellowish red with a Munsell® Soil-Color Chart designation of (5YR 4/6). This soil was stiff and moist at the time of the subsurface investigation.
- **RX, Rock (Tuscan Formation):** The Tuscan Formation rock encountered near the maximum depth of Boring B20-1 generally consists of a gray (10YR 5/1) fresh to slightly weathered, hard to very hard, massive, poorly sorted sandstone. Practical refusal to drilling was encountered in this unit.

NV5 prepared a geologic cross section using the geologic boring logs from exploratory borings B20-1 through B20-3 performed for this investigation. The alignment of the geologic cross section is presented in Figure 2. The geologic cross section is presented in Figure 5.



GENERALIZED GEOLOGIC UNITS AND USCS CLASSIFICATION

Low plasticity fines, and less than 50% Sand (USCS: ML)

Well or poorly graded sand, and greater than 12% silt or clay (USCS: SM)

Tuscan Formation: volcanogenic mudstone, siltstone, lahar, up to 1000 feet thick.

0 FEET 20
SCALE

NV5

GEOLOGIC CROSS SECTION A-A'

PARADISE USD - RIDGEVIEW HIGH SCHOOL

PARADISE, BUTTE COUNTY, CALIFORNIA

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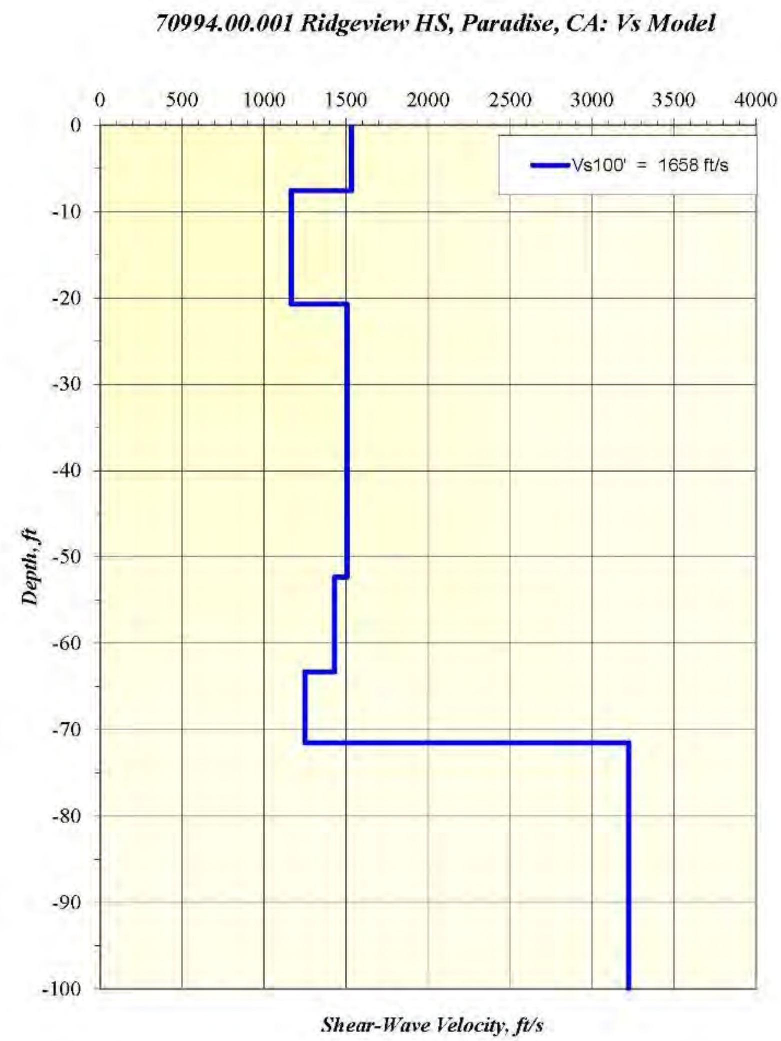
FIGURE 5

2.5.2.2 Seismic Refraction Microtremor Survey

The Seismic Refraction Microtremor Survey (SRMS) performed on 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 beneath the site. The measured S-wave profile is used to determine the CBC Site Class in accordance with Chapter 16A, Section 1613A.3.2 and Chapter 20 of ASCE 7-16.

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. The data was collected in a series of twenty-one, 30-second-long, continuous recording periods. The inset image 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 1658 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 Soil Profile), as described in Chapter 20, Table 20.3-1 Site Classification of ASCE 7-16.

2.5.2.3 Groundwater Conditions

Groundwater was initially encountered within exploratory boring B20-1 at a depth of approximately 45.0 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 State Water Resources Control Board's Groundwater Ambient Monitoring and Assessment Program (GAMA) database (https://gamagroundwater.waterboards.ca.gov/gama/gamamap/public/) and the groundwater depths encountered in the previous borings performed at the adjacent Paradise High School to review historical groundwater elevation data in the immediate area. Based on review of initial groundwater elevation data generated from well drilling reports within the near vicinity of the site and the groundwater depths encountered in the previous borings performed at the adjacent Paradise High School, NV5 estimates that historically high groundwater may be encountered at depths of 30 feet to 40 feet bgs in the late winter or spring during periods of above average and prolonged rainfall.

The hydrogeologic properties of the Tuscan Formation include low permeable mudstones interlayered with siltstone and sandstone layers that don't transmit water very well. Based on our experience of seepage encountered in excavations in the area, the contractor should expect that some seepage may be encountered within the foundation of our utility excavations that may require dewatering.

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 G57 Resistivity (100% saturation)
- ASTM D422 Particle Size Gradation (Sieve Only)
- ASTM D1498 Redox
- ASTM D2216 Soil Moisture Content
- ASTM D2487 Soil Classification by the USCS
- ASTM D2850 Unconsolidated-Undrained Triaxial Compressive Strength
- ASTM D2937 In Place Density of Soil
- ASTM D4318 Atterberg Limits (Dry Method)
- ASTM D4327 Chloride and Sulfate
- ASTM D4972 pH

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

Table 3.0-1, Laboratory Test Results

Boring	Sample		ASTM Test Results(1)							
No.	No.	Depth	D2487 D2488	D2216	D2937	D422		D4318		D2937
		(ft)	USCS (sym)	Moisture Content (%)	Dry Density (pcf)	Passing No. 4 Mesh Sieve (%)	Passing No. 200 Mesh Sieve (%)	Plasticity Index (%)	Liquid Limit (%)	UU Compressive Strength (psf)
B20-1	BLK-1	1-4	SM	-	-	97.6	22.9	21	54	-
B20-1	L2-1-2	5	SM	30.6	86.9	-	-	-	-	5,277.2
B20-1	B3-1-1	20	ML	-	-	100	54.6	NP	NP	-
B20-2	L3-1-2	8.5	SM	26.1	88.2	-	-	-	-	-
B20-3	L1-1-2	1	SM	25.9	83.7	-	-	-	-	3,091.0

Notes:
(1) Laboratory test forms are presented in Appendix D
% percent
ASTM ASTM International
ft feet
No. number
NP non-plastic
sym symbol
pcf pounds per cubic foot
psf pounds per square foot
USCS Unified Soils Classification System
UU Unconsolidated Undrained

4.0 SEISMICITY

4.1 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 online 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 Cascade Range transition areas since 1836. These earthquakes include the following events:

- The February 8, 1940, 5.7 M_L earthquake was referred to as the “The Ghost Earthquake” because, at the time it occurred, it was not associated with a known fault or precise location. The most recent estimate of the Ghost Earthquake location is approximately 23 miles (37 km) northwest of the subject site and approximately 20 miles northwest of Magalia, California, however, it is generally not considered to be associated with the Magalia Fault (Dudley, T, 1988).
- The March 20, 1950, 5.5 M_L earthquake occurred in the southeast area of Lassen Volcanic National Park. This event was located approximately 50 miles (80 km) northeast of the subject site.
- The August 1, 1975, 5.5 M_L Oroville Earthquake main shock occurred on the Cleveland Hill Fault located approximately 7 miles (11 km) south of Lake Oroville near the town of Bangor, California, which is approximately 20 miles (32 km) southeast of the subject site. This earthquake was accompanied by surface faulting which extended for several kilometers (Akers and McQuilkin, 1975). The earthquake sequence consisted of five foreshocks (M_L 3 or greater), the main shock, and numerous aftershocks (Toppozada and Cramer, 1984).
- May 23, 2013, 5.7 M_L Greenville earthquake swarm occurred approximately 6.6 miles (11 km) west-northwest of the town of Greenville in Plumas County, which is approximately 40 miles (64 km) northeast of the subject site. The main shock was followed by an aftershock sequence totaling nearly 300 events in the following 48 hours. Moderate damage occurred to homes in the immediate area of Lake Almanor, and the initial shock was felt as far south as Elk Grove, however, no documented damage was reported in the Paradise area.

No structural damage was recorded or documented to have occurred to structures in the Town of Paradise during these events.

4.2 SEISMIC DESIGN PARAMETERS

NV5 developed the code-based seismic design parameters in accordance with Section 1613 of the 2019 CBC and the Structural Engineers Association of California (SEAOC), *Seismic Design Maps* web application. The internet based application (www.seismicmaps.org) is used for determining seismic design values from the 2016 ASCE 7-16 Standard (erratum released February 2019). The spectral acceleration, site class, site coefficients and adjusted maximum considered earthquake spectral response acceleration, and design spectral acceleration parameters are presented in Table 4.2-1. The Seismic Design Parameter detailed report from the SEAOC analysis is provided in Appendix E.

4.2.1.1 Seismic Design Category

Based on the short period response acceleration ground motion parameters above (S_{DS} = 0.564) and the Risk Category of I or II, and III, the Seismic Design Category is D. Based on the 1-S period response acceleration ground motion parameters above (S_{D1} = 0.289) and the Risk Category of I or II, and III, the Seismic Design Category is D. Therefore, the Seismic Design Category for the site is D.

4.2.1.2 Geometric Mean Peak Ground Acceleration

NV5 used the SEAOC *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}) multiplied by the PGA mapped values found on Figure 22-9 from ASCE 7-16. The PGA_M was calculated using the following equation:

$$PGA_M = F_{PGA}PGA = 1.2 \times 0.305 = 0.366 \text{ g}$$

The Seismic Design Maps report from the SEAOC analysis is provided in Appendix E.

Table 4.2-1, 2019 CBC Seismic Design Parameters

Description	Value	Reference
Latitude North (degree)	39.7622	Google Earth
Longitude West (degree)	-121.6121	Google Earth
Site Coefficient, F_A	1.223	*2019 CBC, Table 1613.2.3(1), SEAOC Seismic Design Maps*
Site Coefficient, F_V	1.500	*2019 CBC, Table 1613.2.3(2), SEAOC Seismic Design Maps*
Site Class	C = Very Dense Soil and Soft Rock	*ASCE 7-16, Chapter 20, Table 20.3-1*
Short (0.2 sec) Spectral Response, S_S (g)	0.692	*ASCE 7-16, Section 11.4.2, SEAOC Seismic Design Maps*
Long (1.0 sec) Spectral Response, S_1 *(g)*	0.289	*ASCE 7-16, Section 11.4.2, SEAOC Seismic Design Maps*
Short (0.2 sec) MCE Spectral Response, S_{MS} *(g)*	0.846	*ASCE 7-16, Section 11.4.4, SEAOC Seismic Design Maps*
Long (1.0 sec) MCE Spectral Response, S_{M1} *(g)*	0.433	*ASCE 7-16, Section 11.4.4, SEAOC Seismic Design Maps*
Short (0.2 sec) Design Spectral Response, S_{DS} *(g)*	0.564	*ASCE 7-16, Section 11.4.5, SEAOC Seismic Design Maps*
Long (1.0 sec) Design Spectral Response, S_{D1} *(g)*	0.289	*ASCE 7-16, Section 11.4.5, SEAOC Seismic Design Maps*
Seismic Design Category (Risk Category I, II or II)	D	*ASCE 7-16, Section 11.6, SEAOC Seismic Design Maps*
Geometric Mean Peak Ground Acceleration (PGA_M) (g)	0.366	*ASCE 7-16, Section 11.8.3, SEAOC Seismic Design Maps*

CBC = California Building Code
MCE = Maximum Considered Earthquake
g = gravitational acceleration (9.81 meters per second^2 = 32.2 feet per second^2)
sec = second

4.3 DEAGGREGATED SEISMIC SOURCE PARAMETERS

To calculate the deaggregated seismic source parameters, NV5 used the USGS's Earthquake Hazard Program Unified Hazard Tool software (https://earthquake.usgs.gov/hazards/interactive/index.php) which uses probabilistic methods to estimate the seismic ground motions for the site to perform the probabilistic seismic hazard deaggregation and produce the maximum horizontal acceleration (MHA) expected to occur at the site resulting from the MCE. The USGS Unified Hazard Tool allows the user to select site conditions by inputting Vs30 values, which allows for the use of more realistic site conditions. The probabilistic MCE spectral response accelerations shall be taken as the spectral response accelerations represented by a 5 percent damped acceleration response spectrum having a 2 percent probability of being exceeded within a 50-year period. This MCE has a 2,475-year return period. ASCE 7-16, Chapter 11, Section 11.2 defines the design earthquake ground motion as the earthquake ground motions that are two-thirds of the corresponding MCE ground motions. A peak horizontal ground acceleration with a 10 percent probability of being exceeded in 50 years, which corresponds to a 475 year return period and represents the MHA for the site for soft rock conditions, was utilized in determining the seismic coefficient (k) for the pseudo-static analysis of the natural slope at the site in accordance with CGS Special Publication 117A.

The probabilistic seismic hazard deaggregation analyses presenting the peak ground acceleration, model magnitude, and modal distance for the MCE is presented in Table 4.3-1. NV5 estimated a peak ground acceleration of 37.90 percent of gravitation acceleration (0.379 g) and therefore the design earthquake ground motion for the site to be used for liquefaction is 0.252 g (2/3 MCE = 2/3[0.379g]). A representative earthquake with a moment magnitude of 9.00 was also estimated for use in liquefaction or slope stability analyses by performing a deaggregation of the probabilistic seismologic data.

Table 4.3-1, Probabilistic Seismic Hazard Deaggregation

Probability of Exceedance	Modal Magnitude (M_w)	Modal Distance to Fault (miles / km)	Peak Ground Acceleration (g)
2 percent in 50 years (2475 year return)	9.01	87.1 / 140.2	0.379
Design Earthquake Ground Motion = MCE (2/3)	9.01	87.1 / 140.2	0.252
MHA, 10 percent in 50 years (475 year return)	9.01	87.1 / 140.2	0.185

km = kilometers
M_w = Maximum Considered Earthquake
g = gravitational acceleration (9.81 meters per second^2 = 32.2 feet per second^2)

A plot of the deaggregated distance, magnitude and ground motion uncertainty for the specified parameters for each return period is provided in Appendix E.

5.0 LIQUEFACTION AND SEISMIC SETTLEMENT

NV5 evaluated the potential for liquefaction occurring at this site based on the evidence of previous site investigations performed in the area of the site using subsurface exploratory boring SPT blow count and field data, probabilistic seismic expected 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, young (Holocene), clean, loose, uniformly-graded sand and loose, silty sand soils that are saturated are the most prone to undergo liquefaction; however, gravelly soil, and some geologically young clay-rich soil may be prone to liquefaction under certain conditions. The site geology is mapped as Olivine Basalt of Paradise, composed of Pliocene (older than 1.2 million years) volcanic rocks consisting of grey, slightly weathered, vesicular, glomeroporphoritic olivine basalt with phenocrysts of clinopyroxene, olivine, and plagioclase. Beneath the basalt is the Tuscan Formation, a series of Pliocene lahars and interbedded tuffs, overlying Paleozoic (240 to 570 mybp) age metamorphic, auriferous channel deposits, and marine sedimentary rocks. The Olivine Basalt and Tuscan Formations are not prone to liquefaction and no liquefaction hazard zones were designated by the local or state regulatory authority in this geologic unit. Taking into account the degree of competency and weathering, solidification, over consolidation and age of the Olivine Basalt and Tuscan Formations, and no previously reported case history of liquefaction occurring within the area, it is NV5's opinion that the probability of liquefaction and seismically-induced settlement to occur is very low and the site will not undergo seismically induced settlement.

5.2 SEISMIC SETTLEMENT AND LATERAL SPREADING

Because the potential for liquefaction of the soil and rock beneath the site is considered low, with the site and surrounding areas to be relatively flat, NV5 considers the site not susceptible to post-liquefaction settlement and lateral spreading that would be detrimental to the proposed site improvements.

6.0 OTHER GEOLOGIC HAZARDS

NV5 is providing a complete evaluation for the potential geologic hazards that could be applicable to the RHS area in order to compile a thorough report for the site that is up to date with the current guidelines and code standards. The evaluation of geologic hazards for the site was based on NV5's review of geologic maps and literature, regional aerial photographs, a site reconnaissance, and analysis of the soil and rock conditions encountered during the June 30, 2020 site investigation. This section provides additional information to meet the 2019 CBC and CGS Note 48 (November, 2019). The RHS site is not located within special geologic hazard zones designated by CGS 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 the subsurface geotechnical investigation are characterized as fine grain (silt and clay) size soils. Atterberg Limits (ASTM D4318) was performed on representative near-surface soil samples collected during the subsurface investigation. The Atterberg Limits test results indicate the near-surface fine grain soil material encountered in exploratory boring B20-1 to be high plastic (MH) soils. Previous Expansion Index (ASTM D4829) testing performed on these near-surface high plastic soils during geotechnical investigations at the adjacent Paradise High School indicate the near-surface soils have a very low to low expansion potential. Based on review of the 2019 CBC, the results of the Atterberg Limits and Expansion Index testing and our experience with similar soils in the area, the potential for expansive soil hazards to affect the proposed building is considered low if these soils are left in place beneath the proposed building.

6.2 SOIL CORROSION POTENTIAL

NV5 performed minimal testing to evaluate the corrosion potential of the onsite shallow soils located at the RHS site that are anticipated to be in contact with concrete foundations and underground pipes associated with the proposed improvements. The soil samples tested were collected at a depth of approximately 1 to 3 feet bgs. The test results are summarized in Table 6.2-1 below.

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

Sample No.	Sample Depth (feet)	Test No.	Description	Test Results
B20-1 Blk-1	1-4	ASTM D1498	Redox	440 mV
		ASTM D4327	Chloride	N.D.
		ASTM D4327	Sulfate	N.D.
		ASTM D4972	pH	4.99
		ASTM G57	Resistivity	13,000 ohms-cm

ASTM = ASTM International
mg/kg = milligram per kilogram
mV = millivolts
N.D. = none detected
ohms-cm = ohms-centimeters

The pH concentration is insufficient to damage 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 and sulfate) the soil is considered 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 manufacturer's recommendations. The laboratory report and brief summary of results are included in Appendix D.

6.3 VOLCANIC HAZARDS

Although volcanic rock deposits are identified as the geologic formations at the site, those volcanic units are Pliocene age. According to the USGS Bulletin 1847, *Potential Areas of Future Volcanic Eruptions in California* (Miller, 1989), the property is not situated within a recognized active volcanic area. The nearest known active volcanic zone is the Mt. Lassen area, located approximately 50 miles north of the site. The most recent volcanic eruptions occurring at Mt. Lassen were from 1914 to 1917. In summary, our opinion is that the potential for encountering a volcanic hazard within the proposed building footprint area to be extremely low.

6.4 FLOODING

The subject property is located on a ridge top between the North Fork of the Feather River to the east and Butte Creek to the west. The site is not located within 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.

There are no dams or reservoirs near the site that pose a potential inundation or flood hazard. 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 will negatively impact the proposed building footprint areas to be extremely low.

6.5 LANDSLIDES

The existing topography at the site and near vicinity consists of low to moderately sloping hillside terrain. The site is not located in an area of known historical landslides. The site and near vicinity are underlain by dense, weathered basaltic lava flow deposits and relatively flat lying to very low dipping (<5 degrees) massive lahar, siltstone, sandstone and conglomerate rock units. No evidence of past landslides or soil creep was identified during our field investigation. Due to the lithified characteristics of the Olivine Basalt Formation, the potential for the occurrence of a landslide hazard at the proposed building area is very low.

6.6 TSUNAMIS AND SEICHES

There are no bodies of water with the potential for tsunamis and or seiches located near the subject property. In summary, we believe that the potential for encountering a tsunamis and/or seiches hazards within the proposed building footprint area to be low.

6.7 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 hard, competent rock with soil weathering. NV5's opinion is that the potential for slumping and land subsidence hazards to occur within the engineering fill embankment, native soil or rock sections encountered within the proposed building improvement areas to be low.

6.8 MINING RELICS

NV5 did not observe any evidence of past mining activities during our site reconnaissance. Our 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, we believe that the potential for encountering past mining-related hazards within the proposed building footprint areas to be low.

6.9 RADON-222 GAS

Butte County and the subject site are not in an area identified as having an increased chance of elevated radon content in soil gas. Radon gas concentrations are considered to be elevated at 4 picoCuries per liter (pCi/L). However, each of the radon gas literature sources reviewed indicated that elevated radon gas in buildings may still exist in areas that are predicted to not have elevated radon gas.

The United States Environmental Protection Agency *Map of Radon Zones* (viewed June 28, 2016 at: www.epa.gov/radon/epa-map-radon-zones) indicates that Butte county is located in Radon Zone 3. This zone consists of counties with a predicted average indoor radon screening level less than 2 pCi/L.

H&K's review of the *Geologic Controls on the Distribution of Radon in California* prepared by the California Geological Survey, dated January 25, 1991 indicates that Butte County is not underlain by geologic deposits that increase the chance of elevated radon gas.

CDHS published the *California Indoor Radon Levels Sorted by Zip Code* (Last updated Feb. 2016). This database summary indicated that, in the 95973 Zip Code for Butte County (City of Chico), radon concentrations were less than the CDHS recommended action level of 4 pCi/L in 9 of 12 indoor air tests.

6.10 NATURALLY OCCURING ASBESTOS

NV5 reviewed geologic literature regarding the distribution and occurrence of naturally occurring asbestos (NOA) in California. The site is not in an area mapped as likely to contain NOA and NV5's field engineer/geologist did not observe the presence of ultramafic rock outcrops (typically associated with the occurrence of NOA) at the site.

Based on review of the California Department of Conservation, Division of Mines and Geology, 2000. *A General Location Guide for Ultramafic Rocks in California - Areas Likely to Contain Naturally Occurring Asbestos*, August, Map scale 1:1,100,000, Open-File Report 2000-19 ultramafic rock is mapped approximately 3.0 miles east of the site.

The *Geologic Map of California, Chico Sheet* (California Department of Conservation, Division of Mines and Geology, 1962) shows a Mesozoic aged ultrabasic intrusive rock unit mapped approximately 3.0 miles east of the site in the foothills of the Sierra Nevada mountain range within the West Branch of the Feather River drainage. Drainage areas for Butte Creek, Big Chico Creek, and the West Branch of the Feather River crosscut the ultrabasic rock unit. The subject site is topographically higher than the drainage areas of the creeks and is separated by over three miles of land, which likely precludes inundation and deposition of sediment that could potentially contain NOA, thus NOA the potential to encounter NOA as the site is considered to be extremely 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. The site is not located within a geologic hazard zone or special studies zone mapped by the CGS, Butte County, or the Town of Paradise. The subject property does not contain geologic hazards that require mitigation in order for the proposed improvements to proceed. Based on the site geology and the observations within the exploratory borings, the site soil profile can be modeled, according to the 2019 CBC, Chapter 16*A*, and ASCE 7-16, Chapter 20, as a Site Class C (Very Dense Soil and Soft Rock Soil Profile) designation for the purposes of establishing seismic design loads for the proposed improvements.
3. Based on the site geology, results of the SRMS survey, subsurface exploratory boring blow counts collected from the borings performed on the site, 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. At the time of the NV5 site investigation, the area of the proposed RHS currently supports AC pavements in a major portion of the site. The northeastern portion of the site is undeveloped land supporting volunteer weeds, grasses, and brush. Several tree stumps were observed in the eastern portion of the site from trees removed following the 2018 Camp Fire.
5. The soil conditions observed to a maximum depth of 47.5 feet below the existing ground surface in our subsurface exploratory excavations (described relative to the existing ground surface) generally consisted of dark reddish brown, loose to medium dense silty sand (SM) and yellowish red, stiff, sandy silt (ML) underlain by competent, gray volcanic rock (RX).
6. NV5's field and laboratory test data indicates that the native sand (SM) soil units encountered beneath the site have the following general geotechnical engineering properties: medium dense to very dense, low to high plasticity, and a moderate bearing capacity that is suitable for supporting shallow foundations.
7. Groundwater was initially encountered in exploratory boring B20-1 at a depth of approximately 45.0 feet bgs at the time of this subsurface investigation. Based on past construction activities in the area, it is common to encounter shallow groundwater seepage in deep foundation boreholes or utility excavations. Based on the above average rainfall, subsurface geologic conditions and review of groundwater elevations encountered during domestic well drilling near the site, NV5 assumes that for design and evaluation purposes, the historically high groundwater table will probably be located at a depth of approximately 30 to 40 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 proposed spread foundations may be founded on conventional continuous and/or spread footings founded in firm, non-expansive native soil or properly compacted fill. NV5's recommendations are presented below.

8.1 EARTHWORK GRADING

NV5's earthwork grading recommendations include demolition and abandonment of existing site improvements, import fill soil, temporary excavations, stripping and grubbing, native soil preparation for engineered fill placement, engineered fill construction with testable earth materials, erosion controls, underground utility trenches, construction dewatering, soil corrosion potential, subsurface groundwater drainage, surface water drainage, grading plan review and construction monitoring.

8.1.1 Demolition and Abandonment of Existing Site Improvements

NV5 anticipates that the existing site improvements within the proposed building areas will need to be demolished and removed from the site as described below.

1. The existing asphalt concrete and aggregate base (AB) rock pavement materials within the proposed building areas should be excavated and disposed of offsite. However, it may be possible to use some of this demolition material to construct engineered fills provided they meet the gradation requirements specified for "testable fill" materials presented in this report. The project geotechnical engineer should approve the use of both AC and AB rock demolition materials for use in constructing engineered fills.
2. All foundations, underground utilities and other existing site improvements, including the underground waste storage tank, that are encountered during construction within the proposed building area should be demolished and removed from the site. These demolition materials should be disposed off-site in compliance with applicable regulatory requirements.
3. Abandonment of any underground utilities within the construction area that will not interfere with the proposed site improvements should be plugged with cement grout to reduce migration of soil and/or water.

8.1.2 Import Fill Soil

Import fill soil should meet the geotechnical engineering material properties described in Section 8.1.6-1 (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 geotechnical engineer should approve all proposed import fill soil for use in constructing engineered fills at the site.

8.1.3 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.

8.1.4 Stripping and Grubbing

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

1. Strip and remove the top 2 to 4 inches of soil from the field areas containing shallow vegetation roots and other deleterious materials. This highly organic topsoil can be stockpiled on-site and used for surface landscaping but should not be used for constructing compacted engineered fills. 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 their representative should approve the use of any soil materials generated from the clearing and grubbing activities.
2. Remove all existing structures and underground utilities extending through the proposed structural improvement areas. 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 geotechnical engineer or their 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. Excessively large amounts of vegetation and other deleterious materials should be removed from the site.

8.1.5 Native Soil Preparation for Engineered Fill Placement

After completing site stripping and grubbing activities, the exposed native soil 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 ± 2 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 geotechnical engineer, or their 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 geotechnical engineer or their 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.) and AC pavement 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 their field representative, to be firm, competent and relatively unyielding. The project geotechnical engineer or their 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 8.1.5-1 or as modified by the project geotechnical 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 8.1.5-1, 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 based on the site conditions encountered during grading.
(2) Whichever criteria provide the greatest number of tests.

ASTM = ASTM International
CY = cubic yards
No. = number

8.1.6 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 8.1.6-1. 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 geotechnical 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 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.

8.1.6.1 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 geotechnical engineer or their 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 ± 2 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 the project geotechnical engineer or their 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 8.1.6.1-1 or as modified by the project geotechnical engineer to better suit the site conditions.

Table 8.1.6.1-1, 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 geotechnical engineer's discretion based on 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 geotechnical 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 geotechnical 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 8.1.5, Paragraph 3.

8.1.6.2 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.

8.1.7 Erosion Controls

Erosion controls should be installed as described below.

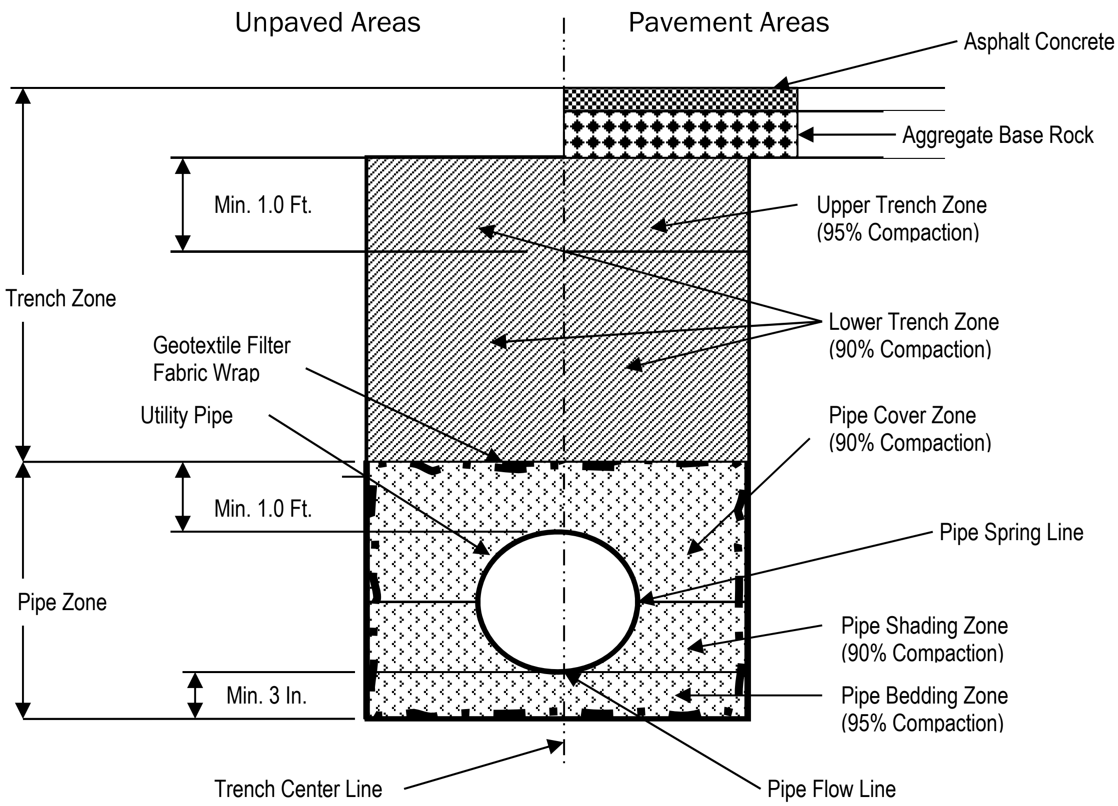
1. Erosion controls should be installed on all cut and fill slopes to minimize erosion caused by surface water runoff.
2. Install on all slopes either an appropriate hydroseed mixture compatible with the soil and climate conditions of the site, as determined by the local United States Soil Conservation 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.

8.1.8 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 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 dewatering methods as discussed in Section 8.1.9 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 8.1.8-1 below.

Detail 8.1.8-1 TYPICAL TRENCH BACKFILL ZONES



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- **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 geotechnical 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 geotechnical 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 8.1.8-1 or as modified by the project geotechnical engineer to better suit the site conditions.

Table 8.1.8-1, 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 geotechnical engineer's discretion based on 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 8.1.5, Paragraph 3.

8.1.9 Construction Dewatering

NV5 does not anticipate the need to perform dewatering of the site during earthwork grading, however, the earthwork contractor should be prepared to dewater 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 dewatering 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 dewatering method for each excavation.

1. NV5 anticipates that dewatering 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 dewatering 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.

8.1.10 Subsurface Groundwater Drainage

NV5 does 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 dewatering recommendations.

8.1.11 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.

8.1.12 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: shallow continuous strip and isolated foundations for buildings, shallow foundations, retaining walls entirely above the groundwater table, retaining wall backfill, and concrete slab-on-grade interior, sidewalk and patio construction. These recommendations are presented hereafter.

8.2.1 Shallow Foundations

Shallow continuous and isolated spread foundations that will support load bearing walls and interior columns 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:
   a. Minimum Width = 12 Inches
   b. Minimum Embedment Depth below the lowest adjacent exterior surface grade as shown in Table 8.2.1-1.
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 8.2.1-1.
   - 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 8.2.1-1, 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	6,000	2,000	2,660	3.0
18	7,500	2,500	3,325	3.0

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 per foot of depth. The passive pressure force and sliding friction coefficient for computing lateral resistance are as follows:
    a. Passive pressure = 300 (H), pounds per square foot (psf), where H = foundation embedment depth (feet) below lowest adjacent soil surface.
    b. Foundation bottom sliding friction coefficient = 0.35 (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 (psi) 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 their 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.

8.2.2 Retaining Walls Entirely Above the Groundwater Table

A California licensed civil engineer should design all retaining walls situated above the groundwater table with drained backfill using the following geotechnical engineering design criteria:

1. The retaining wall recommendations for static loading conditions are based on Rankine earth pressure theory published by W.J.M. Rankine (1857). The retaining wall recommendations for seismic loading conditions are based on the published work by Geraili and Sitar, *Seismic Earth Pressures on Retaining Structures in Cohesionless Soils*, (2013).

2. Retaining walls should be founded on firm competent bedrock or engineered fill consistent with the requirements of Section 8.1.

3. The retaining wall should be designed using the geotechnical engineering design parameters presented in Table 8.2.2-1.

4. The retaining wall backfill soil should be free draining material that meets or exceeds the material requirements of and is placed and compacted consistent with the requirements of Section 8.2.3.

5. The static lateral earth pressures exerted on the retaining walls may be assumed to be equal to an equivalent fluid pressure per foot of depth below the top of the wall. The lateral pressures presented in the table below are ultimate values and, therefore, do not include a safety factor, and assumes a free draining backfill (no hydrostatic forces acting on the wall) and no surcharge loads applied within a distance of 0.50H, where H equals the total vertical wall height.

6. The retaining wall backfill slope shall have a horizontal slope gradient for a minimum horizontal distance of 0.50H, where H equals the total vertical wall height. If a steeper backfill slope ratio is desired, then NV5 should be notified and contracted to perform additional retaining wall designs.

7. The retaining wall foundation excavations should be saturated prior to placing concrete to aid the concrete curing process. However, concrete should not be placed in standing water.

Table 8.2.2-1, Design Parameters for Retaining Walls

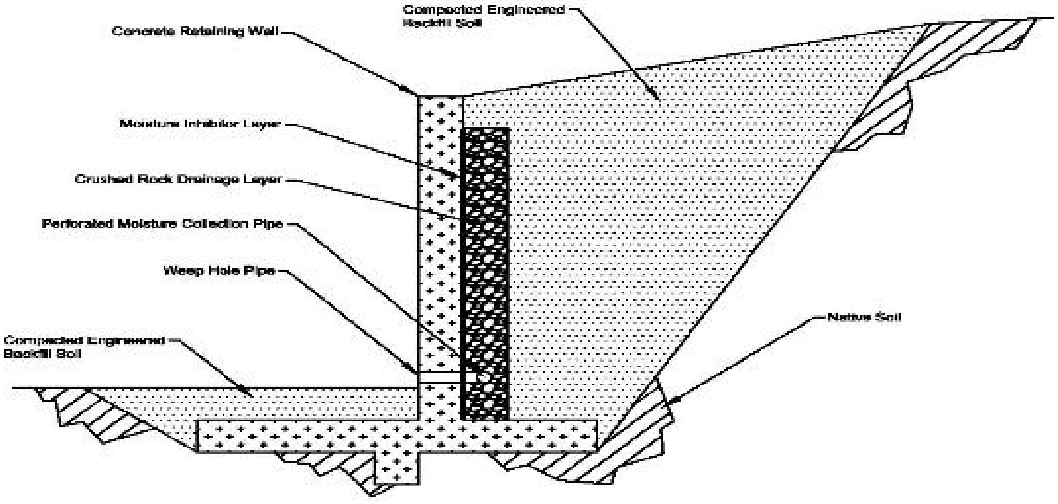
Design Parameters for Retaining Walls		
Loading Conditions	**Static Loads On Retaining Wall With Horizontal Backfill Slope**	**Seismic Load On Retaining Wall With Horizontal Backfill Slope**
Wall Active Condition Pressures (psf)/ft [(1)]	35 (H) [(5)]	4 (H)
Wall Passive Condition Pressures (psf)/ft [(2)]	300 (H)	4 (H)
Wall At-Rest Condition Pressure (psf)/ft [(3)]	50 (H)	10 (H)
P_{active} Force Located Above Foundation Base	0.33 (H)	Not Applicable
$P_{passive}$ Force Located Above Foundation Base	0.33 (H)	Not Applicable
$P_{at\text{-}rest}$ Force Located Above Foundation Base	0.33 (H)	Not Applicable
$P_{earthquake}$ Force Located Above Foundation Base	Not Applicable	0.33(H)
Maximum Allowable Foundation Bearing Capacity (psf), (Live + Dead Loads)	2,000	2,000
Maximum Allowable Foundation Bearing Capacity (psf) (Live + Dead + Wind or Seismic Loads)	2,660	2,660
Minimum Foundation Embedment Depth (in)	12	12
Foundation Bottom Friction Coefficient (dim.) [(4)]	0.30	0.30

Notes:
(1) The active pressure condition applies to a retaining wall with an unrestrained top (deflection allowed).
(2) The passive pressure condition applies to a retaining wall with soil resistance at the base. If passive pressures are used, then NV5 recommends that the top 1.0 feet of soil weight be ignored.
(3) The At-Rest pressure condition applies to a retaining wall with the top restrained (no deflection allowed).
(4) If the design horizontal resistance force acting on the wall foundation is computed by combining both the sliding friction force and passive soil pressure force, then the larger of the two forces should be reduced by 50 percent.
(5) H = The distance to a point in the backfill soil where the pressure is desired. The H distance is measured from the top of the wall for active and at-rest conditions and from one foot below the soil height at the toe of the wall for the passive condition (See Note 2 for passive condition).

8.2.3 Retaining Wall Backfill

Place and compact all retaining wall backfill and drainage layer materials as described below. NV5 did not review the final improvement plans for the site. If sub-structure retaining walls for below grade rooms, basements, garages, etc., are designed for this project, then these structures should also incorporate a water proofing sealant as described below. The water proofing sealant products should be installed by a qualified waterproofing contractor according to the manufacturer's directions. A typical retaining wall and backfill material zones figure is shown below.

TYPICAL CANTILEVER RETAINING WALL AND BACKFILL MATERIALS



NOT TO SCALE

1. **Waterproofing:** Waterproofing materials should be installed behind retaining walls prior to backfilling if retaining walls will be constructed for below grade rooms, basements, garages, elevator shafts, etc. The waterproofing materials should be installed by a qualified waterproofing contractor according to the manufacturer's directions.

2. **Drainage Layer:** A drainage layer should be placed between the wall and backfill material to prevent buildup of hydrostatic pressures behind the wall. Additionally, care should be taken during placement of the drainage layer materials so as not to crush, tear, or damage the waterproofing materials. The drainage layer can be constructed from drain rock, geosynthetic drain nets or a combination of both as described below.

   a. **Caltrans Class II Permeable Material Method:** Place a minimum 12-inch thick layer of Caltrans Class II Permeable Material directly against the wall or waterproofing system (as described below) without a geotextile wrapping to separate the backfill soil from the wall. The drainage material should extend from the wall bottom to within 12 inches of the wall top.

   b. **Geotextile Wrapped Drain Rock Method:** Place a minimum 12-inch-thick layer of drain rock wrapped in a geotextile filter fabric directly against the wall or waterproofing system (as described below) to separate the backfill soil from the wall. The drain rock should extend from the wall bottom to within 12 inches of the wall top. A minimum 6-ounce per square yard (oz/sy) non-woven geotextile fabric, such as Amoco 4506 manufactured by Amoco Fabrics and Fibers Company or equivalent should be used.

   c. **Geosynthetic Composite Drainnet (Geonet) Method:** Place a geosynthetic composite drain-net (geonet) directly against the wall or waterproofing system (as described below) to separate the backfill soil from the wall. The composite geonet should extend from the wall bottom to within 12 inches of the wall top. A geosynthetic composite drainnet such as Hydroduct 200 or Hydroduct 220 distributed by Grace Construction Products or equivalent should be used.

3. **Drainage Layer Collection and Discharge Pipes:** A minimum 4-inch diameter schedule 40, polyvinylchloride (PVC) perforated drainpipe should be placed at the wall base inside the geotextile wrapped drain rock or wrapped by the composite geonet. ¼–inch diameter perforations should be drilled into the pipe. The perforations should be oriented in cross section view at 90 degrees to one another and along the pipe length on 6-inch centers. The pipe should be placed such that the perforations are oriented 45 degrees from the vertical. A minimum of 3 inches of drain rock should be placed below the perforated PVC pipe. The pipe should direct water away from the wall by gravity with a minimum 1 percent slope. The pipe should collect groundwater collected by the drainage layer discharged to the surface at the end of the wall or through weep-hole penetrations through the wall.

4. **Backfill Placement and Compaction Equipment:** Heavy conventional motorized compaction equipment should not be used directly adjacent to a retaining wall unless the wall is designed with sufficient steel reinforcements and/or bracing to resist the additional lateral pressures. Compaction of backfill materials within 5 feet of the retaining wall should be accomplished by lightweight, hand-operated, walk-behind, vibratory equipment. Additionally, care should be taken during placement of the general backfill materials so as not to crush, tear or damage the waterproofing and/or drainage layer materials.

5. **Backfill Materials and Compaction:** The backfill material should be free draining and classified by the USCS as a coarse-grained material (i.e., GP, GW, GC, GM, SP, SW, SC, and SM). Materials classified by the USCS as a fine-grained material (i.e., CL, CH, ML, or MH) should not be used as retaining wall backfill. The retaining wall backfill material placed between the drainage layer and temporary cut-slope should be moisture conditioned to between ± 3 percentage points of the ASTM D1557 optimum moisture content and then compacted to a minimum of 90 percent and a maximum of 95 percent of the ASTM D1557 maximum dry density.

8.2.4 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, sidewalks and patios 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-15, Guide for Concrete Floor and Slab Construction, reported by ACI Committee 302.
- ASTM E1643-18a, Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.

- ASTM E1745-17, Standard Specifications for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs.
- ASTM F710-19, Standard Practice for Preparing Concrete Floors to Receive Resilient Flooring.

The interior building concrete slab-on-grade floor and exterior 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 professional 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 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 professional engineer.

   Rebar: As a minimum, use No. 3 rebar (ASTM A615/A615M-18e1 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 E1745-17), 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-cement 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 concrete slabs should use either crushed rock or Class II AB rock. The rock layer 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 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 assumed that the subsurface soil and groundwater conditions encountered at the location of the exploratory borings were 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.

APPENDIX A:

Important Information about This Geotechnical Engineering Report (Included with permission of GBA, Copyright 2019)

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
exposure to problems associated with subsurface conditions at project sites and development of them 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 herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation

everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed

and imes

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences 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.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- 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, the reliability of a geotechnical-engineering report can be 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 you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site 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.*

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

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement 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 through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are

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 judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed 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 continuing 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 available 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-phase observations.

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 information 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. 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. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. 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 provide 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 obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent 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:

Site Data Report

BCA

architecture
planning
interiors

Site Data Report – California Geological Survey
Ridgeview High School

Paradise Unified School District
5944 Maxwell Drive, Paradise, CA 95969

1. **Type of service:** High School

2. **Construction materials used for the project** - Concrete foundations, wood frame construction

3. **Type of construction** - New Classroom & Administration Building.

4. **Extent of construction for existing buildings** – None

5. **Seismic force resisting system used for each structure in the project** –
   a) New Building – bearing walls / light frame (wood) walls sheathed with wood structural panels.

6. **Foundation system that will be used for each structure in the project** -
   a) New Building – spread footings with concrete slab on grade.

7. **Analysis procedure used and basis of design** –
   a) New Buildings – analysis procedure is ASCE 7-16 equivalent lateral force procedure and the 2019 CBC code.

8. **Building characteristics such as number of stories above and below grade, foot print area at grade, grade slope on site**, etc.
   a) New Buildings – Single Story; 12,514 SF
   b) Site has slope from the street to the back of the property.

9. **Special features such as requirement for shoring, underpinning, retaining walls, etc.** –None.

Sincerely,
BCA Architects

Brian P. Whitmore, AIA
President

NV5

APPENDIX C:

Exploratory Boring Logs

EXPLORATORY BORING LOG

NV5

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. **B20-1**

Sheet: 1 **Of** 3

Project Name: Paradise USD - Ridgeview High School | **Project No.:** 70994.00 | **Task:** 001 | **Start Date:** 6-30-20

Location: Maxwell Drive and Pleasant Lane, Paradise, CA | **Estimated Ground Surface Elevation (Ft. AMSL):** 1895.00 | **Finish Date:** 6-30-20

Logged By: Santiago Carrillo | **Drilling Cmpny:** H1 Drilling | **Drill Rig Type:** CME-75

Driller: Rian Humphreys | **Drilling Method:** Hollow Stem Auger (HSA) | **Hammer Type:** 140 Pound Auto Trip Hammer

Boring Dia. (In.): 6.0 | **Total Depth (Ft.):** 47.5 | **Backfill or Well Design:** Neat Cement Grout

Ground Water Information

Date	6/30/20			
Time (24 Hour)	9:55			
Depth (Ft.)	45.0			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
8:15			HSA			0	3" Asphalt Concrete / 3" Aggregate Base Rock
					BLK-1	1	(SM) SILTY SAND, Fld. Est.: 60% Fine Sand and 40% High Plastic Clay-Silt Fines; Dark Reddish Brown (2.5YR 3/4); Loose; Moist. Severely Weathered; Decomposed Volcanic Rock.
8:18		5	2.5SS			2	
		3					
	1.0	5		0.3/1.5	L1-1-1	3	
			HSA			4	
8:24		4	2.5SS			5	
		9			L2-2-2		
	3.0	10		1.2/1.5	L2-1-2	6	Medium Dense
			HSA			7	
						8	
						9	
8:32		15	SPT			10	
		9					
	1.5	9		1.5/1.5	B1-1-1	11	Fld. Est.: 45% Fine Sand, 40% Low Plasticity Clay-Silt Fines, and 5% Gravel.
			HSA			12	
						13	
						14	
8:37		13	SPT			15	
		10					
		9		.1/1.5	B2-1-1	16	Yellowish Red (5YR 4/6); Very Moist
			HSA			17	
						18	
						19	(ML) SANDY SILT, Fld. Est.: 60% Low Plastic Clay-Silt Fines and 40% Fine Sand; Yellowish Red (5YR 4/6); Severely Weathered; Decomposed Volcanic Rock.
						20	

NOTES: HSA - Hollow Stem Augers
2.5SS - 2.5" Split Spoon Sampler
SPT - Standard Penetration Test

EXPLORATORY BORING LOG

NV5
48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B20-1
Sheet: 2 **Of** 3

Project Name: Paradise USD - Ridgeview High School
Project No.: 70994.00 **Task:** 001 **Start Date:** 6-30-20
Location: Maxwell Drive and Pleasant Lane, Paradise, CA
Estimated Ground Surface Elevation (Ft. AMSL): 1895.00 **Finish Date:** 6-30-20
Logged By: Santiago Carrillo **Drilling Cmpny:** H1 Drilling **Drill Rig Type:** CME-75
Driller: Rian Humphreys **Drilling Method:** Hollow Stem Auger (HSA) **Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 6.0 **Total Depth (Ft.):** 47.5 **Backfill or Well Design:** Neat Cement Grout

Ground Water Information

Date	6/30/20			
Time (24 Hour)	9:55			
Depth (Ft.)	45.0			

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
8:50		3	SPT			20	(ML) SANDY SILT, Fld. Est.: 60% Low Plastic Clay-Silt Fines and 40% Fine Sand; Yellowish Red (5YR 4/6); Stiff; Moist; Severely Weathered; Decomposed Volcanic Rock.
		4					
	2.0	3		1.5/1.5	B3-1-1	21	
			HSA			22–24	
8:55		4	SPT			25	(SM) SILTY SAND, Fld. Est.: 65% Fine Sand and 35% Low Plastic Clay-Silt Fines; Yellowish Red (5YR 4/6); Medium Dense; Moist; Severely Weathered; Decomposed Volcanic Rock.
		10					
	1.5	11		1.0/1.5	B4-1-1	26	
			HSA			27–29	
9:00		4	SPT			30	
		5					
	3.0	7		1.4/1.5	B5-1-1	31	White Mottling
			HSA			32–34	
			SPT			35	
9:10		8					
		6				36	
		15	HSA	0.1/1.5	B6-1-1	37–40	

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

NOTES: HSA - Hollow Stem Augers
2.5SS - 2.5" Split Spoon Sampler
SPT - Standard Penetration Test

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. **B20-1**

Project Name: Paradise USD - Ridgeview High School	**Project No.:** 70994.00	**Task:** 001	**Start Date:** 6-30-20
Location: Maxwell Drive and Pleasant Lane, Paradise, CA	**Estimated Ground Surface Elevation (Ft. AMSL):** 1895.00		**Finish Date:** 6-30-20
Logged By: Santiago Carrillo	**Drilling Cmpny:** H1 Drilling		**Drill Rig Type:** CME-75
Driller: Rian Humphreys	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 6.0	**Total Depth (Ft.):** 47.5	**Backfill or Well Design:** Neat Cement Grout	

Sheet: 3 **Of** 3

Ground Water Information

Date	6/30/20			
Time (24 Hour)	10:19			
Depth (Ft.)	45.0			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
9:30		5	SPT			40	(SM) SILTY SAND, Fld. Est.: 65% Fine Sand and 35% Low Plastic Clay-Silt Fines; Yellowish Red (5YR 4/6); Medium Dense; Moist; Severely Weathered; Decomposed Volcanic Rock.
		8					
	1.25	7		1.5/1.5	B7-1-1	41	
			HSA				
						42–44	
9:40		6	SPT	1.5/1.5	B8-1-1	45	(groundwater level symbol)
		6				46	
		7					
9:55		50/2"	SPT	0/1.5		47	(RX) ROCK, Gray (10YR 5/1); Competent; Slightly Weathered to Fresh
						48–60	

NOTES: HSA - Hollow Stem Augers
2.5SS - 2.5" Split Spoon Sampler
SPT - Standard Penetration Test

EXPLORATORY BORING LOG

N|V|5

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. **B20-2**

Sheet: 1 **Of** 1

Project Name: Paradise USD - Ridgeview High School	**Project No.:** 70994.00	**Task:** 001	**Start Date:** 6-30-20
Location: Maxwell Drive and Pleasant Lane, Paradise, CA	**Estimated Ground Surface Elevation (Ft. AMSL):** 1873.00		**Finish Date:** 6-30-20
Logged By: Santiago Carrillo	**Drilling Cmpny:** H1 Drilling		**Drill Rig Type:** CME-75
Driller: Rian Humphreys	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 6.0	**Total Depth (Ft.):** 20.0	**Backfill or Well Design:** Soil Cuttings	

Ground Water Information

Date	6/30/20			
Time (24 Hour)	12:00			
Depth (Ft.)	None			

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.
11:20			HSA		
11:22		10	2.5SS		
		14			
	4.5+	17		0.5/1.5	L1-1-1
			HSA		
					BLK-1
11:27		10	2.5SS		
		21			L2-2-2
	4.5+	32		0.6/1.5	L2-1-2
			HSA		
11:38		9	2.5SS		
	4.5+	21			L3-2-2
	4.5+	29		0.9/1.5	L3-1-2
			HSA		
11:48		15	2.5SS		
	2.5	38			L4-2-2
	4.5+	38		.85/1.5	L4-1-2
			HSA		
11:59		7	2.5SS		
		8			
12:00	3.5	14		1.3/1.5	B1-1-1

Depth B.G.S. (Ft.): 0 – 20

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

3" Asphalt Concrete / 3" Aggregate Base Rock

(SM) SILTY SAND, Fld. Est.: 60% Fine Sand and 40% High Plastic Clay-Silt Fines; Dark Reddish Brown (2.5YR 3/4); Medium Dense; Moist. Severely Weathered; Decomposed Volcanic Rock.

NOTES: HSA - Hollow Stem Augers
2.5SS - 2.5" Split Spoon Sampler

EXPLORATORY BORING LOG

48 BELLARMINE COURT, SUITE 40, CHICO, CA., 95928
PHONE: 530-894-2487, FAX: 530-894-2437

Boring No. B20-3
Sheet: 1 **Of** 1

Project Name: Paradise USD - Ridgeview High School	**Project No.:** 70994.00	**Task:** 001	**Start Date:** 6-30-20
Location: Maxwell Drive and Pleasant Lane, Paradise, CA	**Estimated Ground Surface Elevation (Ft. AMSL):** 1878.00		**Finish Date:** 6-30-20
Logged By: Santiago Carrillo	**Drilling Cmpny:** H1 Drilling	**Drill Rig Type:** CME-75	
Driller: Rian Humphreys	**Drilling Method:** Hollow Stem Auger (HSA)	**Hammer Type:** 140 Pound Auto Trip Hammer	
Boring Dia. (In.): 6.0	**Total Depth (Ft.):** 18.5	**Backfill or Well Design:** Soil Cuttings	

Ground Water Information

Date	6/30/20			
Time (24 Hour)	13:36			
Depth (Ft.)	None			

Soil And/Or Rock Material Descriptions

SOIL: USCS Symbol; Name; Particle Size Gradation %; Munsel Color; Density/Consistency; Moisture; Odor; Organics; Cementation; Texture; Refuse; Etc.
ROCK: Unit Name; Lithology; Munsel Color; Cementation; Weathering; Competency; Bedding/Foliation; Fracture/Joint Spacing & Roughness; RQD; Moisture.

24 Hour Clock Time (HH:MM)	Pocket Penetrometer (TSF)	Uncorrected Blow Counts (Blows / 6-inch)	Drilling Method and/or Sampler Type	Sample Recovery (Ft./Ft.)	Sample No.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
13:08			HSA			0	3" Asphalt Concrete / 3" Aggregate Base Rock
13:10		5	2.5SS			1	(SM) SILTY SAND, Fld. Est.: 60% Fine Sand and 40% High Plastic Clay-Silt Fines; Dark Reddish Brown (2.5YR 3/4); Medium Dense; Moist. Severely Weathered; Decomposed Volcanic Rock.
		5			L1-2-2		
	1.5	6		0.85/1.5	L1-1-2	2	
			HSA				
					BLK-1	3	
13:13		5	2.5SS				
		13			L2-2-2	4	
		13		0.8/1.5	L2-1-2		
			HSA			5	
						6	
						7	
						8	
13:22		6	2.5SS				
	2.0	12			L3-2-2	9	
	3.25	14		1.0/1.5	L3-1-2		
			HSA			10	
						11	Yellowish Brown (5YR 4/6)
						12	
						13	
13:28		4	2.5SS				
		20			L4-2-2	14	Gray (10YR 5/1); less weathered
		19		1.4/1.5	L4-1-2		
			HSA			15	
						16	
						17	Hard Drilling
						18	
13:36		50/1"	SPT	0/1.5			
						19	
						20	

NOTES: HSA - Hollow Stem Augers
2.5SS - 2.5" Split Spoon Sampler
SPT - Standard Penetration Test

APPENDIX D:

Soil Laboratory Test Results

ATTERBERG INDICES

ASTM D4318

DSA LEA No. 284

DSA File No. N/A
DSA App No. N/A

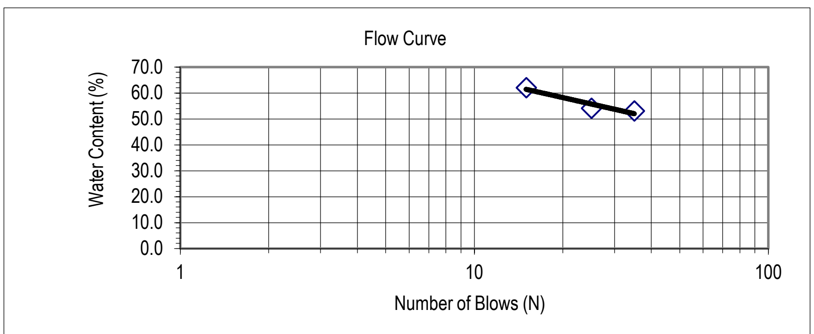
Project No.	70994.00.001	Project Name	PUSD Ridgeview HS			Date:	07/06/20
Sample No.	BLK-1	Boring/Trench	B20-1	Depth, (ft.):	1-4	Tested By:	LGH
Description:	(SM) Silty Sand, Dark Reddish Brown (2.5YR 3/4)					Checked By:	DJP
Sample Location:						Lab. No.	C20-122

Estimated % of Sample Retained on No. 40 Sieve:
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:	Z	W	V			X	D	
Wt. Pan (gr)	37.44	37.76	37.34			38.21	38.27	
Wt. Wet Soil + Pan (	45.29	46.76	45.25			45.62	45.51	
Wt. Dry Soil + Pan (	42.57	43.60	42.22			43.80	43.71	
Wt. Water (gr)	2.72	3.16	3.03			1.82	1.80	
Wt. Dry Soil (gr)	5.13	5.84	4.88			5.59	5.44	
Water Content (%)	53.0	54.1	62.1			32.6	33.1	
Number of Blows, N	35	25	15					

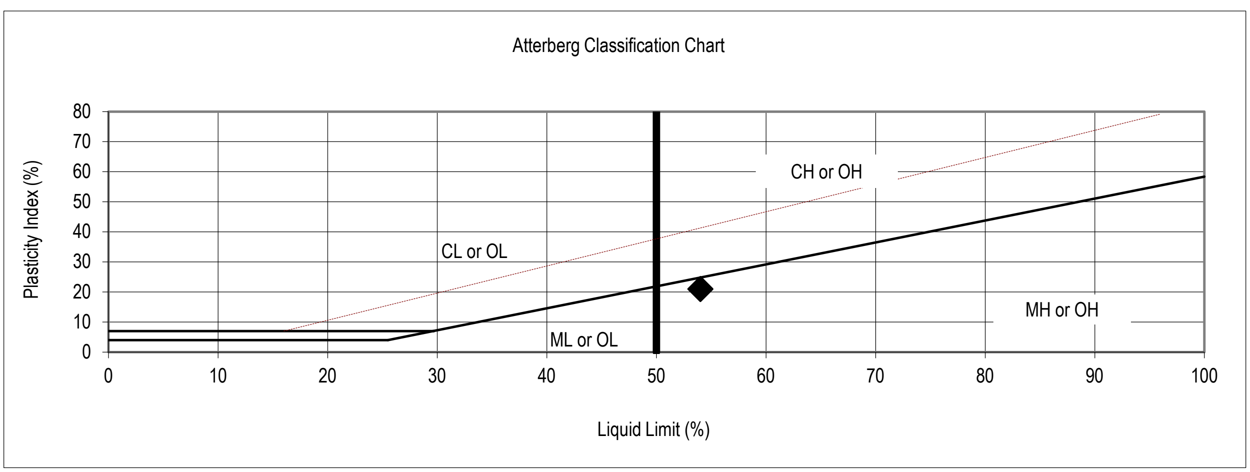
LIQUID LIMIT = 54

PLASTIC LIMIT = 33



Plasticity Index = 21

Group Symbol = MH



PARTICLE SIZE DISTRIBUTION TEST WORK SHEET

ASTM D422, C136

DSA File No. N/A

DSA App No. N/A

DSA LEA No. 284

Sieve Only Analysis Worksheet

Project No.	70994.00.001	Project Name:	PUSD Ridgeview HS			Date:	07/06/20
Sample No.	BLK-1	Boring/Trench:	B20-1	Depth, (ft.):	1-4	Tested By:	LGH
Description:	(SM) Silty Sand, Dark Reddish Brown (2.5YR 3/4)					Checked By:	DJP
Sample Location:						Lab. No.	C20-122

Moisture Content Data:

Pan ID		
Pan Weight		(gm)
Wet Soil + Pan		(gm)
Dry Soil + Pan		(gm)
Water Weight	0.00	(gm)
Dry Soil Weight	0.00	(gm)
Moisture Content	0.0	(%)

Total Material Sample Data:

Pan ID		
Pan Weight		(gm)
Wet Soil + Pan Wt.	3,098.50	(gm)
Total Wet Weight	3,098.50	(gm)
Total Dry Weight	3,098.50	(gm)
Total Dry Wt. >#4 Sieve	73.70	(gm)
Total Dry Wt.<#4 Sieve	3,024.80	(gm)
Total Dry Wt. <#200 Sieve	708.45	(gm)
Total Percent <#200 Sieve	22.86	(%)

GRAVEL PORTION SIEVE ANALYSIS
(Portion Retained On > #4 Sieve)

Sieve Size	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Wet Weight Retained On Sieve (gm)	Dry Weight Retained On Sieve (gm)	Dry Weight Accum. On Sieve (gm)	Dry Weight Passing Sieve (gm)	Dry Weight Percent Passing (%)
6 Inch	6.0000	152.40		0.00	0.00	3,098.50	100.0
3 Inch	3.0000	76.20		0.00	0.00	3,098.50	100.0
2 Inch	2.0000	50.80		0.00	0.00	3,098.50	100.0
1.5 Inch	1.5000	38.10		0.00	0.00	3,098.50	100.0
1.0 Inch	1.0000	25.40	0.00	0.00	0.00	3,098.50	100.0
3/4 Inch	0.7500	19.05	33.50	33.50	33.50	3,065.00	98.9
1/2 Inch	0.5000	12.70	3.30	3.30	36.80	3,061.70	98.8
3/8 Inch	0.3750	9.53	3.20	3.20	40.00	3,058.50	98.7
#4	0.1870	4.75	33.70	33.70	73.70	3,024.80	97.6
PAN			3,024.80	3,024.80			

SAND PORTION SIEVE ANALYSIS
(Portion Retained On < #4 Sieves)

Representative Sample Data:

Pan ID		
Pan Weight		(gm)
Wet Soil + Pan	348.40	(gm)
Wet Soil	348.40	(gm)
Dry Soil	348.40	(gm)

#200 Wash Data:

Portion >#200 Sieve:	266.80	(gm)
Portion <#200 Sieve:	81.60	(gm)
Percent <#200 Sieve	23.42	(%)
Total Wt. <#200 Sieve	708.45	(gm)

Sieve Size	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight Rep. Sample Retained On Sieve (gm)	Dry Weight Rep. Sample Percent Retained (%)	Total Sample Weight Retained (gm)	Accum. Grand Total On Sieve (gm)	Total Percent Passing (%)
#10	0.079	2.000	7.4	2.12	64.25	137.95	95.5
#20	0.033	0.850	23.60	6.77	204.89	342.84	88.9
#40	0.017	0.425	39.00	11.19	338.60	681.44	78.0
#60	0.010	0.250	50.70	14.55	440.18	1,121.61	63.8
#100	0.006	0.150	64.30	18.46	558.25	1,679.87	45.8
#200	0.003	0.075	81.80	23.48	710.19	2,390.05	22.9
PAN			Discard				

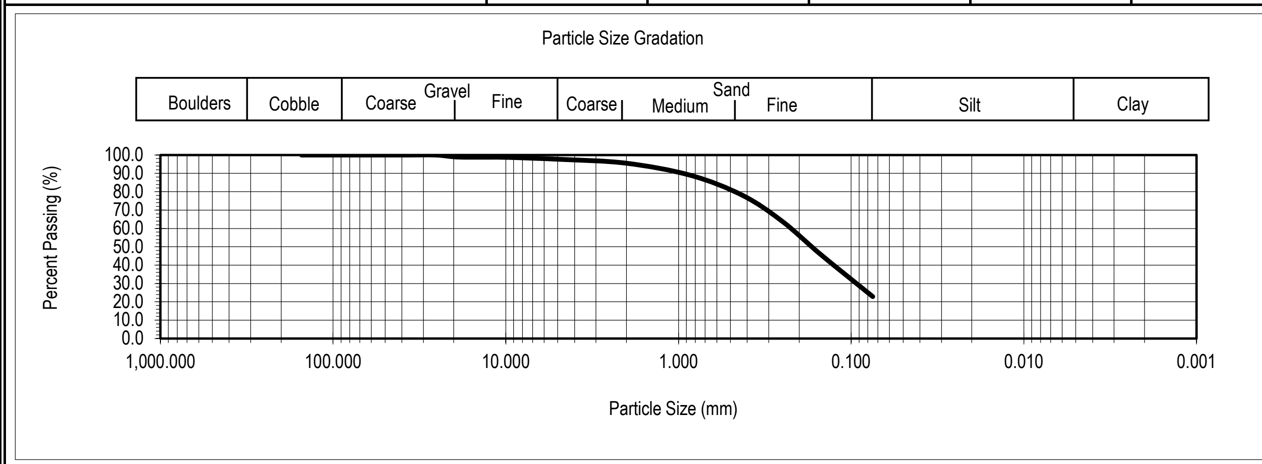
DSA LEA No.: 284

PARTICLE SIZE DISTRIBUTION

ASTM D422, C136

Project No. 70994.00.001	Project Name: PUSD Ridgeview HS		Date: 7/6/2020
Sample No. BLK-1	Boring/Trench: B20-1	Depth, (ft.): 1-4	Tested By: LGH
Description: (SM) Silty Sand, Dark Reddish Brown (2.5YR 3/4)			Checked By: DJP
Sample Location:			Lab. No. C20-122

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	3,098.5	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,098.5	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,098.5	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	3,098.5	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	3,098.5	**100.0**
3/4 Inch	0.7500	19.1	33.50	33.5	3,065.0	**98.9**
1/2 Inch	0.5000	12.7	3.30	36.8	3,061.7	**98.8**
3/8 Inch	0.3750	9.5	3.20	40.0	3,058.5	**98.7**
#4	0.1870	4.7500	33.70	73.7	3,024.8	**97.6**
#10	0.0790	2.0066	64.25	137.9	2,960.6	**95.5**
#20	0.0335	0.8500	204.89	342.8	2,755.7	**88.9**
#40	0.0167	0.4250	338.60	681.4	2,417.1	**78.0**
#60	0.0098	0.2500	440.18	1,121.6	1,976.9	**63.8**
#100	0.0059	0.1500	558.25	1,679.9	1,418.6	**45.8**
#200	0.0030	0.0750	710.19	2,390.1	708.4	**22.9**
Hydrometer						



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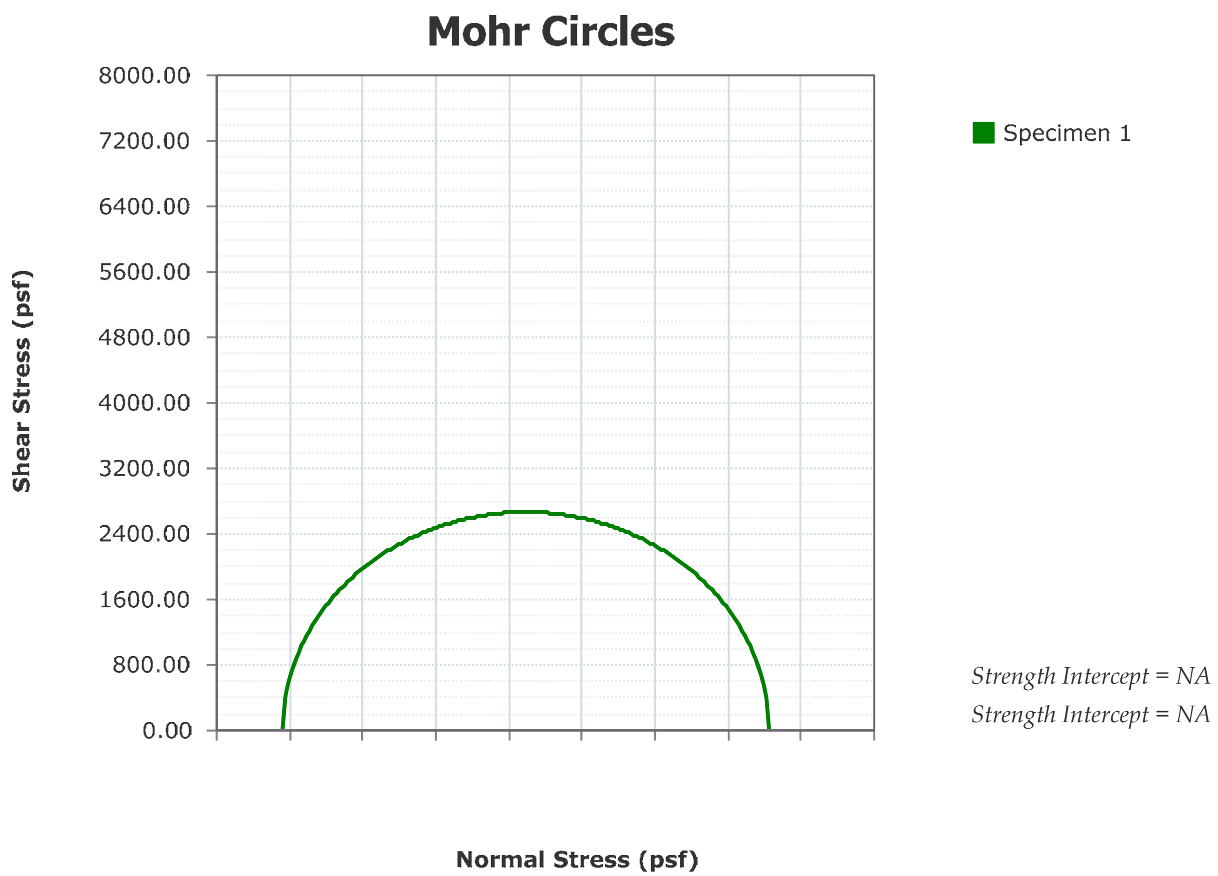
Unconsolidated Undrained Test

ASTM D2850



Project:	PUSD - Ridgeview HS
Project Number:	70994.00.001
Sampling Date:	
Sample Number:	L2-1-2
Sample Depth:	5 ft
Location:	B20-1
Client Name:	Paradise USD
Remarks:	

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Unconsolidated Undrained Test

ASTM D2850

Before Test	Specimen Number 1	2	3	4	5	6	7	8
Membrane Thickness (in)	0.001							
Initial Cell Pressure (psi)	5.0							
Height (in)	6.034							
Diameter (in)	2.373							
Water Content (%)	30.6							
Wet Density (Units)	113.4							
Dry Density (pcf)	86.9							
Degree of Saturation (%)	87.2							
Void Ratio	0.955							
Height To Diameter Ratio	2.543							
Test Data	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Comp. Strength at Failure (psf)	5277.22							
σ1 at Failure (psf)	5997.22							
σ3 at Failure (psf)	720.00							
Rate of Strain (in/min)	0.120680							
Axial Strain at Failure (%)	14.67							
After Test	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Final Water Content (%)	34.2							

Project:	PUSD - Ridgeview HS
Project Number:	70994.00.001
Sampling Date:	
Sample Number:	L2-1-2
Sample Depth:	5 ft
Location:	B20-1
Client Name:	Paradise USD
Project Remarks:	

Specimen 1 Failure Sketch

Specimen 2 Failure Sketch

Specimen 3 Failure Sketch

Specimen 4 Failure Sketch

Specimen 5 Failure Sketch

Specimen 6 Failure Sketch

Specimen 7 Failure Sketch

Specimen 8 Failure Sketch

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Unconsolidated Undrained Test

ASTM D2850

Specimen 1

Test Description:	Unconsolidated Undrained Triaxial		
Other Associated Tests:			
Device Details:			
Test Specification:			
Test Time:	7/14/2020		
Technician:	SC	Sampling Method:	Undisturbed
Specimen Code:		Specimen Lab #:	
Specimen Description:			
Specific Gravity:	2.720		
Plastic Limit:	0	Liquid Limit:	0
Height (in):	6.034	Diameter (in):	2.373
Area (in^2):	4.423	Volume (in^3):	26.69
Large Particle:			
Moisture Material:	Specimen		
Moist Weight (g):	794.7		
Test Remarks:			

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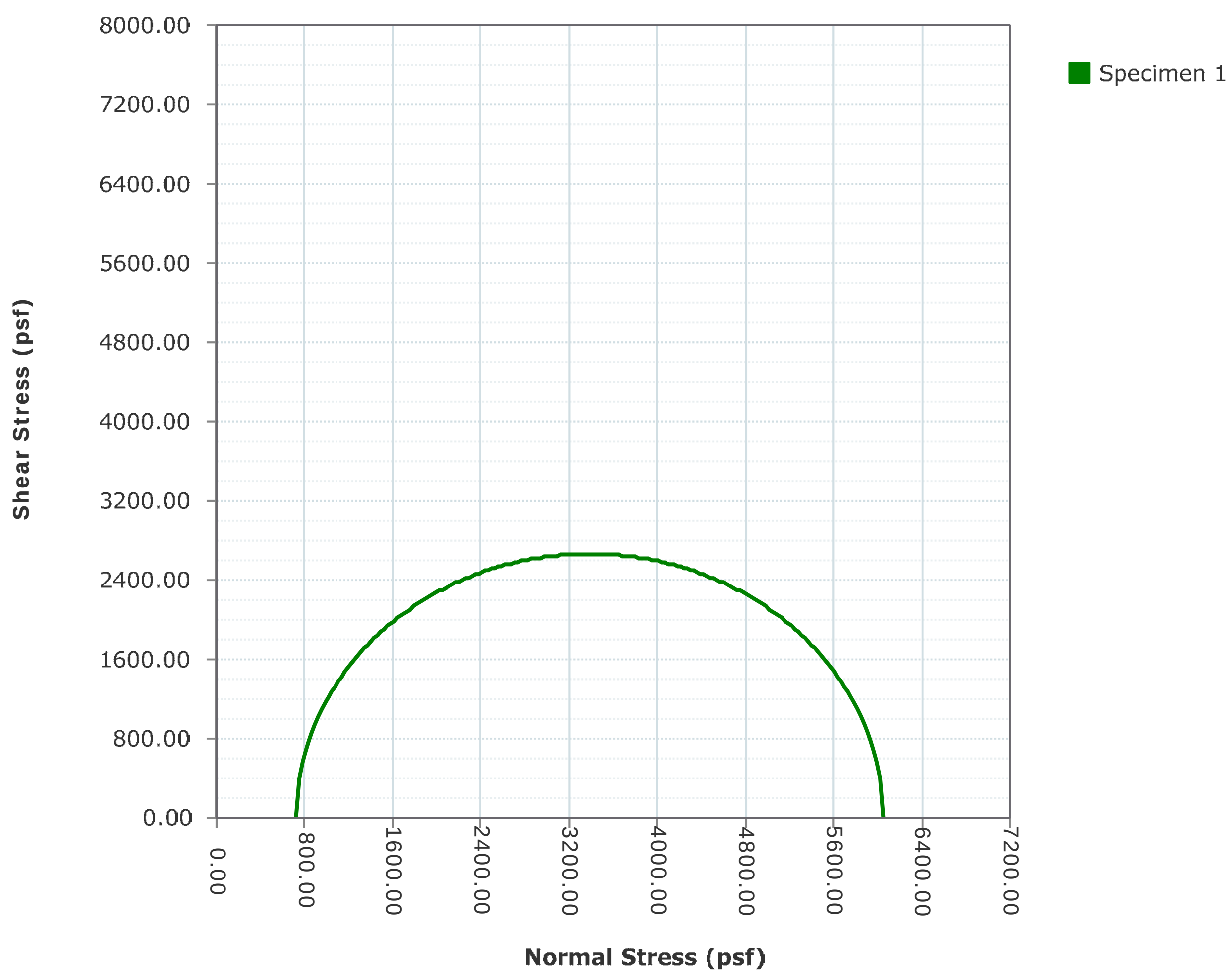
Mohr Circles (Total Stress) Graph

ASTM D2850



Tangent Results	
Strength Intercept (psi)	NA
Friction Angle (°)	NA

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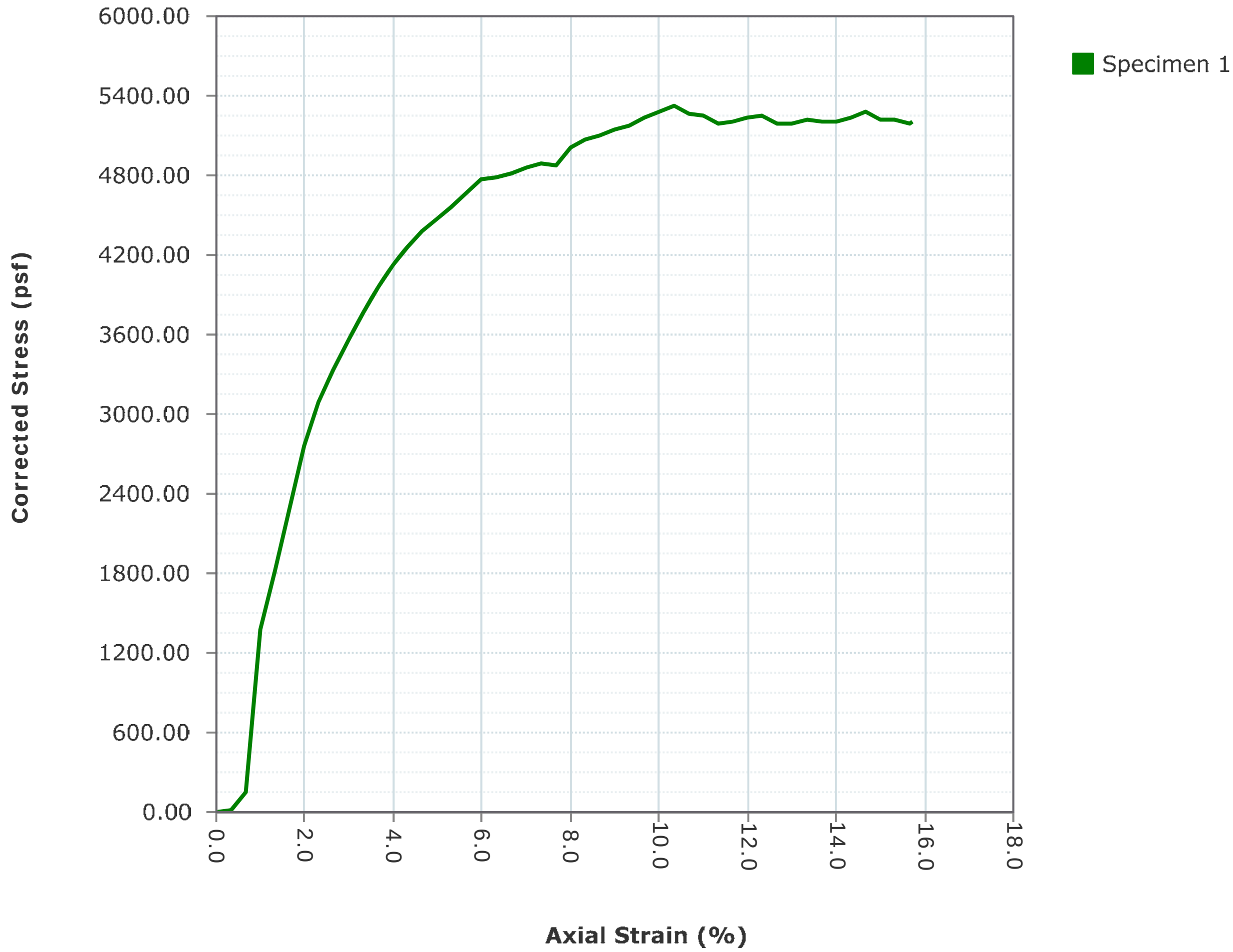
Stress-Strain Graph

ASTM D2850



Unconsolidated Undrained Test - Tabulated Data - Specimen 1

ASTM D2850

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Disp. (in)	Corrected Load (Lbf)	Corrected Disp. (in)	Corrected Area (in²)	Axial Strain (%)	Stress (psf)	Corrected Compressive Stress (psf)	σ1 (psf)	σ3 (psf)	σ1 / σ3	p (psf)	q (psf)
0	00:00:00	1.1	0.0002	0.0	0.000	4.423	0.0	0.00	0.00	720.00	720.00	1.000	720.00	0.00
1	00:00:10	1.3	0.0204	0.3	0.020	4.438	0.3	8.70	8.51	728.51	720.00	1.012	724.25	4.25
2	00:00:20	5.5	0.0404	4.5	0.040	4.452	0.7	145.55	144.26	864.26	720.00	1.200	792.13	72.13
3	00:00:30	43.9	0.0603	42.8	0.060	4.467	1.0	1,394.71	1,380.81	2,100.81	720.00	2.918	1,410.41	690.41
4	00:00:40	57.4	0.0800	56.3	0.080	4.482	1.3	1,834.17	1,809.89	2,529.89	720.00	3.514	1,624.95	904.95
5	00:00:50	72.9	0.1002	71.8	0.100	4.497	1.7	2,337.91	2,299.15	3,019.15	720.00	4.193	1,869.57	1,149.57
6	00:01:00	87.4	0.1198	86.3	0.120	4.512	2.0	2,810.74	2,755.03	3,475.03	720.00	4.826	2,097.51	1,377.51
7	00:01:10	98.1	0.1394	97.0	0.139	4.527	2.3	3,158.62	3,085.75	3,805.75	720.00	5.286	2,262.88	1,542.88
8	00:01:20	106.2	0.1592	105.1	0.159	4.542	2.6	3,422.66	3,332.47	4,052.47	720.00	5.628	2,386.23	1,666.23
9	00:01:30	113.5	0.1799	112.4	0.180	4.558	3.0	3,661.22	3,552.16	4,272.16	720.00	5.934	2,496.08	1,776.08
10	00:01:40	120.5	0.2006	119.4	0.200	4.575	3.3	3,888.78	3,759.59	4,479.59	720.00	6.222	2,599.79	1,879.79
11	00:01:50	127.2	0.2211	126.1	0.221	4.591	3.7	4,106.42	3,956.04	4,676.04	720.00	6.494	2,698.02	1,978.02
12	00:02:00	132.9	0.2411	131.8	0.241	4.607	4.0	4,290.48	4,119.18	4,839.18	720.00	6.721	2,779.59	2,059.59
13	00:02:10	137.9	0.2606	136.8	0.260	4.622	4.3	4,454.83	4,262.51	4,982.51	720.00	6.920	2,851.26	2,131.26
14	00:02:20	142.0	0.2805	140.9	0.280	4.638	4.6	4,587.22	4,374.10	5,094.10	720.00	7.075	2,907.05	2,187.05
15	00:02:30	145.7	0.3005	144.6	0.300	4.654	5.0	4,707.61	4,473.29	5,193.29	720.00	7.213	2,956.65	2,236.65
16	00:02:40	148.9	0.3200	147.8	0.320	4.670	5.3	4,811.39	4,556.34	5,276.34	720.00	7.328	2,998.17	2,278.17
17	00:02:50	153.2	0.3404	152.1	0.340	4.687	5.6	4,951.65	4,672.45	5,392.45	720.00	7.490	3,056.23	2,336.23
18	00:03:00	156.7	0.3609	155.6	0.361	4.704	6.0	5,067.56	4,764.63	5,484.63	720.00	7.618	3,102.32	2,382.32
19	00:03:10	158.0	0.3815	156.9	0.381	4.721	6.3	5,109.46	4,786.54	5,506.54	720.00	7.648	3,113.27	2,393.27
20	00:03:20	159.5	0.4025	158.4	0.402	4.739	6.7	5,156.83	4,812.99	5,532.99	720.00	7.685	3,126.50	2,406.50
21	00:03:30	161.7	0.4233	160.7	0.423	4.756	7.0	5,230.84	4,864.04	5,584.04	720.00	7.756	3,152.02	2,432.02
22	00:03:40	163.0	0.4431	161.9	0.443	4.773	7.3	5,272.26	4,885.25	5,605.25	720.00	7.785	3,162.62	2,442.62
23	00:03:50	163.5	0.4631	162.4	0.463	4.790	7.7	5,287.21	4,881.53	5,601.53	720.00	7.780	3,160.76	2,440.76
24	00:04:00	168.3	0.4829	167.3	0.483	4.807	8.0	5,445.58	5,009.88	5,729.88	720.00	7.958	3,224.94	2,504.94
25	00:04:10	171.2	0.5027	170.1	0.503	4.824	8.3	5,538.45	5,077.20	5,797.20	720.00	8.052	3,258.60	2,538.60

Unconsolidated Undrained Test - Tabulated Data - Specimen 1

ASTM D2850

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Disp. (in)	Corrected Load (Lbf)	Corrected Disp. (in)	Corrected Area (in²)	Axial Strain (%)	Stress (psf)	Corrected Compressive Stress (psf)	σ1 (psf)	σ3 (psf)	σ1 — σ3	p (psf)	q (psf)
26	00:04:20	172.5	0.5224	171.4	0.522	4.842	8.7	5,582.00	5,098.90	5,818.90	720.00	8.082	3,269.45	2,549.45
27	00:04:30	174.8	0.5428	173.7	0.543	4.860	9.0	5,656.92	5,148.17	5,868.17	720.00	8.150	3,294.09	2,574.09
28	00:04:40	176.6	0.5630	175.5	0.563	4.878	9.3	5,714.42	5,181.41	5,901.41	720.00	8.196	3,310.71	2,590.71
29	00:04:50	179.1	0.5834	178.1	0.583	4.896	9.7	5,797.24	5,236.86	5,956.86	720.00	8.273	3,338.43	2,618.43
30	00:05:00	181.2	0.6039	180.1	0.604	4.914	10.0	5,864.17	5,277.38	5,997.38	720.00	8.330	3,358.69	2,638.69
31	00:05:10	183.7	0.6242	182.6	0.624	4.933	10.3	5,945.02	5,330.21	6,050.21	720.00	8.403	3,385.10	2,665.10
32	00:05:20	182.2	0.6441	181.1	0.644	4.951	10.7	5,896.35	5,267.12	5,987.12	720.00	8.315	3,353.56	2,633.56
33	00:05:30	182.5	0.6642	181.4	0.664	4.970	11.0	5,906.67	5,256.65	5,976.65	720.00	8.301	3,348.32	2,628.32
34	00:05:40	180.9	0.6841	179.8	0.684	4.988	11.3	5,854.34	5,190.75	5,910.75	720.00	8.209	3,315.38	2,595.38
35	00:05:50	182.2	0.7039	181.2	0.704	5.007	11.7	5,898.60	5,210.69	5,930.69	720.00	8.237	3,325.35	2,605.35
36	00:06:00	183.6	0.7236	182.5	0.723	5.025	12.0	5,942.52	5,230.10	5,950.10	720.00	8.264	3,335.05	2,615.05
37	00:06:10	184.8	0.7438	183.7	0.744	5.044	12.3	5,981.46	5,244.31	5,964.31	720.00	8.284	3,342.15	2,622.15
38	00:06:20	183.5	0.7641	182.4	0.764	5.064	12.7	5,940.28	5,188.22	5,908.22	720.00	8.206	3,314.11	2,594.11
39	00:06:30	184.1	0.7847	183.0	0.784	5.084	13.0	5,958.05	5,183.43	5,903.43	720.00	8.199	3,311.71	2,591.71
40	00:06:40	186.3	0.8052	185.2	0.805	5.104	13.3	6,030.27	5,225.73	5,945.73	720.00	8.258	3,332.86	2,612.86
41	00:06:50	186.3	0.8256	185.2	0.825	5.124	13.7	6,030.60	5,205.62	5,925.62	720.00	8.230	3,322.81	2,602.81
42	00:07:00	187.2	0.8452	186.1	0.845	5.143	14.0	6,058.44	5,209.95	5,929.95	720.00	8.236	3,324.98	2,604.98
43	00:07:10	188.8	0.8651	187.8	0.865	5.163	14.3	6,113.38	5,237.04	5,957.04	720.00	8.274	3,338.52	2,618.52
44	00:07:20	191.0	0.8851	189.9	0.885	5.183	14.7	6,184.17	5,277.22	5,997.22	720.00	8.329	3,358.61	2,638.61
45	00:07:30	189.4	0.9050	188.3	0.905	5.203	15.0	6,132.17	5,212.59	5,932.59	720.00	8.240	3,326.30	2,606.30
46	00:07:40	190.4	0.9248	189.3	0.925	5.223	15.3	6,162.91	5,218.52	5,938.52	720.00	8.248	3,329.26	2,609.26
47	00:07:50	190.3	0.9450	189.2	0.945	5.244	15.7	6,161.59	5,196.77	5,916.77	720.00	8.218	3,318.39	2,598.39
48	00:07:52	190.6	0.9489	189.5	0.949	5.248	15.7	6,171.14	5,200.80	5,920.80	720.00	8.223	3,320.40	2,600.40

ATTERBERG INDICES

ASTM D4318

DSA LEA No. 284

DSA File No.	N/A
DSA App No.	N/A

Project No.	70994.00.001	Project Name	PUSD Ridgeview HS			Date:	07/06/20
Sample No.	B3-1-1	Boring/Trench	B20-1	Depth, (ft.):	20	Tested By:	LGH
Description:	(ML) Sandy Silt, Yellowish Red (5YR 4/6)					Checked By:	DJP
Sample Location:						Lab. No.	C20-122

Estimated % of Sample Retained on No. 40 Sieve:

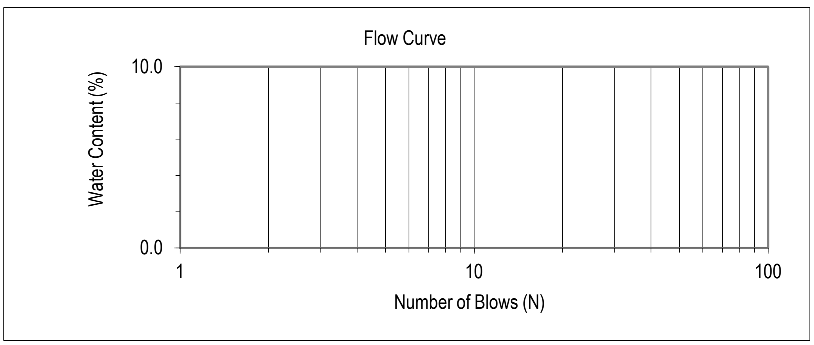
Sample Air Dried: yes

Test Method A or B: A

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:								
Wt. Pan (gr)								
Wt. Wet Soil + Pan (gr)								
Wt. Dry Soil + Pan (gr)								
Wt. Water (gr)								
Wt. Dry Soil (gr)								
Water Content (%)								
Number of Blows, N								

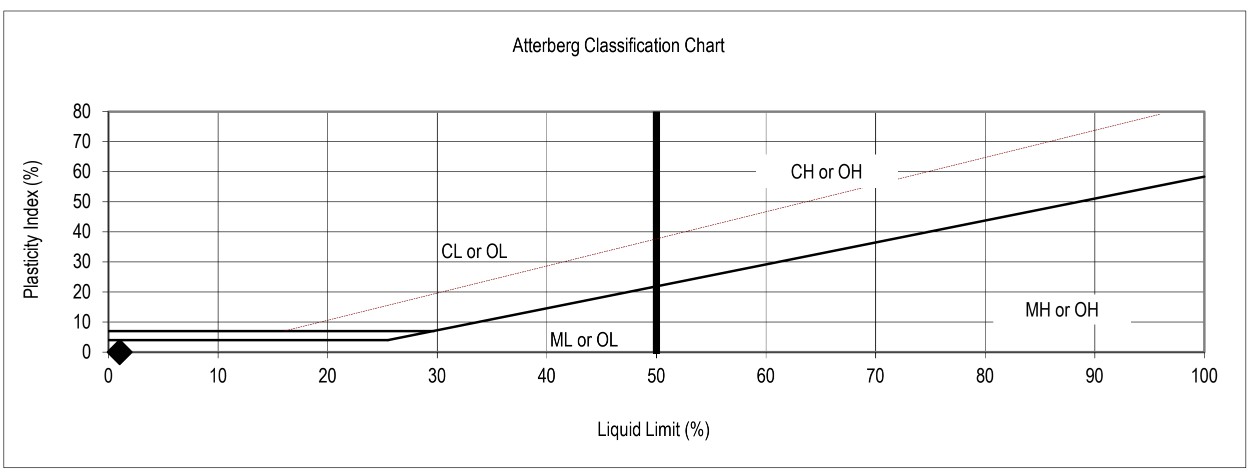
LIQUID LIMIT = NP

PLASTIC LIMIT = #VALUE!



Plasticity Index = #VALUE!

Group Symbol = NP



PARTICLE SIZE DISTRIBUTION TEST WORK SHEET

ASTM D422, C136

DSA File No. N/A
DSA LEA No. 284 **DSA App No.** N/A

Sieve Only Analysis Worksheet

Project No.	70994.00.001	Project Name:	PUSD Ridgeview HS			Date:	07/06/20
Sample No.	B3-1-1	Boring/Trench:	B20-1	Depth, (ft.):	20	Tested By:	LGH
Description:	(ML) Sandy Silt, Yellowish Red (5YR 4/6)					Checked By:	DJP
Sample Location:						Lab. No.	C20-122

Moisture Content Data:			Total Material Sample Data:		
			Pan ID		
			Pan Weight		(gm)
Pan ID			Wet Soil + Pan Wt.	301.10	(gm)
Pan Weight		(gm)	Total Wet Weight	301.10	(gm)
Wet Soil + Pan		(gm)	Total Dry Weight	301.10	(gm)
Dry Soil + Pan		(gm)	Total Dry Wt. >#4 Sieve	0.00	(gm)
Water Weight	0.00	(gm)	Total Dry Wt.<#4 Sieve	301.10	(gm)
Dry Soil Weight	0.00	(gm)	Total Dry Wt. <#200 Sieve	164.50	(gm)
Moisture Content	0.0	(%)	Total Percent <#200 Sieve	54.63	(%)

GRAVEL PORTION SIEVE ANALYSIS
(Portion Retained On > #4 Sieve)

Sieve Size	Particle Diameter		Wet Weight	Dry Weight			
	Inches (in.)	Millimeter (mm)	Retained On Sieve (gm)	Retained On Sieve (gm)	Accum. On Sieve (gm)	Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.40		0.00	0.00	301.10	100.0
3 Inch	3.0000	76.20		0.00	0.00	301.10	100.0
2 Inch	2.0000	50.80		0.00	0.00	301.10	100.0
1.5 Inch	1.5000	38.10		0.00	0.00	301.10	100.0
1.0 Inch	1.0000	25.40		0.00	0.00	301.10	100.0
3/4 Inch	0.7500	19.05		0.00	0.00	301.10	100.0
1/2 Inch	0.5000	12.70		0.00	0.00	301.10	100.0
3/8 Inch	0.3750	9.53		0.00	0.00	301.10	100.0
#4	0.1870	4.75		0.00	0.00	301.10	100.0
PAN			301.10	301.10			

SAND PORTION SIEVE ANALYSIS
(Portion Retained On < #4 Sieves)
Representative Sample Data:

			#200 Wash Data:		
Pan ID					
Pan Weight		(gm)	Portion >#200 Sieve:	136.60	(gm)
Wet Soil + Pan	301.10	(gm)	Portion <#200 Sieve:	164.50	(gm)
Wet Soil	301.10	(gm)	Percent <#200 Sieve	54.63	(%)
Dry Soil	301.10	(gm)	Total Wt. <#200 Sieve	164.50	(gm)

Sieve Size	Particle Diameter		Dry Weight Rep. Sample		Total Sample	Accum.	Total
	Inches (in.)	Millimeter (mm)	Retained On Sieve (gm)	Percent Retained (%)	Weight Retained (gm)	Grand Total On Sieve (gm)	Percent Passing (%)
#10	0.079	2.000	6.9	2.29	6.90	6.90	97.7
#20	0.033	0.850	29.80	9.90	29.80	36.70	87.8
#40	0.017	0.425	32.60	10.83	32.60	69.30	77.0
#60	0.010	0.250	21.50	7.14	21.50	90.80	69.8
#100	0.006	0.150	22.40	7.44	22.40	113.20	62.4
#200	0.003	0.075	23.40	7.77	23.40	136.60	54.6
PAN			Discard				

PARTICLE SIZE DISTRIBUTION

ASTM D422, C136

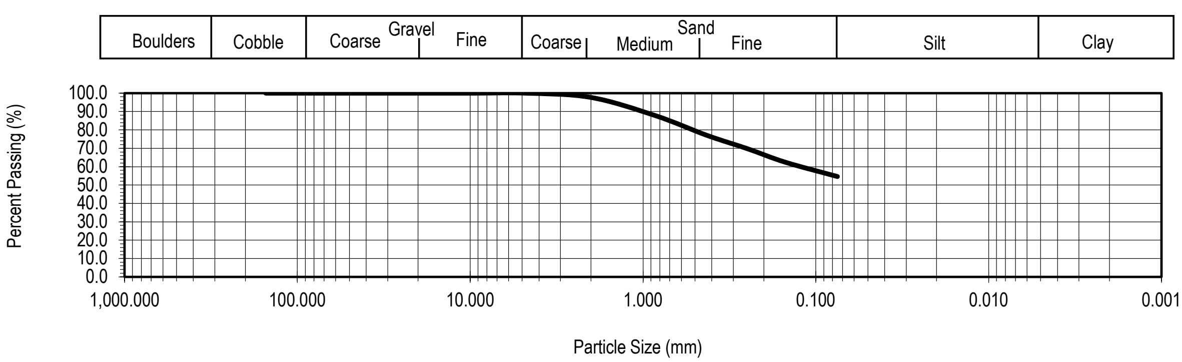
DSA LEA No.: 284

Project No.	**70994.00.001**	Project Name:	**PUSD Ridgeview HS**	Date:	**7/6/2020**
Sample No.	**B3-1-1**	Boring/Trench:	**B20-1** Depth, (ft.): **20**	Tested By:	**LGH**
Description:	**(ML) Sandy Silt, Yellowish Red (5YR 4/6)**			Checked By:	**DJP**
Sample Location:				Lab. No.	**C20-122**

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve: Retained On Sieve (gm)	Dry Weight on Sieve: Accumulated On Sieve (gm)	Dry Weight on Sieve: Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	301.1	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	301.1	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	301.1	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	301.1	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	301.1	**100.0**
3/4 Inch	0.7500	19.1	0.00	0.0	301.1	**100.0**
1/2 Inch	0.5000	12.7	0.00	0.0	301.1	**100.0**
3/8 Inch	0.3750	9.5	0.00	0.0	301.1	**100.0**
#4	0.1870	4.7500	0.00	0.0	301.1	**100.0**
#10	0.0790	2.0066	6.90	6.9	294.2	**97.7**
#20	0.0335	0.8500	29.80	36.7	264.4	**87.8**
#40	0.0167	0.4250	32.60	69.3	231.8	**77.0**
#60	0.0098	0.2500	21.50	90.8	210.3	**69.8**
#100	0.0059	0.1500	22.40	113.2	187.9	**62.4**
#200	0.0030	0.0750	23.40	136.6	164.5	**54.6**

Hydrometer

Particle Size Gradation



MOISTURE & DENSITY

ASTM D2216, D2937, C566

DSA LEA No. 284

DSA File No. N/A
DSA App No. N/A

Project No. 70994.00.001 | Project Name: PUSD Ridgeview HS | Date: 07/06/20

Tested By: LGH
Checked By: DJP
Lab. No. C20-122

	Units								
SAMPLE LOCATION DATA									
Boring/Trench No.	Units	B20-2							
Sample No.		L3-1-2							
Depth Interval	(ft.)	8.5							
Sample Description		Silty Sand, Dark Reddish Brown (2.5YR 3/4)							
USCS Symbol		SM							
SAMPLE DIMENSION AND WEIGHT DATA									
Sample Length	(in)	5.881							
Sample Diameter	(in)	2.349							
Sample Volume	(cf)	0.0147							
Wet Soil + Tube Wt.	(gr)	1019.20							
Tube Wt.	(gr)	275.30							
Wet Soil Wt.	(gr)	743.90							
MOISTURE CONTENT DATA									
Tare No.		ZZ-8							
Tare Wt.	(gr)	173.90							
Wet Soil + Tare Wt.	(gr)	916.70							
Dry Soil + Tare Wt.	(gr)	762.90							
Water Wt.	(gr)	153.80							
Dry Soil Wt.	(gr)	589.00							
Moisture Content	(%)	26.1							
TEST RESULTS									
Wet Unit Wt.	(pcf)	111.2							
Moisture Content	(%)	26.1							
Dry Unit Wt.	(pcf)	88.2							
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.	(%)								

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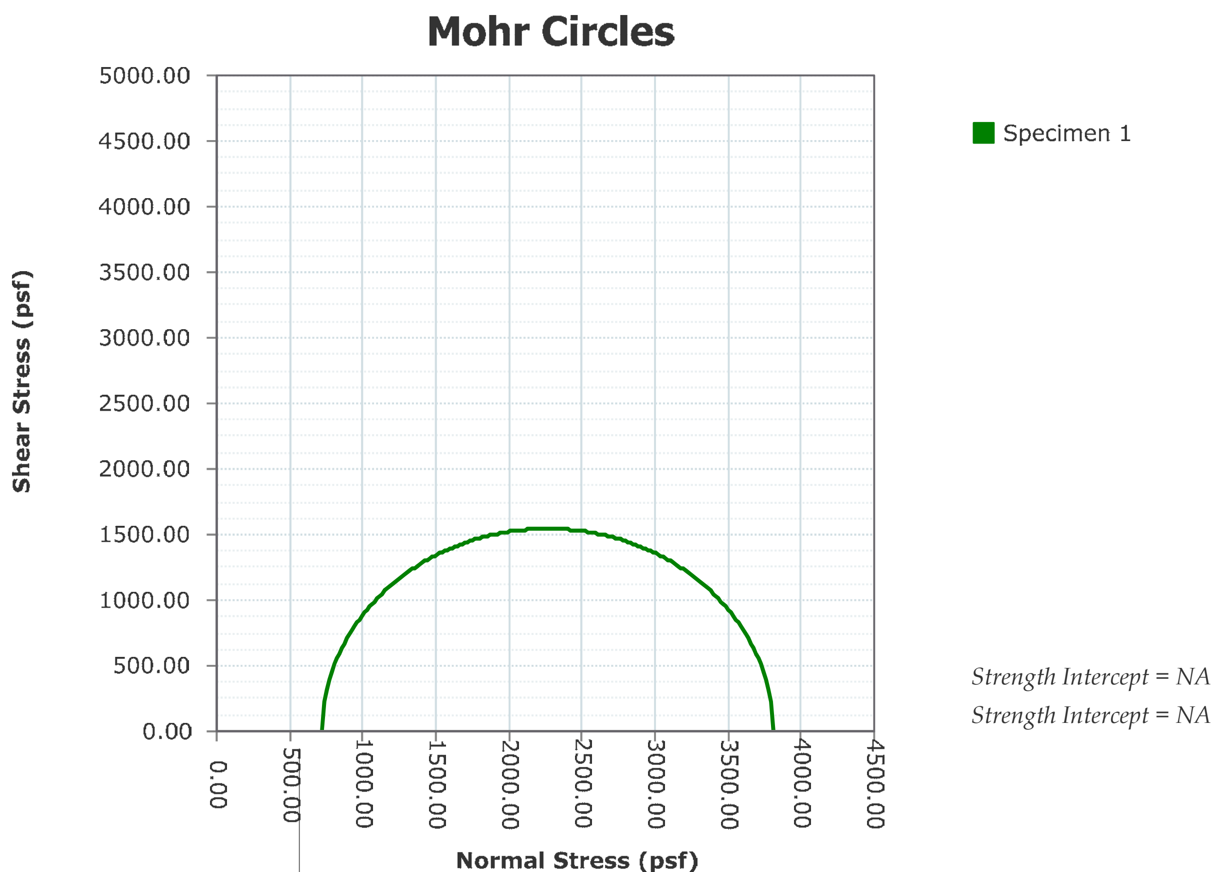
Unconsolidated Undrained Test

ASTM D2850



Project:	PUSD Ridgeview High School
Project Number:	70994.00.001
Sampling Date:	
Sample Number:	L1-1-2
Sample Depth:	1 ft
Location:	B20-3
Client Name:	Paradise USD
Remarks:	

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Unconsolidated Undrained Test

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Before Test	Specimen Number 1	2	3	4	5	6	7	8
Membrane Thickness (in)	0.001							
Initial Cell Pressure (psi)	5.0							
Height (in)	5.997							
Diameter (in)	2.371							
Water Content (%)	25.9							
Wet Density (Units)	105.4							
Dry Density (pcf)	83.7							
Degree of Saturation (%)	68.5							
Void Ratio	1.028							
Height To Diameter Ratio	2.529							
Test Data	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Comp. Strength at Failure (psf)	3091.03							
σ1 at Failure (psf)	3811.03							
σ3 at Failure (psf)	720.00							
Rate of Strain (in/min)	0.119940							
Axial Strain at Failure (%)	15.81							
After Test	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Final Water Content (%)	29.0							

Project:	PUSD Ridgeview High School
Project Number:	70994.00.001
Sampling Date:	
Sample Number:	L1-1-2
Sample Depth:	1 ft
Location:	B20-3
Client Name:	Paradise USD
Project Remarks:	

Specimen 1 Failure Sketch

Specimen 2 Failure Sketch

Specimen 3 Failure Sketch

Specimen 4 Failure Sketch

Specimen 5 Failure Sketch

Specimen 6 Failure Sketch

Specimen 7 Failure Sketch

Specimen 8 Failure Sketch

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Unconsolidated Undrained Test

ASTM D2850

Specimen 1			
Test Description:	Unconsolidated Undrained Triaxial		
Other Associated Tests:			
Device Details:			
Test Specification:			
Test Time:	7/14/2020		
Technician:	SC	Sampling Method:	Undisturbed
Specimen Code:		Specimen Lab #:	
Specimen Description:			
Specific Gravity:	2.720		
Plastic Limit:	0	Liquid Limit:	0
Height (in):	5.997	Diameter (in):	2.371
Area (in^2):	4.415	Volume (in^3):	26.48
Large Particle:			
Moisture Material:	Specimen		
Moist Weight (g):	732.5		
Test Remarks:			

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Mohr Circles (Total Stress) Graph

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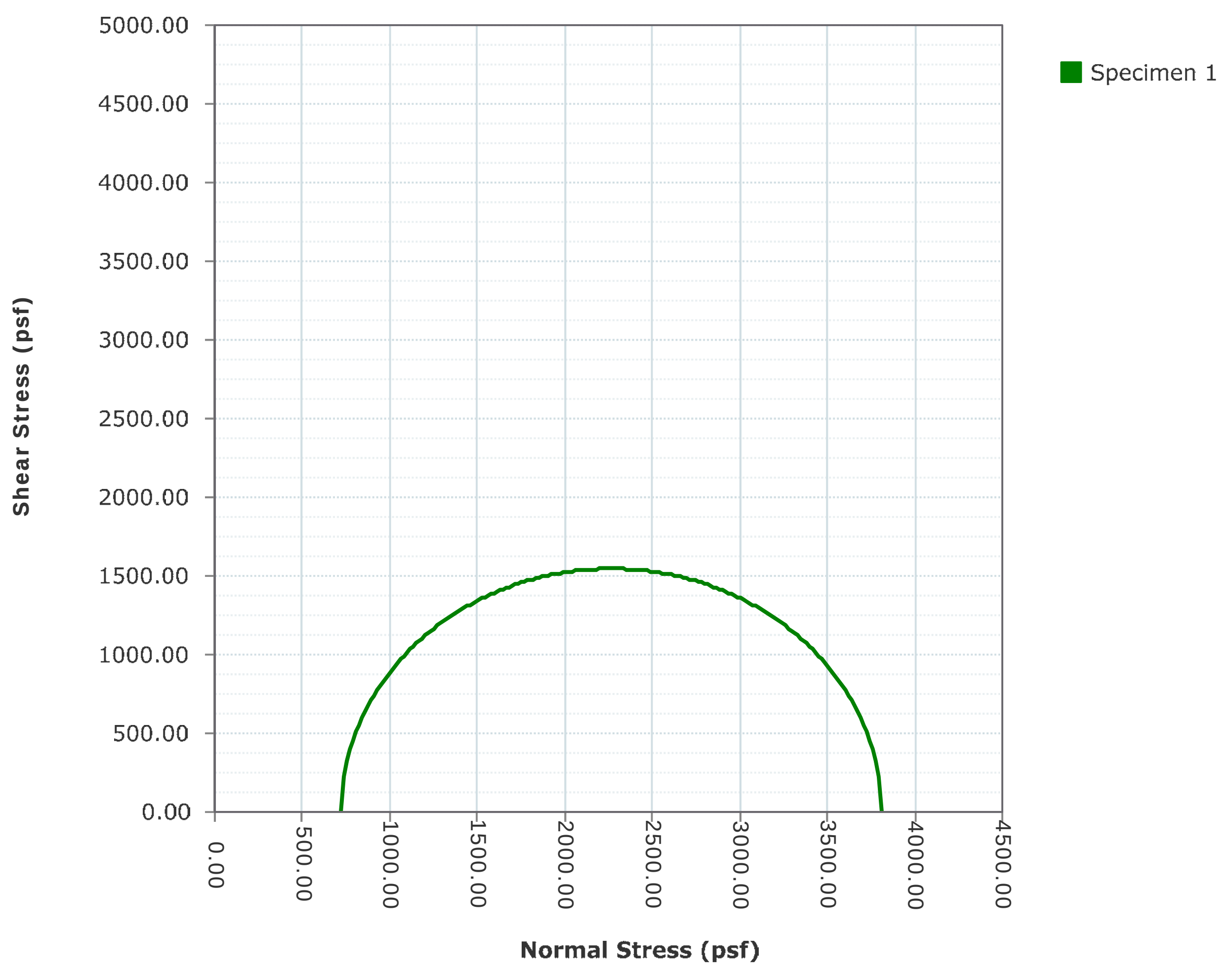

Tangent Results	
Strength Intercept (psi)	NA
Friction Angle (°)	NA

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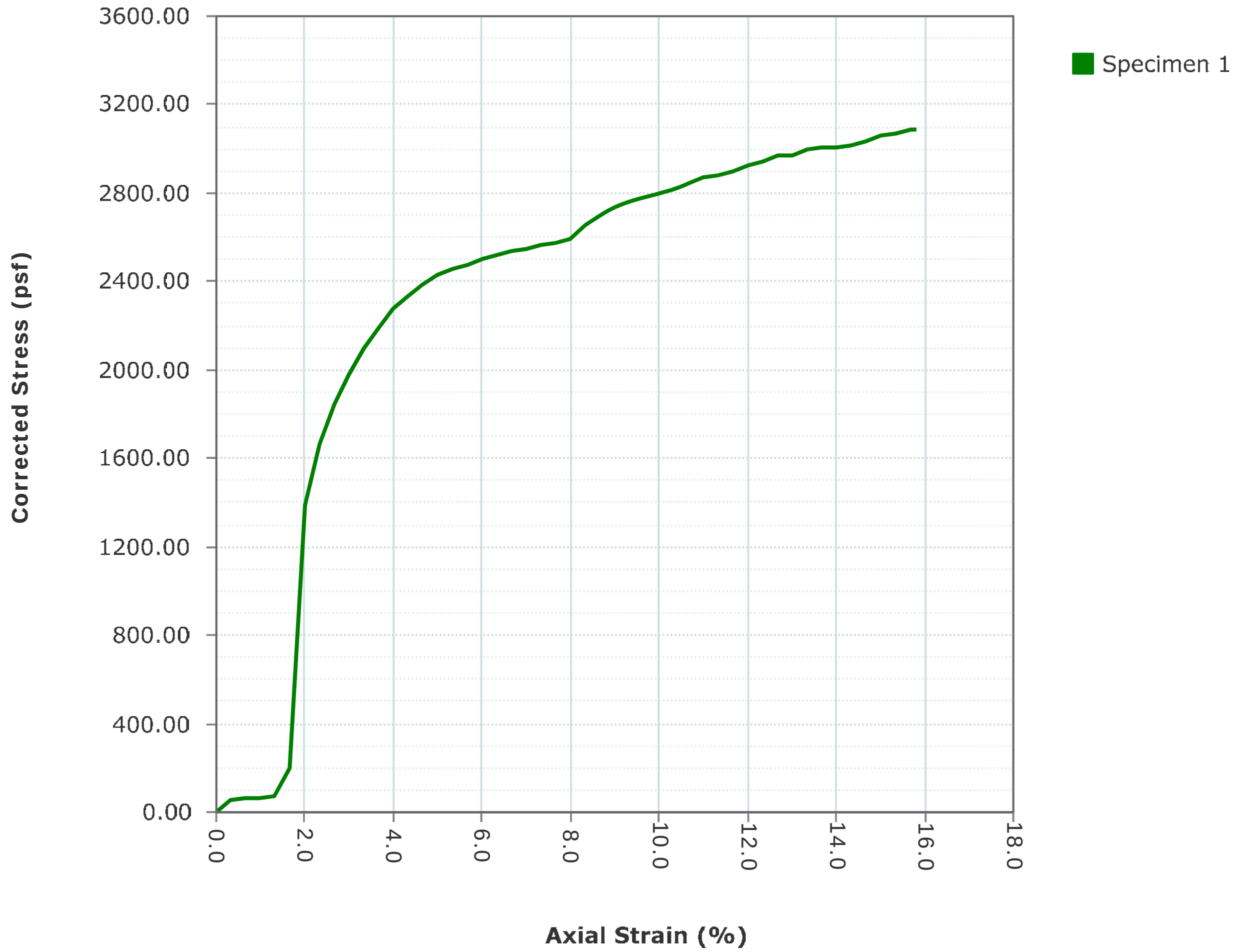
Stress-Strain Graph

ASTM D2850



Unconsolidated Undrained Test - Tabulated Data - Specimen 1

ASTM D2850

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Disp. (in)	Corrected Load (Lbf)	Corrected Disp. (in)	Corrected Area (in²)	Axial Strain (%)	Stress (psf)	Corrected Compressive Stress (psf)	σ1 (psf)	σ3 (psf)	σ1 — σ3	p (psf)	q (psf)
0	00:00:00	1.2	-0.0003	0.0	0.000	4.415	0.0	0.00	0.00	720.00	720.00	1.000	720.00	0.00
1	00:00:10	2.8	0.0187	1.6	0.019	4.429	0.3	53.62	53.30	773.30	720.00	1.074	746.65	26.65
2	00:00:20	3.0	0.0385	1.8	0.039	4.444	0.6	59.62	58.93	778.93	720.00	1.082	749.46	29.46
3	00:00:30	3.1	0.0580	2.0	0.058	4.459	1.0	64.71	63.62	783.62	720.00	1.088	751.81	31.81
4	00:00:40	3.4	0.0782	2.2	0.079	4.474	1.3	73.26	71.67	791.67	720.00	1.100	755.84	35.84
5	00:00:50	7.3	0.0990	6.1	0.099	4.490	1.7	199.81	195.71	915.71	720.00	1.272	817.85	97.85
6	00:01:00	44.6	0.1198	43.4	0.120	4.505	2.0	1,415.18	1,386.85	2,106.85	720.00	2.926	1,413.42	693.42
7	00:01:10	53.4	0.1397	52.2	0.140	4.521	2.3	1,702.63	1,662.89	2,382.89	720.00	3.310	1,551.44	831.44
8	00:01:20	59.2	0.1597	58.0	0.160	4.536	2.7	1,892.17	1,841.68	2,561.68	720.00	3.558	1,640.84	920.84
9	00:01:30	63.8	0.1794	62.7	0.180	4.552	3.0	2,044.08	1,982.82	2,702.82	720.00	3.754	1,711.41	991.41
10	00:01:40	67.7	0.1995	66.6	0.200	4.567	3.3	2,171.04	2,098.71	2,818.71	720.00	3.915	1,769.35	1,049.35
11	00:01:50	70.9	0.2190	69.7	0.219	4.583	3.7	2,273.11	2,190.00	2,910.00	720.00	4.042	1,815.00	1,095.00
12	00:02:00	73.8	0.2391	72.6	0.239	4.599	4.0	2,368.62	2,274.08	2,994.08	720.00	4.158	1,857.04	1,137.04
13	00:02:10	76.0	0.2585	74.8	0.259	4.614	4.3	2,440.79	2,335.48	3,055.48	720.00	4.244	1,887.74	1,167.74
14	00:02:20	77.8	0.2785	76.6	0.279	4.630	4.6	2,498.69	2,382.55	3,102.55	720.00	4.309	1,911.27	1,191.27
15	00:02:30	79.5	0.2992	78.3	0.300	4.647	5.0	2,553.49	2,425.96	3,145.96	720.00	4.369	1,932.98	1,212.98
16	00:02:40	80.7	0.3202	79.6	0.320	4.664	5.3	2,594.52	2,455.88	3,175.88	720.00	4.411	1,947.94	1,227.94
17	00:02:50	81.7	0.3401	80.5	0.340	4.681	5.7	2,625.97	2,476.90	3,196.90	720.00	4.440	1,958.45	1,238.45
18	00:03:00	82.9	0.3603	81.7	0.361	4.698	6.0	2,664.53	2,504.34	3,224.34	720.00	4.478	1,972.17	1,252.17
19	00:03:10	83.7	0.3799	82.5	0.380	4.714	6.3	2,691.21	2,520.59	3,240.59	720.00	4.501	1,980.29	1,260.29
20	00:03:20	84.4	0.3998	83.3	0.400	4.731	6.7	2,716.12	2,534.93	3,254.93	720.00	4.521	1,987.47	1,267.47
21	00:03:30	85.1	0.4194	83.9	0.420	4.747	7.0	2,736.35	2,544.85	3,264.85	720.00	4.535	1,992.43	1,272.43
22	00:03:40	86.0	0.4395	84.8	0.440	4.765	7.3	2,765.59	2,562.79	3,282.79	720.00	4.559	2,001.40	1,281.40
23	00:03:50	86.6	0.4588	85.4	0.459	4.781	7.7	2,786.13	2,572.87	3,292.87	720.00	4.573	2,006.43	1,286.43
24	00:04:00	87.6	0.4789	86.5	0.479	4.799	8.0	2,820.57	2,595.21	3,315.21	720.00	4.604	2,017.61	1,297.61
25	00:04:10	89.9	0.4994	88.7	0.500	4.817	8.3	2,893.06	2,651.99	3,371.99	720.00	4.683	2,046.00	1,326.00

Unconsolidated Undrained Test - Tabulated Data - Specimen 1

ASTM D2850

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Disp. (in)	Corrected Load (Lbf)	Corrected Disp. (in)	Corrected Area (in²)	Axial Strain (%)	Stress (psf)	Corrected Compressive Stress (psf)	σ1 (psf)	σ3 (psf)	σ1 / σ3	p (psf)	q (psf)
26	00:04:20	91.9	0.5203	90.7	0.521	4.835	8.7	2,958.46	2,701.63	3,421.63	720.00	4.752	2,070.82	1,350.82
27	00:04:30	93.3	0.5404	92.1	0.541	4.853	9.0	3,005.34	2,734.36	3,454.36	720.00	4.798	2,087.18	1,367.18
28	00:04:40	94.6	0.5603	93.4	0.561	4.870	9.3	3,046.57	2,761.80	3,481.80	720.00	4.836	2,100.90	1,380.90
29	00:04:50	95.5	0.5801	94.3	0.580	4.888	9.7	3,076.29	2,778.60	3,498.60	720.00	4.859	2,109.30	1,389.30
30	00:05:00	96.6	0.5998	95.5	0.600	4.906	10.0	3,113.93	2,802.32	3,522.32	720.00	4.892	2,121.16	1,401.16
31	00:05:10	97.6	0.6196	96.4	0.620	4.924	10.3	3,145.55	2,820.43	3,540.43	720.00	4.917	2,130.22	1,410.22
32	00:05:20	98.8	0.6396	97.7	0.640	4.943	10.7	3,185.28	2,845.42	3,565.42	720.00	4.952	2,142.71	1,422.71
33	00:05:30	100.0	0.6590	98.8	0.659	4.961	11.0	3,223.28	2,868.92	3,588.92	720.00	4.985	2,154.46	1,434.46
34	00:05:40	100.8	0.6792	99.7	0.679	4.979	11.3	3,250.59	2,882.31	3,602.31	720.00	5.003	2,161.16	1,441.16
35	00:05:50	101.8	0.6996	100.6	0.700	4.999	11.7	3,281.35	2,898.42	3,618.42	720.00	5.026	2,169.21	1,449.21
36	00:06:00	103.2	0.7201	102.0	0.720	5.018	12.0	3,326.94	2,927.29	3,647.29	720.00	5.066	2,183.64	1,463.64
37	00:06:10	104.1	0.7403	103.0	0.741	5.037	12.3	3,357.86	2,943.23	3,663.23	720.00	5.088	2,191.61	1,471.61
38	00:06:20	105.4	0.7602	104.3	0.760	5.056	12.7	3,400.65	2,969.42	3,689.42	720.00	5.124	2,204.71	1,484.71
39	00:06:30	105.9	0.7800	104.7	0.780	5.076	13.0	3,415.87	2,971.45	3,691.45	720.00	5.127	2,205.72	1,485.72
40	00:06:40	107.2	0.7998	106.0	0.800	5.095	13.3	3,457.00	2,995.79	3,715.79	720.00	5.161	2,217.90	1,497.90
41	00:06:50	107.8	0.8194	106.6	0.820	5.114	13.7	3,478.01	3,002.64	3,722.64	720.00	5.170	2,221.32	1,501.32
42	00:07:00	108.3	0.8393	107.2	0.840	5.134	14.0	3,494.67	3,005.42	3,725.42	720.00	5.174	2,222.71	1,502.71
43	00:07:10	109.0	0.8588	107.8	0.859	5.153	14.3	3,516.32	3,012.62	3,732.62	720.00	5.184	2,226.31	1,506.31
44	00:07:20	110.1	0.8790	108.9	0.879	5.174	14.7	3,553.30	3,032.32	3,752.32	720.00	5.212	2,236.16	1,516.16
45	00:07:30	111.5	0.8994	110.4	0.900	5.194	15.0	3,599.40	3,059.44	3,779.44	720.00	5.249	2,249.72	1,529.72
46	00:07:40	112.3	0.9201	111.2	0.920	5.216	15.3	3,626.17	3,069.67	3,789.67	720.00	5.263	2,254.83	1,534.83
47	00:07:50	113.3	0.9401	112.1	0.940	5.236	15.7	3,655.76	3,082.54	3,802.54	720.00	5.281	2,261.27	1,541.27
48	00:07:54	113.7	0.9480	112.6	0.948	5.245	15.8	3,671.63	3,091.03	3,811.03	720.00	5.293	2,265.51	1,545.51

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

27 July 2020

Job No. 2007059
Cust. No. 12830

Mr. Dominic Potestio
H&K/NV5
48 Bellarmine Court, Suite 40
Chico, CA 95928

Subject: Project No.: 70994.00.001
Project Name: Ridgeview High School, Paradise, CA
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. Potestio:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on July 14, 2020. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "mildly corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The pH of the soil is 4.99 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potential is 440-mV and is indicative of potentially "non-corrosive" soils resulting from aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client:	H&K/NV5
Client's Project No.:	70994.00.001
Client's Project Name:	Ridgeview High School, Paradise, CA
Date Sampled:	13-Jul-20
Date Received:	14-Jul-20
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 27-Jul-2020

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2007059-001	B20-1 Blk-1 (1'-4')	440	4.99	-	13,000		N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Jul-2020	25-Jul-2020	-	24-Jul-2020	-	25-Jul-2020	25-Jul-2020

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

Chain of Custody

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775

CERCO analytical

Job No.	CU#.	Client Project I.D.
2007059	12830	70994.00.001

Schedule		Date Sampled	Date Due
Analyte			

Full Name Dominic Potestio

Phone 530.894.2487
Fax 530.894.2437
Cell 530.362.0105

Company and/or Mailing Address
Holdrege & Kull, 48 Bellarmine Court, Ste 40, Chico CA 95928 ☒

Sample Source
B20-1 Blk-1 Ridgeview High School, Paradise, CA

ANALYSIS — ASTM

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated			Brief Evaluation						
001	B20-1 Blk-1 (1'-4')	7/13/20	09:00	S	bag			1	x	x	x	x	x			X						

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

SAMPLE RECEIPT
Total No. of Containers
Rec'd Good Cond/Cold
Conforms to Record
Temp. a t Lab -°C
Sampler

		Date	Time
Relinquished By:	[signature]	7/13/20	09:00
Received By:	[signature]	7/13/20	0900
Relinquished By:		14	
Received By:			
Relinquished By:			
Received By:			

Comments:

THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM METAL TUBES

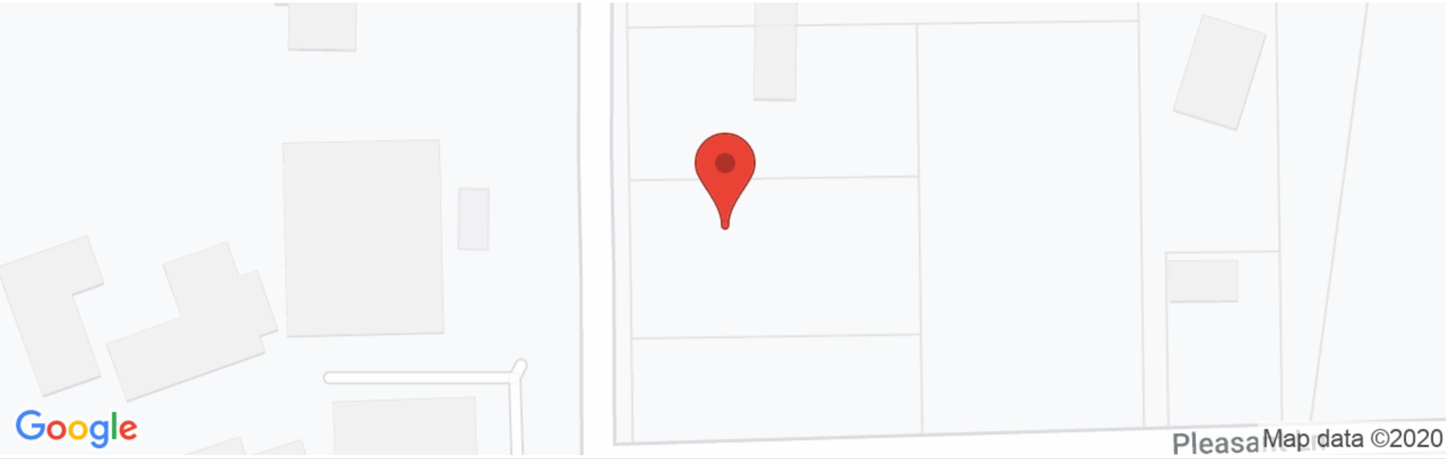
Email Address: dominic.potestio@nv5.com

APPENDIX E:

Seismic Design Parameters

Ridgeview High School

Latitude, Longitude: 39.7622, -121.6121



Date	7/11/2020, 9:25:43 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	0.692	MCE_R ground motion. (for 0.2 second period)
S_1	0.289	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.846	Site-modified spectral acceleration value
S_{M1}	0.433	Site-modified spectral acceleration value
S_{DS}	0.564	Numeric seismic design value at 0.2 second SA
S_{D1}	0.289	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.223	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.305	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.366	Site modified peak ground acceleration
T_L	16	Long-period transition period in seconds
SsRT	0.692	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.761	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.289	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.319	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}	0.909	Mapped value of the risk coefficient at short periods
C_{R1}	0.905	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

Input

Edition

Dynamic: Conterminous U.S. 2014 (upd…

Latitude

Decimal degrees

39.7622

Longitude

Decimal degrees, negative values for western longitudes

-121.6121

Site Class

537 m/s (Site class C)

Spectral Period

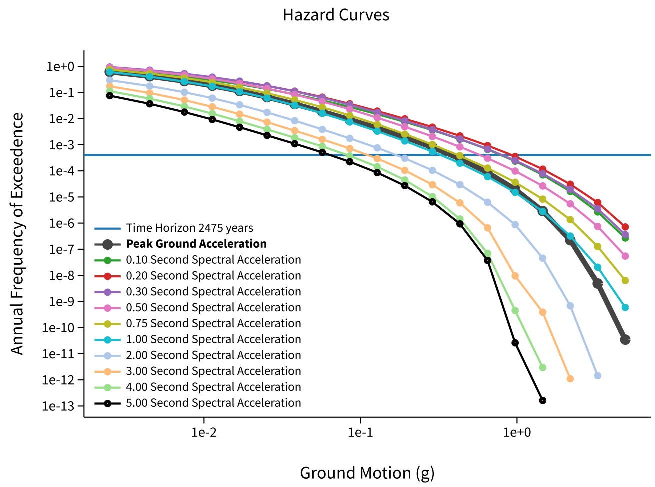
Peak Ground Acceleration

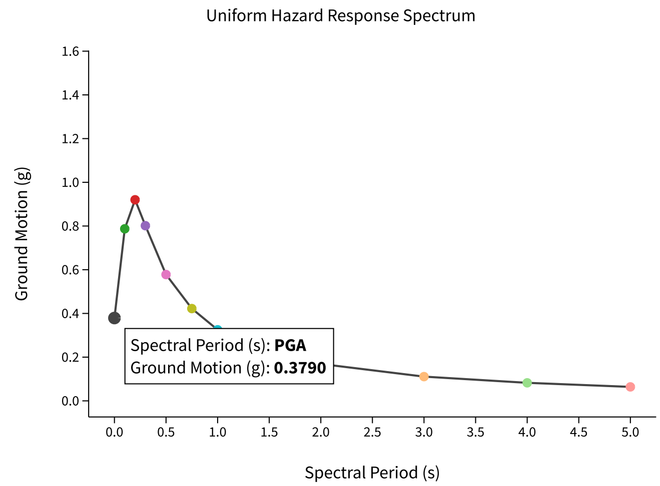
Time Horizon

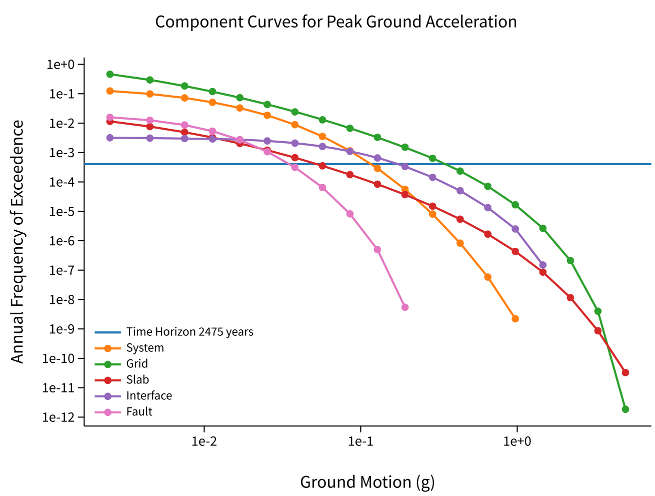
Return period in years

2475

Hazard Curve





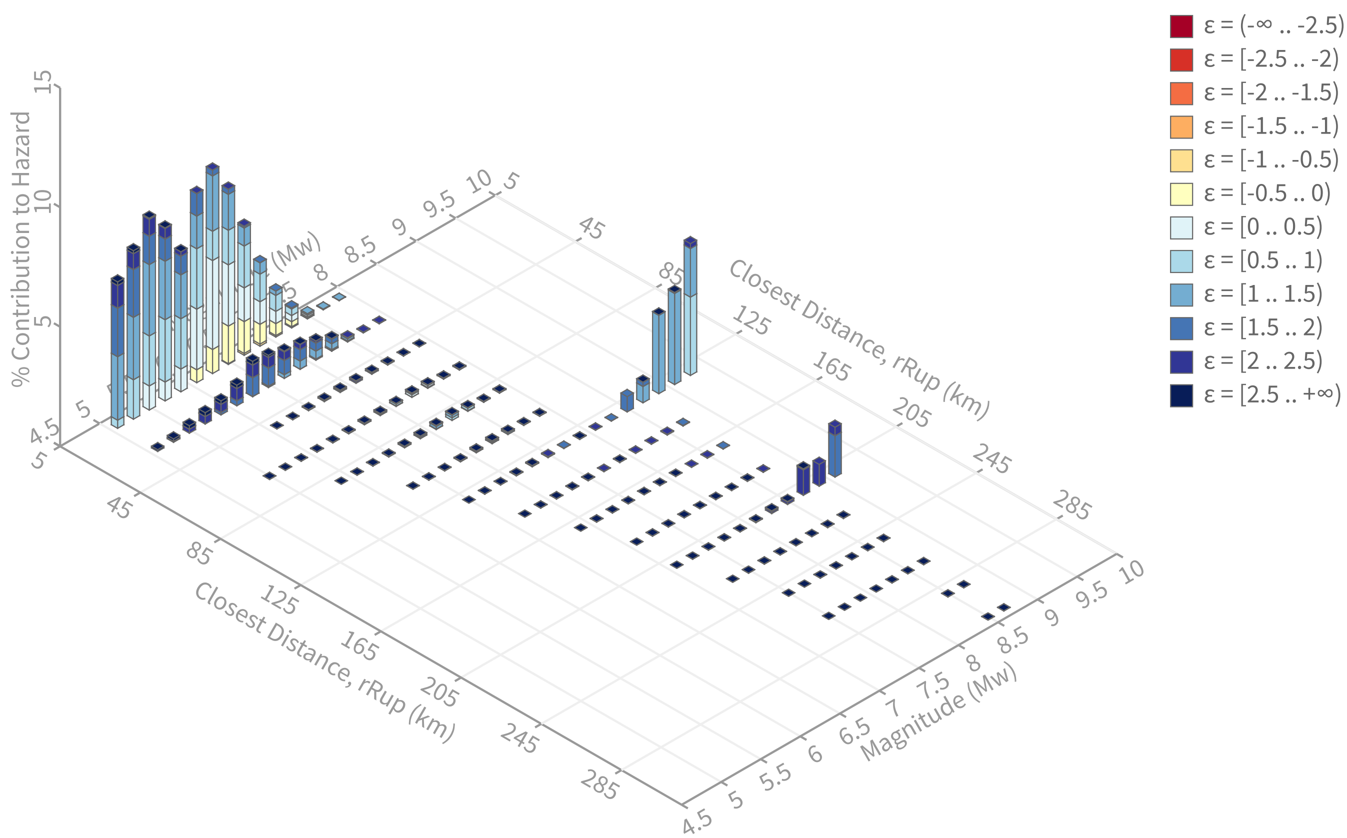


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.3789539 g

Recovered targets

Return period: 2578.8489 yrs
Exceedance rate: 0.0003877699 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.55 %

Mean (over all sources)

m: 6.64
r: 40.37 km
ε_0: 1.08 σ

Mode (largest m-r bin)

m: 6.3
r: 10.62 km
ε_0: 0.59 σ
Contribution: 8.51 %

Mode (largest m-r-ε_0 bin)

m: 9.01
r: 138.77 km
ε_0: 1.2 σ
Contribution: 3.81 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31 (opt)	Grid							39.59
PointSourceFinite: -121.612, 39.803		6.62	5.76	0.52	121.612°W	39.803°N	0.00	4.14
PointSourceFinite: -121.612, 39.857		10.36	5.99	0.94	121.612°W	39.857°N	0.00	3.25
PointSourceFinite: -121.612, 39.821		7.80	5.83	0.67	121.612°W	39.821°N	0.00	2.94
PointSourceFinite: -121.612, 39.803		6.62	5.76	0.52	121.612°W	39.803°N	0.00	2.77
PointSourceFinite: -121.612, 39.839		9.05	5.91	0.81	121.612°W	39.839°N	0.00	2.38
PointSourceFinite: -121.612, 39.857		10.36	5.99	0.94	121.612°W	39.857°N	0.00	2.11
PointSourceFinite: -121.612, 39.821		7.80	5.83	0.67	121.612°W	39.821°N	0.00	1.92
PointSourceFinite: -121.612, 39.839		9.05	5.91	0.81	121.612°W	39.839°N	0.00	1.63
PointSourceFinite: -121.612, 39.866		11.02	6.03	1.00	121.612°W	39.866°N	0.00	1.17
PointSourceFinite: -121.612, 39.929		15.89	6.29	1.36	121.612°W	39.929°N	0.00	1.13
PointSourceFinite: -121.612, 39.920		15.19	6.25	1.31	121.612°W	39.920°N	0.00	1.10
UC33brAvg_FM32 (opt)	Grid							39.53
PointSourceFinite: -121.612, 39.803		6.62	5.76	0.52	121.612°W	39.803°N	0.00	4.14
PointSourceFinite: -121.612, 39.857		10.36	5.99	0.94	121.612°W	39.857°N	0.00	3.25
PointSourceFinite: -121.612, 39.821		7.80	5.83	0.68	121.612°W	39.821°N	0.00	2.93
PointSourceFinite: -121.612, 39.803		6.62	5.76	0.52	121.612°W	39.803°N	0.00	2.77
PointSourceFinite: -121.612, 39.839		9.05	5.91	0.82	121.612°W	39.839°N	0.00	2.38
PointSourceFinite: -121.612, 39.857		10.36	5.99	0.94	121.612°W	39.857°N	0.00	2.10
PointSourceFinite: -121.612, 39.821		7.80	5.83	0.68	121.612°W	39.821°N	0.00	1.92
PointSourceFinite: -121.612, 39.839		9.05	5.91	0.82	121.612°W	39.839°N	0.00	1.63
PointSourceFinite: -121.612, 39.866		11.02	6.03	1.00	121.612°W	39.866°N	0.00	1.17
PointSourceFinite: -121.612, 39.929		15.90	6.29	1.36	121.612°W	39.929°N	0.00	1.13
PointSourceFinite: -121.612, 39.920		15.19	6.25	1.31	121.612°W	39.920°N	0.00	1.10
sub0_ch_bot.in	Interface							12.04
Cascadia Megathrust - whole CSZ Characteristic		138.77	9.12	1.15	122.945°W	40.376°N	301.46	12.04
sub0_ch_mid.in	Interface							3.31
Cascadia Megathrust - whole CSZ Characteristic		201.24	8.95	1.98	123.829°W	40.347°N	289.73	3.31