GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
PROPOSED BURNS VALLEY DEVELOPMENT
BURNS VALLEY ROAD
CLEARLAKE, LAKE COUNTY, CALIFORNIA

February 26, 2021

Prepared For:

CITY OF CLEARLAKE
14050 Olympic Drive
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125620-0071075.00.001

February 26, 2021
Project No. 71075.00.001

Ms. Adeline Brown,
Engineering Tech/Construction Manager
City of Clearlake
14050 Olympic Drive
Clearlake, California 95422

Reference: **Geotechnical Engineering Investigation Report**
Proposed Burns Valley Development
Burns Valley Road, Clearlake, Lake County, California

Dear Ms. Brown,

NV5 conducted a geotechnical engineering investigation for the proposed Burns Valley Development located at Burns Valley Road, Clearlake, California. NV5's geotechnical engineering investigation of the site was performed consistent with the scope of services presented in the November 6, 2020 proposal (PC20.230).

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 previous experience with similar projects, sites and conditions in the area. The approximately 25-acre parcel is proposed for mixed-use development including multi-story apartment buildings, a single-story commercial building, and a City of Clearlake Public Works (CCPW) Yard with an approximately 20,000-square-foot (sf) shop utilizing conventional design and construction practices. There were no seismic hazards identified on the site or in the immediate area that require design mitigation. Portions of the site support loose undocumented fills that are not considered suitable for support of the proposed improvements. Therefore, 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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ACRONYMS

°F	degrees Fahrenheit
AB	aggregate base
AC	asphalt concrete
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below ground surface
Cal/EPA	California Environmental Protection Agency
CAT	Caterpillar
CBC	California Building Code
CCPW	City of Clearlake Public Works
CEC	California Engineering Company
CGS	California Geological Survey
CQA	Construction Quality Assurance
DTSC	Department of Toxic Substances Control
DWR	Department of Water Resources
EERI	Earthquake Engineering Research Institute
EFP	equivalent fluid pressure
FS	factor of safety
ft/s	feet per second
GBA	Geoprofessional Business Association
H:V	horizontal to vertical slope ratio
IBC	International Building Code
km	kilometer
MCE	maximum considered earthquake
M_L	local magnitude earthquake
msl	mean sea level
M_w	modal magnitude
NEIC	National Earthquake Information Center
OSHA	Occupational Safety and Hazards Administration
oz/sy	ounce per square yard
P-wave	seismic compression wave
PCA	Portland Cement Association
pcf	pounds per cubic foot
PGA_M	peak ground acceleration
PG&E	Pacific Gas & Electric
PI	plasticity index
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinylchloride
Q_{al}	Quaternary Alluvium
S-wave	shear-wave
SEAOC	Structural Engineers Association of California
sf	square foot
SPT	standard penetration test
SRMS	Seismic Refraction Microtremor Survey

ACRONYMS (CONCLUDED)

SSD	saturated surface dry
TI	traffic index
USCS	Unified Soils Classification System
USGS	United States Geological Survey

1.0 INTRODUCTION

NV5 performed a geotechnical engineering investigation and prepared a geotechnical engineering investigation report for the proposed Burns Valley Development mixed-use project at Burns Valley Road in Clearlake, California, consistent with the scope of services presented in NV5's *Proposal for Geotechnical Engineering Services* (PC20.230), dated November 6, 2020. NV5's findings, conclusions and recommendations are presented herein.

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

1.1 SCOPE-OF-SERVICES

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

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

1.2 SITE LOCATION AND DESCRIPTION

The proposed Burns Valley Development are located at Burns Valley Road, in Clearlake, California, identified as Lake County Assessor's Parcel Numbers 010-026-40, 010-026-29 and 039-570-18. The proposed development is located at the southwest corner of Burns Valley Road and Rumsey Road. The site is centered at about latitude 38.9638 north and longitude -122.6349 west on the United States Geological Survey's (USGS), 7.5 minute Clearlake Highlands Quadrangle topographic map. The property elevation is approximately 1360 feet above mean sea level (msl), based on review of the USGS 7.5-minute Clearlake Highlands Quadrangle topographic map, and is generally flat with a gentle downgrade slope from east to west. Figure 1 shows the approximate site location and vicinity.

SITE LOCATION MAP

CITY OF CLEARLAKE BURNS VALLEY DEVELOPMENT

BURNS VALLEY ROAD, CLEARLAKE, LAKE COUNTY, CALIFORNIA

DRAWN BY:	SC
CHECKED BY:	DJP
PROJECT:	71075.01.001
DATE:	FEBRUARY 2021

FIGURE 1

At the time the site investigation was performed on January 12 and 13, 2021, the following conditions were observed and are shown in the inset image:

The area of the proposed Burns Valley Development is comprised of Lake County Assessor's Parcel Numbers 010-026-40, 010-026-29 and 039-570-18. Each of the three parcels is described respectively.

- Parcel 010-026-40 is an irregular-shaped property generally comprised of an existing tree orchard and high concentrations of weeds and grasses. The terrain was relatively flat with a gentle downward slope from the east to the west. A drainage channel transected the east portion of the property in the southwest direction. To the east of the drainage channel the surface topography was relatively higher in elevation than the rest of the site. Large stockpiles consisting of soils, concrete and asphalt rubble, boulders, and other deleterious debris were present. Overhead power poles and power lines were present along the north and east boundaries of the property. The property was bounded to the east and north by Burns Valley Road; to the west by Burns Valley Creek; and, to the south by apartments, commercial buildings and a retail shopping center.

- Parcel 010-026-29 is a rectangular shaped property supporting a large number of mature oak trees, agricultural tress, and high concentrations of weeds and grasses. Concrete foundation remnants of a former structure and a large construction crane were present in the southern portion of the property. A drainage channel transected the center of the site and extended in the southwest direction. A California Department of Water Recourses (DWR) monitoring well was present in the northeast portion of the site. A water well pump house was present in the north half of the property. The site was bounded to the north and east by Burns Valley Road, to the south by fallow land and stockpiles; and, to the west by a senior living community.

- Parcel 039-570-18 is a rectangular shaped property comprised of fallow land supporting low to moderate concentrations of weeds and grasses. Sparse mature trees and fence posts were present throughout the site. Numerous utility markings were present indicating the presence of underground utilities. The property is bounded to the north by existing tree orchards; to the west by an existing Pacific Gas & Electric (PG&E) facility; to the south by Olympic Drive; and, to the east by a retail shopping center. Evidence of a former structure was observed in the northern portion of the parcel.

1.3 PROPOSED IMPROVEMENTS

Based on the preliminary project information provided by representatives of California Engineering Company (CEC), NV5 understands the approximately 30-acre parcel is proposed for mixed-use development including multi-story apartment buildings, a single-story commercial building, and a City of Clearlake Public Works (CCPW) Yard with an approximately 20,000-square-foot (sf) shop. The proposed residential and commercial structures are anticipated to be constructed with wood or light-metal framing and supported on shallow concrete foundations with interior concrete slab-on-grade floors. The proposed CCPW shop is anticipated to consist of a metal, prefabricated building, or constructed with light-metal framing, and supported on shallow concrete foundations with an interior concrete slab-on-grade floor.

Associated development is indicated to include construction of an asphalt concrete paved police department parking lot, recreational fields (baseball/softball, soccer, etc.), underground utilities, exterior slab-on-grade concrete flatwork, rigid concrete and asphalt concrete pavements, and landscaping. Earthwork grading may include general site preparation, and minor cuts and fills to balance the site to meet the proposed building grades. Figure 2 shows the proposed site location and approximate exploratory boring locations.

1.4 INVESTIGATION PURPOSE

The purpose of the geotechnical investigation was to obtain sufficient on-site information about the soil, rock and groundwater conditions to 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.

LEGEND

B21-1 APPROXIMATE EXPLORATORY BORING LOCATION, NV5, 2021

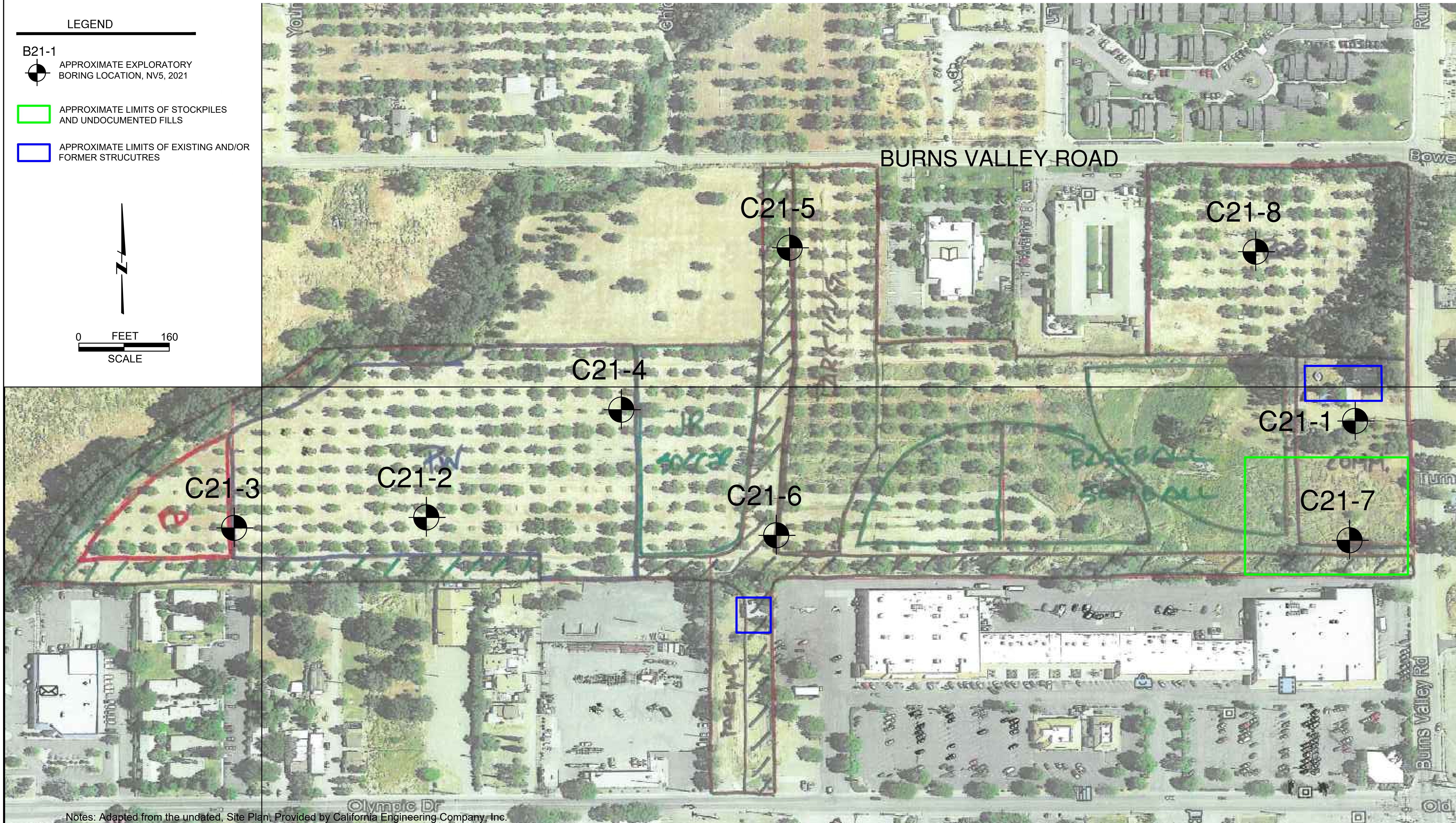
APPROXIMATE LIMITS OF STOCKPILES AND UNDOCUMENTED FILLS

APPROXIMATE LIMITS OF EXISTING AND/OR FORMER STRUCUTRES

N

0 FEET 160

SCALE



Notes: Adapted from the undated, Site Plan, Provided by California Engineering Company, Inc.

NV5

SITE PLAN AND EXPLORATORY CORE LOCATION MAP

CITY OF CLEARLAKE BURNS VALLEY DEVELOPMENT

BURNS VALLEY ROAD, CLEARLAKE, LAKE COUNTY, CALIFORNIA

DRAWN BY:	SC
CHECKED BY:	DJP
PROJECT:	71075.01.001
DATE:	FEBUARY 2021

FIGURE 2

2.0 SITE INVESTIGATION

NV5 performed a site investigation to characterize the existing surface and subsurface conditions beneath the proposed 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 track-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 reviewed the following reports associated with the project site area. The following identifies each report and summarizes the findings, conclusions and recommendations presented in each report:

- NV5, 2021, *Field Investigation Summery Report, Sulphur Fire Road Rehabilitation Project, Various Streets, Clearlake, California*, prepared by NV5, February XX.

  The investigation consisted of evaluating various streets within the City of Clearlake. The evaluation consisted of logging the existing pavement conditions and thickness, collecting representative sample of the underlying subgrade materials for subgrade quality testing. Based on the field and laboratory information recommendations were provided for roadway rehabilitation with asphalt concrete overlay or full depth reconstruction.

- NV5, 2021, *Reconnaissance Geotechnical Engineering Report, City of Clearlake Sulphur Fire Cuts Rehabilitation Assessments, Clearlake, California*, prepared by NV5, January 11.

  The investigation consisted of evaluating seven existing damaged road cuts for slope stability failure modes. The cuts only showed evidence of shallow erosion caused by surface water runoff, shallow sloughing and/or shallow soil creep. Recommendations for standard soil erosion prevention rehabilitation practices were provided to mitigate the erosion concerns.

2.2 REGIONAL GEOLOGY

The proposed Burns Valley Development is situated in the Coast Range Geomorphic Province of California. The Coast Range Geomorphic Province is characterized as northwest-trending mountain ranges and valleys that are subparallel to the San Andreas Fault. Strata of the Coast Range dip beneath alluvium of the Great Valley to the east and rise above the Pacific Ocean to the west. The Coast Range is comprised of thick Mesozoic and Cenozoic sedimentary rocks that were uplifted by the San Andreas Fault, terraced, and wave-cut. In the northern region, the Coast Range is dominated by irregular and knobby topography of the Franciscan Complex. Locally, the Franciscan rocks are overlain by volcanic cones and flows of the Clearlake volcanic field.

In the Clearlake area, the geology is dominated by the late Pliocene to early Holocene Clearlake volcanic field. The volcanic field consists of lava domes, cinder cones, and maars comprised of basalt and rhyolite. Cobb Mountain and Mount Konocti are the two highest peaks in the volcanic field. The Geysers, which host the largest complex of geothermal plants in the world, are located within the volcanic field.

2.3 SITE GEOLOGY

Based on review of the *Geologic Map of the Santa Rosa Quadrangle*, published by the California Division of Mines and Geology (Wagner and Bortugno, 1982), the geology immediately underlying the subject site is comprised of Quaternary Alluvium. Quaternary Alluvium is comprised of Pleistocene to Holocene Age alluvial deposits of sand, gravel, silt, and clay.

2.4 REGIONAL FAULTING AND SEISMIC SOURCES

Regional faulting is associated with the Maacama Fault Zone and Konocti Bay Fault Zone to the west, the Bartlett Springs Fault Zone to the north and east and the Hunting Creek-Berryessa Fault Zone to the south. NV5 reviewed the Official Maps of Earthquake Fault Zones delineated by the California Geological Survey through December 2010, on the internet at http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps. These maps are updates to Special Publication 42, Interim Revision 2007 edition *Fault Rupture Hazard Zones in California*, which describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. Special Publication 42 and the 2010 on-line update indicate that the site is not located within an Alquist-Priolo active fault zone. However, the Clearlake Highlands Alquist-Priolo active fault zone is located approximately 3 miles to the west of the site.

According to the Fault Activity Map of California (2010) by the California Geological Survey, Geologic Data Map No. 6 (http://maps.conservation.ca.gov/cgs/fam/), the closest known active fault which has surface displacement within Holocene time (about the last 11,000 years) is the Konocti Bay Fault Zone. The mapped fault zone is located approximately 3 miles west of the subject site. The Fault Activity Map of California (2010) also shows the Bartlett Springs Fault Zone located 6 miles (13 kilometer [km]) northeast of the site and the Hunting Creek-Berryessa Fault Zone located 10 miles (15 km) east of the site to be known active faults with surface displacement within Holocene time.

2.5 FIELD INVESTIGATION

NV5 performed a field investigation of the site on January 12 and 13, 2021. 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, Inc. (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 existing building footprint, surrounding improvements and the approximate exploratory boring locations. The area of the proposed Burns Valley Development is comprised of Lake County Assessor's Parcel Numbers 010-026-40, 010-026-29 and 039-570-18. Each of the three parcels is described respectively.

Parcel 010-026-40 is an irregular-shaped property generally comprised of an existing tree orchard and high concentrations of weeds and grasses. The terrain was relatively flat with a gentle downward slope from the east to the west. A drainage channel transected the east portion of the property in the southwest direction. To the east of the drainage channel the surface topography was relatively higher in elevation than the rest of the site. Large stockpiles consisting of soils, concrete and asphalt rubble, boulders, and other deleterious debris were present. Overhead power poles and power lines were present along the north and east boundaries of the property. The property was bounded to the east and north by Burns Valley Road; to the west by Burns Valley Creek; and, to the south by apartments, commercial buildings and a retail shopping center.

Parcel 010-026-29 is a rectangular shaped property supporting a large number of mature oak trees, agricultural tress, and high concentrations of weeds and grasses. Concrete foundation remnants of a former structure and a large construction crane were present in the southern portion of the property. A drainage channel transected the center of the site and extended in the southwest direction. A California DWR monitoring well was present in the northeast portion of the site. A water well pump house was present in the north half of the property. The site was bounded to the north and east by Burns Valley Road, to the south by fallow land and stockpiles; and, to the west by a senior living community.

Parcel 039-570-18 is a rectangular shaped property comprised of fallow land supporting low to moderate concentrations of weeds and grasses. Sparse mature trees and fence posts were present throughout the site. Numerous utility markings were present indicating the presence of underground utilities. The property is bounded to the north by existing tree orchards; to the west by an existing PG&E facility; to the south by Olympic Drive; and, to the east by a retail shopping center. Evidence of a former structure was observed in the northern portion of the parcel.

2.5.2 Subsurface Conditions

The subsurface soil, rock and groundwater conditions were investigated by drilling exploratory borings. The subsurface information obtained from this investigation method is described in the following subsections.

2.5.2.1 Exploratory Boring Information

NV5 provided engineering oversight for the excavation of 8 exploratory soil borings at the project site. The borings were advanced with a track-mounted CME-55 drill rig equipped with 8-inch outside diameter, continuous flight, hollow stem augers. Figure 2 shows the approximate locations of the subsurface exploratory excavations. The borings were excavated to a maximum depth of 51.5 feet below ground surface (bgs). Engineering judgment was used to extrapolate the observed soil, rock and groundwater conditions to areas located between and beyond the subsurface exploratory excavations.

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 overshot cathead 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 B. The soil and rock descriptions include: visual field estimates of the particle size percentages (by dry weight), color, relative density or consistency, moisture content and cementation that comprise each soil material encountered.

A generalized profile of the soil, rock and groundwater conditions encountered to the maximum depth excavated (51.5 feet) for 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.

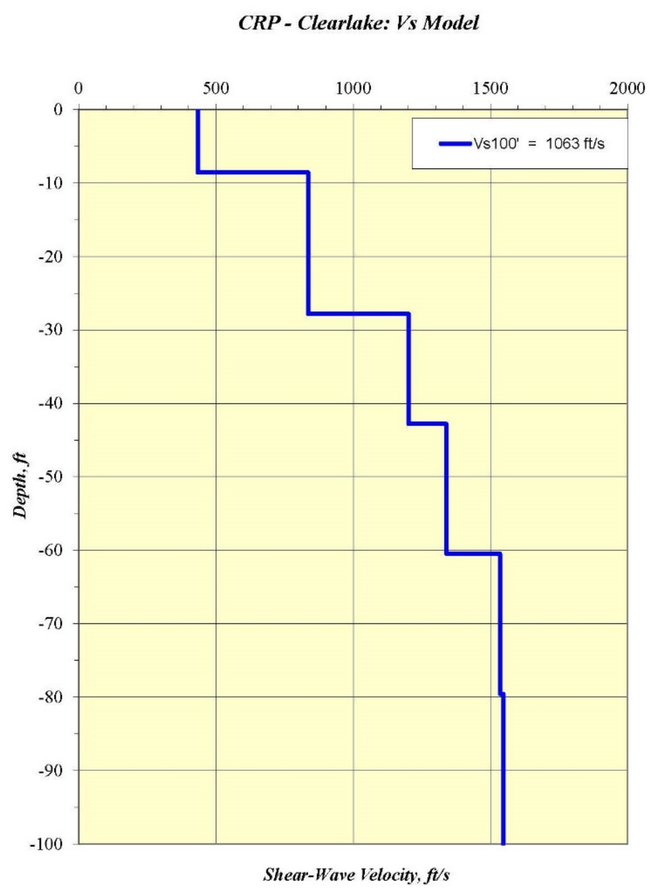
- **ML, Low Plasticity Silt Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages 70 percent low plasticity silt and clay fines and 30 percent fine sand. This soil is predominantly dark yellowish brown with a Munsell® Soil-Color Chart designation of (10YR, 4/4). This soil was stiff and damp at the time of the subsurface investigation.
- **SC, Clayey Sand Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 55 percent fine sand, 20 percent low plasticity silt and clay fines, and 25% Gravel. This soil is predominantly dark yellowish brown with a Munsell® Soil-Color Chart designation of (10YR, 4/6). This soil was medium dense and moist to damp at the time of the subsurface investigation
- **CL, Low Plasticity Clay Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages 85 percent low plasticity silt and clay fines and 15 percent fine sand. This soil is predominantly brown with a Munsell® Soil-Color Chart designation of (10YR, 4/3). This soil was stiff and moist at the time of the subsurface investigation.
- **GM, Silty Gravel Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 60 percent gravel, 30 percent fine sand and 10 percent low plasticity silt and clay fines. This soil is predominantly light gray with a Munsell® Soil-Color Chart

designation of (10YR, 7/1). This soil was medium dense and wet at the time of the subsurface investigation.

- **CH, High Plasticity Clay Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages 85 percent high plasticity silt and clay fines and 15 percent fine sand. This soil is predominantly dark greenish gray with a Munsell® Soil-Color Chart designation of (GLEY 1, 4/1). This soil was firm and wet at the time of the subsurface investigation.
- **GP, Poorly Graded Gravel Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 80 percent gravel, 10 percent fine sand and 10 percent low plasticity silt and clay fines. This soil is predominantly gray with a Munsell® Soil-Color Chart designation of (10YR, 5/1). This soil was dense and very moist at the time of the subsurface investigation.
- **SM, Silty Sand Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 55 percent fine sand and 45 percent low plasticity silt and clay fines. This soil is predominantly dark grayish brown with a Munsell® Soil-Color Chart designation of (2.5YR, 4/2). This soil was medium dense and wet at the time of the subsurface investigation.

2.5.2.2 Seismic Refraction Microtremor Survey

A Seismic Refraction Microtremor Survey (SRMS) was performed at a nearby site, approximately ½-mile southeast of the subject property, using 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 California Building Code (CBC) Site Class in accordance with Chapter 16, Section 1613.3.2 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 1063 feet per second (ft/s). This weighted shear wave velocity corresponds to the higher range of Site Class D, as described in Chapter 20, Table 20.3-1 Site Classification of ASCE 7-16.

2.5.2.3 Groundwater Conditions

The groundwater table was encountered at depths ranging between 19 to 30 feet below ground surface in exploratory borings B21-1, B21-2, B21-4, B21-6, and B21-8. 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 Department of Water Resources Water Data Library database (wdl.water.ca.gov/waterdatalibrary) to review historical groundwater elevation data in the immediate area. Based on review of groundwater elevation data generated from a monitoring well located in the northeast portion of the project site, NV5 estimates that the historically high groundwater occurs at a depth of approximately 10 to 20 feet bgs in the late winter or spring during periods of above average and prolonged rainfall.

Fluctuations in groundwater elevation may also occur from agricultural irrigation in the area and the adjacent Burns Valley Creek

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 D422 Particle Size Gradation (Sieve Only)
- ASTM D2216 Soil Moisture Content
- ASTM D2487 Soil Classification by the USCS
- ASTM D2844 Resistance Value (R-Value)
- ASTM D2850 Unconsolidated-Undrained Triaxial Compression Test
- ASTM D2937 In Place Density of Soil
- ASTM D4318 Atterberg Limits (Dry Method)

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

Table 3.0-1, Laboratory Test Results

Boring	Sample		ASTM Test Results(1)								
No.	No.	Depth	D2487 D2488	D2216	D2937	D422		D4318		D2850	D2844
		(ft)	USCS (sym)	Moisture Content (%)	Dry Density (pcf)	Passing No. 4 Mesh Sieve (%)	Passing No. 200 Mesh Sieve (%)	Plasticity Index (%)	Liquid Limit (%)	UU Triaxial Compressive Strength (psf)	Resistance Value (R-Value) (dim)
B21-1	BK-1	0-3	SC	--	--	61.4	20.1	11	30	--	--
B21-1	B2-1-1	31.0	CH	--	--	--	--	31	54	--	--
B21-2	BK-2	1-3	CL	--	--	89.1	57.1	18	39	--	--
B21-2	L2-1-2	6.0	CL	16.1	100.8	--	--	--	--	--	--
B21-5	BK-4	0-4	ML	--	--	--	--	--	--	--	22
B21-8	L1-1-2	1.0	CL	18.5	101.6	--	--	--	--	1,538.51	--

Notes:
(1) Laboratory test forms are presented in Appendix C
% percent
ASTM ASTM International
dim dimensionless
ft feet
No. Number
Pcf pounds per cubic foot
psf pounds per square foot
sym symbol
UU Unconsolidated-Undrained
USCS Unified Soils Classification System

4.0 HISTORICAL SEISMICITY

The regional geology and faulting are discussed in Section 2 of this report. NV5 used the USGS National Earthquake Information Center (NEIC) Earthquake Search Results on-line database (http://earthquake.usgs.gov/earthquakes/search) to identify historical seismic activity within a 100 km (62 miles) radial distance of the subject site. A search for earthquakes was limited to moderate to strong events with a minimum magnitude of 5.0 local magnitude [M_L]). The results produced three recent events that occurred within 100 km of the site since 2014. These earthquakes include the following events:

- August 24, 2014, 6.0 M_L South Napa earthquake main shock occurred at approximately 03:20 hours in the Napa Valley. The earthquake epicenter was approximately 87 km (54 miles) south of the subject site. The earthquake damaged many structures in the Napa County and Sonoma County surrounding areas. The mean intensity estimated at the distance of the subject property ranged from 2.9 to 3.4, which indicates weak to light shaking and no damage.
- December 14, 2016, 5.0 M_L earthquake occurred approximately 8 km northwest of The Geysers, approximately 26 km (16 miles) southwest of the subject site. The event recorded a mean intensity of 4.1 at the distance to the subject site, which indicates light shaking and no damage.
- August 10, 2016, 5.1 M_L earthquake occurred approximately 20 km northeast of Upper Lake, approximately 34 km (21 miles) north-northwest of the subject site. The event recorded a mean intensity of 3.4 at the distance to the subject site, which indicates light shaking and no damage.

Additionally, a number of moderate to strong earthquakes were recorded within the past 150 years, although many of them occurred more than 100 years ago.

- 1962 and 1869, a 5.2M_L (1969) earthquake and a 5.0M_L (1869) earthquake occurred approximately 40km (25 miles) northwest of the subject site, near Ukiah.
- 1969 and 1893, 5.1M_L earthquakes occurred approximately 58 km (36 miles) south of the site, near Santa Rosa.
- 1898 and 1891, a 6.2M_L (1898) earthquake and a 5.5M_L (1891) earthquake occurred approximately 84 km (52 miles) south-southeast of the site, near Sonoma.
- 1968, a 5.0M_L earthquake occurred approximately 80 km (50 miles) from the site, in Glenn County.
- April 1892, three earthquakes (5.5M_L, 6.2M_L, and 6.4M_L) occurred approximately 89 km (55 miles) southeast of the site, near Vacaville.
- 1902, a 5.4M_L earthquake occurred approximately 100 km (62 miles) southeast of the site, near Fairfield.

The Geysers area, located approximately 24 km (15 miles) from the site, also is very active and produces dozens of small earthquakes, below magnitude of 4.0 M_L, on a daily to weekly basis.

5.0 LIQUEFACTION AND SEISMIC SETTLEMENT

NV5 did not perform a detailed evaluation of the potential for seismically induced soil liquefaction at the site. However, NV5 believes that the site has a low potential for soil liquefaction. The following supports our assessment.

5.1 LIQUEFACTION

Soil liquefaction results when the shear strength of a saturated soil decreases to zero during cyclic loading that is generally caused by machine vibrations or earthquake shaking. Generally, saturated, clean, loose, uniformly graded sand and loose, silty sand soils of Holocene age are the most prone to undergo liquefaction. However, saturated, gravelly soil and some silt and clay-rich soil may be prone to liquefaction under certain conditions. The onsite soil is Pleistocene to Holocene age soil consisting of Quaternary Alluvium (Q_{al}) primarily composed of stiff, damp to wet, cohesive soil and dense to very dense, damp to moist, sandy and silty gravels. Groundwater was encountered in exploratory borings B20-1 through B20-3 at depths of approximately 19 to 30 feet bgs. Groundwater data collected from nearby groundwater monitoring wells indicate the historical high groundwater table in the area may be encountered as shallow as approximately 10 feet bgs. NV5 considers 10 feet bgs to be the historical high groundwater elevation and used this data in the liquefaction analysis.

NV5 evaluated the liquefaction potential of the site using the procedures presented in the 2008 Earthquake Engineering Research Institute (EERI) Monograph publication *Soil Liquefaction During Earthquakes* by I. M. Idriss and R. W. Boulanger (Idriss, I. M. & Boulanger, R. W., 2008). It should be noted that NV5 used the maximum considered earthquake (MCE) modal magnitude $9M_w$ from a Cascadian subduction zone event. The shear stress reduction coefficient currently established does not use historical data from model magnitude $9M_w$, however current evaluations using recent magnitude 9M events are being evaluated. The determination of a shear stress reduction coefficient for a $9M_w$ earthquake exceeds the current model computations, therefore, NV5 conservatively assumed no stress reductions which is represented by an r_d value of 1 for all depths. This is a very conservative approach for liquefaction analyses.

The California Geological Society (CGS) Special Publication 117A suggests a minimum factor of safety (FS) of 1.3 for liquefaction analyses when using their ground motion maps. NV5 used a computed FS of less than 1.3 to indicate the occurrence of liquefaction at the site. The computed liquefaction FS for the project site soils ranged from 0.13 to greater than 2.0 for the soil layer intervals evaluated. The calculation spreadsheet of this analysis is included in Appendix D. Table 5.1-1 summarizes the findings of each borehole analyses using a depth to groundwater of 10 ft bgs.

Table 5.1-1, Liquefaction Potential Calculated From Borings

Assumed Groundwater Level (ft bgs)	Earthquake Magnitude (Mm)	Deterministic PGA (g)	Boring ID (No.)	Liquefaction Interval FS<1.3 (ft bgs)	Seismically Induced Settlement (inches)	Expected Manifestation (Yes/No)
10.0	9.0	0.628	B21-1	25 to 30	0.75	No
			B21-2	N/A	0.0	No

Notes
ft = feet
bgs = below ground surface
Mm = Moment Magnitude
g = gravitational acceleration

The liquefaction evaluation is a simplified procedure that has a number of limitations that cause it to produce conservative results. These limitations include the lack of a stress reduction coefficient (r_d) value for earthquake magnitudes over 8M, as well as the assumption that penetration resistance is a good indicator for liquefaction; however, other factors such as over consolidation and age of the deposit can influence the liquefaction potential. The procedure used does not take into account the age and over consolidation of the units.

Based on the subsurface exploratory boring 2.5-inch diameter California Modified split spoon sampler and standard penetration test (SPT) sampler blow counts, field data, expected seismic peak ground acceleration and literature review, NV5 believes the probability of liquefaction occurring during ground shaking caused by a maximum considered earthquake to be low at the site.

5.2 SEISMIC SETTLEMENT AND LATERAL SPREADING

The results of the liquefaction analysis performed for this investigation indicate a calculated seismic settlement of less than 1.0 inches. These settlement estimates represent ground settlement within the soil layers prone to liquefaction, not settlement at the ground surface.

Based on the relative flat terrain across the site and adjacent to the site and the existing development surrounding the site, NV5 considers there to be a low probability for the occurrence of lateral spreading that would be detrimental to the proposed site improvements.

6.0 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 were properly incorporated, are still applicable or need modifications.
2. Undocumented fills were observed in the southeastern portion of the site that extended to at least 36 inches feet bgs. These undocumented fills cannot be relied upon for support of the proposed improvements, due to their unknown quality, unknown method of placement, and potential for settlement. Recommendations for mitigating the undocumented fills are presented in Section 7.1 of this report.
3. Based on the site geology, the observations within the exploratory borings, the site soil profile can be modeled, according to the 2019 CBC, Chapter 16, and ASCE 7-16, Chapter 20, as a Site Class D (Stiff Soil Profile) designation for the purposes of establishing seismic design loads for the proposed improvements.
4. Based on the results of the liquefaction analyses, the subsurface exploratory boring blow counts, other field data, and literature review, NV5 believes that the probability of liquefaction occurring during a nearby earthquake to be low.
5. The site is comprised of Lake County Assessor's Parcel Numbers 010-026-40, 010-026-29 and 039-570-18. Each of the three parcels is described respectively.

   Parcel 010-026-40 is an irregular-shaped property generally comprised of an existing tree orchard and high concentrations of weeds and grasses. The terrain was relatively flat with a gentle downward slope from the east to the west. A drainage channel transected the east portion of the property in the southwest direction. To the east of the drainage channel the surface topography was relatively higher in elevation than the rest of the site. Large stockpiles consisting of soils, concrete and asphalt rubble, boulders, and other deleterious debris were present. Overhead power poles and power lines were present along the north and east boundaries of the property. The property was bounded to the east and north by Burns Valley Road; to the west by Burns Valley Creek; and, to the south by apartments, commercial buildings and a retail shopping center.

   Parcel 010-026-29 is a rectangular shaped property supporting a large number of mature oak trees, agricultural tress, and high concentrations of weeds and grasses. Concrete foundation remnants of a former structure and a large construction crane were present in the southern portion of the property. A drainage channel transected the center of the site and extended in the southwest direction. A California DWR monitoring well was present in the northeast portion of the site. A water well pump house was present in the northern half of the property. The site was bounded to the north and east by Burns Valley Road, to the south by fallow land and stockpiles; and, to the west by a senior living community.

Parcel 039-570-18 is a rectangular shaped property comprised of fallow land supporting low to moderate concentrations of weeds and grasses. Sparse mature trees and fence posts were present throughout the site. Numerous utility markings were present indicating the presence of underground utilities. The property is bounded to the north by existing tree orchards; to the west by an existing Pacific Gas & Electric (PG&E) facility; to the south by Olympic Drive; and, to the east by a retail shopping center. Evidence of a former structure was observed in the northern portion of the parcel.

6. The soil conditions observed to a maximum depth of 51.5 feet below the existing ground surface in our subsurface exploratory excavations (described relative to the existing ground surface) generally consisted of: dark yellowish brown, stiff, damp, sandy silt (ML); dark yellowish brown, medium dense, moist to damp, clayey sand (SC); brown, stiff to very stiff, moist, lean clay (CL); light gray, medium dense, wet, silty gravel (GM); dark greenish gray, firm, wet, fat clay (CH); gray, dense, very moist, poorly graded gravel (GP); and, dark grayish brown, medium dense, damp, silty sand.
7. NV5's field and laboratory test data indicates that the clayey sand (SC), lean clay (CL) and silt (ML) soil units encountered beneath the site has the following general geotechnical engineering properties: medium dense/stiff to very stiff, low plasticity and low to moderate bearing capacity that is suitable for supporting shallow foundations.
8. The groundwater table was encountered at depths ranging between 19 to 30 feet below ground surface in the exploratory borings B21-1, B21-2, B21-4, B21-6 and B21-8. Based on the above average rainfall, subsurface geologic conditions and review of monitoring well data near the site, NV5 assumes that for design and evaluation purposes, the historically high groundwater table will probably be encountered at a depth of approximately 10 to 20 feet bgs.

7.0 RECOMMENDATIONS

Undocumented fills were observed on the site and are not considered suitable for support of the proposed structural improvements. 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 apartment building may be founded on conventional continuous and/or spread footings founded in undisturbed native soils or properly compacted fill. NV5's recommendations are presented below.

7.1 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, cut-fill transitions, cut and fill slope grading, erosion controls, underground utility trenches, construction dewatering, soil corrosion potential, subsurface groundwater drainage, surface water drainage, grading plan review and construction monitoring.

7.1.1 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 foundation remnants and exterior concrete slab-on-grade within the proposed building areas should be razed and disposed off-site. 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 asphalt concrete (AC) and aggregate base (AB) rock demolition materials for use in constructing engineered fills.
2. All foundations, underground utilities and other existing site improvements 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.

7.1.2 Import Fill Soil

Import fill soil should meet the geotechnical engineering material properties described in Section 7.1.6.1 (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 engineer should approve all proposed import fill soil for use in constructing engineered fills at the site.

7.1.3 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.

7.1.4 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 4 to 6 inches of organic-laden topsoil and other deleterious materials from the building area. Remove all existing trees within the proposed building pad areas. 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. Completely remove all existing stockpiles, undocumented fill materials, concrete rubble, and other deleterious debris from the site. 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, other deleterious materials and oversized rock materials should be removed from the site.

7.1.5 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 Unified Soils Classification System (USCS) (i.e., GP, GW, GC, GM, SP, SW, SC or SM) then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified as a low plasticity fine-grained soil by the USCS (i.e., CL, ML), then it should be moisture conditioned to between 2 and 4 percentage points greater than the ASTM D1557 optimum moisture content. If soil is classified as a high plasticity fine-grained soil by the USCS (i.e., CH, MH), the soil should be removed from the building pad area or contact NV5 for further recommendations.
2. The native soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry unit weight (density). The moisture content, density

and relative percent compaction should be tested by the project engineer or his/her field representative to evaluate whether the compacted soil meets or exceeds the minimum percent compaction and moisture content requirements. The earthwork contractor shall assist the project engineer or his/her field representative by excavating test pads with the on-site earth moving equipment. Native soil preparation beneath concrete slab-on-grade structures (i.e., floors, sidewalks, patios, etc.) and AC pavement should be prepared as specified in Section 7.2 (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 engineer or his/her field representative to be firm, competent and relatively unyielding. The project engineer or his/her field representative may also evaluate the surface material by hand probing with a ¼-inch-diameter steel probe; however, this evaluation method should not be performed in place of proof rolling as described above.
4. Construction Quality Assurance (CQA) tests should be performed using the minimum testing frequencies presented in Table 7.1.5-1 or as modified by the project engineer to better suit the site conditions.
5. The native soil surface should be graded to minimize ponding of water and to drain surface water away from the building foundations and associated structures. Where possible, surface water should be collected, conveyed and discharged into natural drainage courses, storm sewer inlet structures, permanent engineered storm water runoff percolation/evaporation basins or engineered infiltration subdrain systems.

Table 7.1.5-1, 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 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

7.1.6 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 7.1.6.1. 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 7.1.6.2 or as modified by the project engineer. If soil is to be imported to the site for constructing engineered fills, then NV5 should be allowed to evaluate the suitability of the borrowed soil source by taking representative soil samples for laboratory testing. Testable earth materials are generally considered to be soils with gravel and larger particle sizes retained on the No. 4 mesh sieve that make up less than 30 percent by dry weight of the total mass. The relative percent compaction of testable earth materials can readily be determined by the following ASTM test procedures: laboratory compaction curve (D1557), field moisture and density (D6938). Construction of engineered fills with non-expansive and expansive testable earth materials is described below.

7.1.6.1 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 engineer or his/her field representative should approve the use of oversized materials for constructing engineered fills.
2. Non-expansive soil used to construct engineered fills should be uniformly moisture conditioned. If the soil is classified by the USCS as coarse grained (i.e., GP, GW, GC, GM, SP, SW, SC or SM), then it should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content. If the soil is classified by the USCS as fine grained (i.e., CL, ML), then it should be moisture conditioned to between 2 and 4 percentage points greater than the ASTM D1557 optimum moisture content.
3. Engineered fills should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose lifts (layers) prior to compacting.
4. The soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
5. The earthwork contractor should compact each loose soil lift with a tamping foot compactor such as a Caterpillar (CAT) 815 Compactor or equivalent as approved by NV5's project engineer or his/her field representative. A smooth steel drum roller compactor should not be used to compact loose soil lifts for construction of engineered fills.
6. The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 7.1.6.1-1 or as modified by the project engineer to better suit the site conditions.

Table 7.1.6.1-1, 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 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 engineer's field representative during construction to evaluate whether the compacted soil meets or exceeds the minimum compaction and moisture content requirements. The earthwork contractor shall assist the project engineer's field representative by excavating test pads with the on-site earth-moving equipment.
8. The prepared finished grade or finished subgrade soil surface should be proof-rolled as mentioned above in Section 7.1.5, Paragraph 3.

7.1.6.2 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.

7.1.7 Cut and Fill Slope Grading

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

7.1.8 Erosion Controls

Erosion controls should be installed as described below.

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

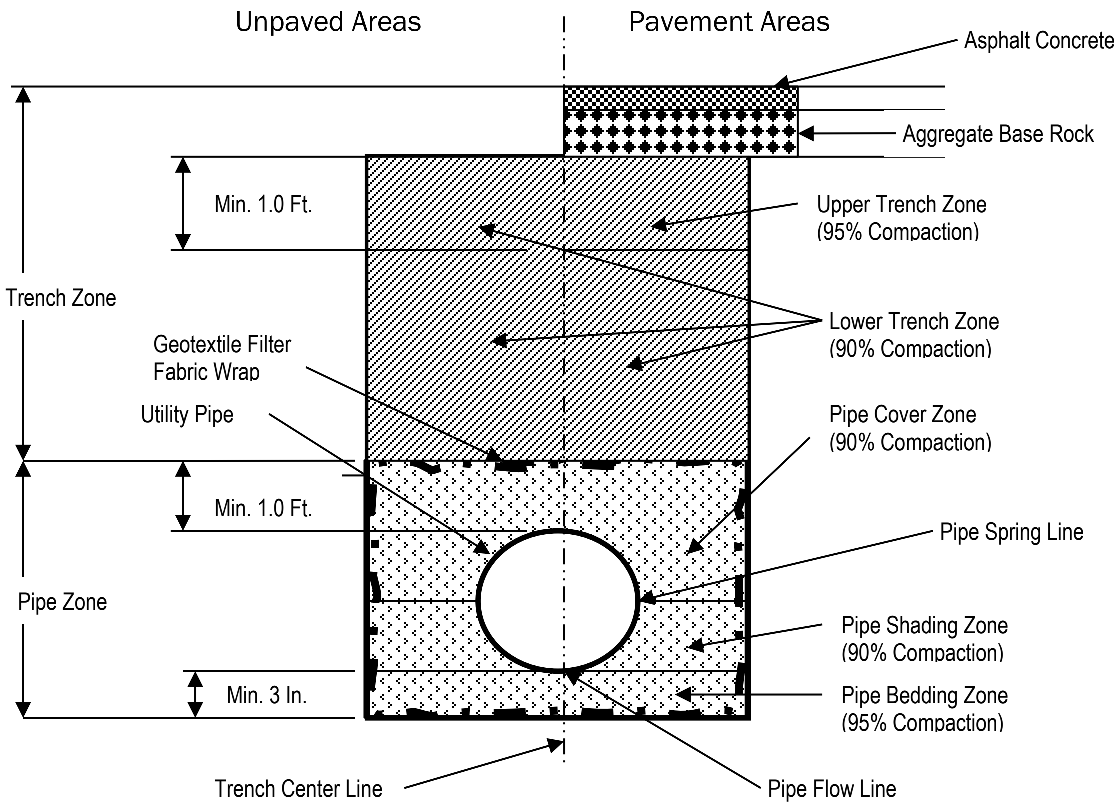
3. Install surface water drainage ditches at the top of cut and fill slopes (as necessary) to collect and convey both sheet flow and concentrated flow away from the slope face.
4. The intercepted surface water should be discharged into a natural drainage course or into other collection and disposal structures.

7.1.9 Underground Utility Trenches

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

1. **Trench Excavation Equipment:** NV5 anticipates that the contractor will be able to excavate all underground utility trenches with a Case 580 Backhoe or equivalent, however, deeper utility trenches (10-feet or greater) may require larger equipment.
2. **Trench Shoring:** All utility trenches that are excavated deeper than 5 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 7.1.10 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 7.1.9-1 below.

Detail 7.1.9-1 TYPICAL TRENCH BACKFILL ZONES



Not to Scale

- **Pipe Zone Backfill Material Type:** Trench backfill used within the pipe zone, which includes the bedding zone, the shading zone and the cover zone, should consist of ¾-inch-minus, washed, crushed rock, imported sand, or Class 2 AB. 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® Mirafi 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® Mirafi 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:** Crushed rock placed in the pipe bedding zone (beneath the utilities) should be consolidated using mechanical equipment to a firm unyielding condition. Imported sand or Class II AB 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:** Crushed rock placed within the pipe shading zone should be consolidated using mechanical equipment to a firm unyielding condition, shovel slicing material to support the pipe bells or haunches. Imported sand or Class II AB 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, support the pipe bells or haunches and to promote compaction.
- **Pipe Cover Zone Compaction:** Crushed rock placed within the pipe cover zone should be consolidated using mechanical equipment to a firm unyielding condition. Native soils, imported sand, and Class II AB 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:** Crushed rock placed within the lower trench zone should be consolidated using mechanical equipment to a firm unyielding condition. Soils, including imported sand and Class 2 AB, 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):** Crushed rock placed within the upper trench zone should be consolidated using mechanical equipment to a firm unyielding condition. Soils, including imported sand and Class 2 AB, 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):** Crushed rock placed within the upper trench zone should be consolidated using mechanical equipment to a firm unyielding condition. Soils, including imported sand and Class 2 AB, 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 7.1.9-1 or as modified by the project engineer to better suit the site conditions.

Table 7.1.9-1, Minimum Testing Frequencies for Utility Trench Backfill

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

Notes:
(1) These are minimum testing frequencies that may be increased or decreased at the project engineer's discretion 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 7.1.5, Paragraph 3.

7.1.10 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® Mirafi® 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.

7.1.11 Soil Corrosion Potential

The selected materials used for constructing underground utilities should be evaluated by a corrosion engineer for compatibility with the on-site soil and groundwater conditions. NV5 did not perform any testing to determine the corrosion potential of the shallow soils that are anticipated to be in contact with the underground pipes and concrete structures associated with the improvements. NV5's experience with soil encountered in the Clearlake area is that their corrosion potential is moderately corrosive. Buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion depending on the critical nature of the structure.

7.1.12 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.

7.1.13 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.

7.1.14 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 were 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.

7.2 STRUCTURAL IMPROVEMENTS

NV5's structural improvement design criteria recommendations include seismic design parameters, shallow foundations, retaining walls entirely above the groundwater table, retaining wall backfill, concrete slab-on-grade interior floors, sidewalk and patio construction, rigid concrete pavement for heavy truck traffic areas and fire lanes, and flexible pavement. These recommendations are presented hereafter.

7.2.1 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 Standard (erratum released February 2019) and the 2018 International Building Code (IBC). The spectral acceleration, site class, site coefficients and adjusted maximum considered earthquake spectral response acceleration, and design spectral acceleration parameters are presented in Table 7.2.1-1. The Seismic Design Parameter detailed report from the SEAOC analysis is provided in Appendix E.

7.2.1.1 Long-Period Seismic Site Coefficient (F_V)

Using Table 1613.2.3(2) of the 2019 CBC, NV5 calculated the long-period site coefficient (F_v) using S_1=0.541 and linear interpolation of the values presented in the table. Linear interpolating the values resulted in the following equations for calculating F_v:

- $F_v = (-2 \times S_1)+2.6$ (S_1 is less than 0.3)
- $F_v = (-1 \times S_1)+2.3$ (S_1 is greater than 0.3)

$$F_v = (-1 \times S_1) + 2.3 = (-1 \times 0.541) + 2.3 = 1.759$$

7.2.1.2 Seismic Design Category

Based on the short period response acceleration ground motion parameters (S_{DS} = 1.2), the 1-S period response acceleration ground motion parameters (S_{D1} = .634), and the Risk Category of I through III, the Seismic Design Category is D.

7.2.1.3 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:

$$PGA_M = F_{PGA}PGA = 1.2 \times 0.523 = 0.628 \text{ g}$$

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

7.2.1.4 Site-Specific Ground Motion Hazard Analysis

Based on the preliminary information provided to NV5 on the proposed building sizes and types, NV5 understands a ground motion hazard analysis is not required for the site provided the seismic response coefficient (C_s) is determined in accordance with Exception 2 found in Section 11.4.8 of ASCE 7-16.

Table 7.2.1-1 2019 CBC Seismic Design Parameters

Description	Value	Reference
Latitude North (degrees)	39.9638	Google Earth
Longitude West (degrees)	-121.6349	Google Earth
Site Coefficient, F_A	1.2	*2019 CBC, Table 1613.2.3(1),* SEAOC *Seismic Design Maps*
Site Coefficient, F_V	1.759	2019 CBC, *Table 1613.2.3(2),* SEAOC *Seismic Design Maps*
Site Class	D = Stiff Soil	ASCE 7-16 Chapter 20, Table 20.3-1
Short (0.2 sec) Spectral Response, S_S (g)	1.5	ASCE 7-16, Section 11.4.2, SEAOC Seismic Design Maps
Long (1.0 sec) Spectral Response, S_1 *(g)*	0.541	ASCE 7-16, Section 11.4.2, SEAOC Seismic Design Maps
Short (0.2 sec) MCE Spectral Response, S_{MS} *(g)*	1.8	ASCE 7-16, Section 11.4.4, SEAOC Seismic Design Maps
Long (1.0 sec) MCE Spectral Response, S_{M1} *(g)*	0.952	ASCE 7-16, Section 11.4.4, SEAOC Seismic Design Maps
Short (0.2 sec) Design Spectral Response, S_{DS} *(g)*	1.2	ASCE 7-16, Section 11.4.5, SEAOC Seismic Design Maps
Long (1.0 sec) Design Spectral Response, S_{D1} *(g)*	0.634	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)	0.628	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

7.2.2 Shallow Foundations

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

1. The base of all shallow foundations should bear on firm, competent non-expansive native soil, or non-expansive engineered fill compacted consistent with the earthwork recommendations of Section 7.1.
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 7.2.2-1.
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 7.2.2-1.
   - The calculated factor of safety 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 FS and confirm that it is not less than the over-strength factor for this structure.

Table 7.2.2-1, Foundation Bearing Pressures for Shallow 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
24	9,000	3,000	3,990	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 (EFP) per foot of depth. The passive pressure force and sliding friction coefficient for computing lateral resistance are as follows:
   a. Passive pressure = 225 (H), pounds per square foot (psf), where H = foundation embedment depth (feet) below lowest adjacent soil surface.
   b. Foundation bottom sliding friction coefficient = 0.30 (dimensionless).

5. Minimum steel reinforcement for continuous strip foundations should consist of four No. 4 bars with two bar placed near the top and two 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 his/her field representative should observe the excavations to document that the following requirements are 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.

7.2.3 Retaining Walls Entirely Above the Groundwater Table

A California licensed professional 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 native soils or engineered fill consistent with the requirements of Section 7.1.

3. The retaining wall should be designed using the geotechnical engineering design parameters presented in Table 7.2.3-1.

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 7.2.4.

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 7.2.3-1, Design Parameters for Retaining Walls

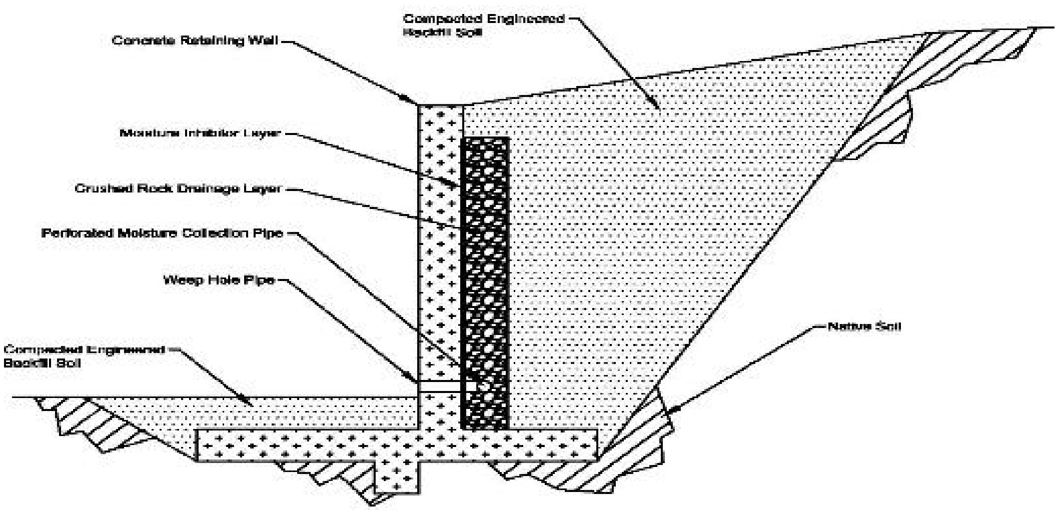
Design Parameters for Retaining Walls		
Loading Conditions	Static Loads On Retaining Wall With Horizontal Backfill Slope	Seismic Load On Retaining Wall With Horizontal Backfill Slope
Wall Active Condition Pressures (psf)/ft (1)	50 (H) (5)	9 (H^2)
Wall Passive Condition Pressures (psf)/ft (2)	225 (H)	9 (H^2)
Wall At-Rest Condition Pressure (psf)/ft (3)	70 (H)	21 (H^2)
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).

7.2.4 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 Mirafi 140N manufactured by Tencate Geosynthetics 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.

7.2.5 Concrete Slab-On-Grade Interior Floors, Sidewalk and Patio Construction

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

The concrete slab-on-grade building floors, patios, and sidewalk areas should be evaluated by a California-licensed professional 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 slab-on-grade concrete 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.

7.2.5.1 Interior Office Floors

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/A 615M-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-content ratio can be incorporated into the concrete slab-on-grade floor mix design to reduce its permeability and water-vapor transmissivity properties. A waterproofing consultant should be contacted to provide detailed recommendations if moisture sensitive flooring materials will be installed on the concrete slab-on-grade floors.

4. Minimum 4-Inch-Thick Crushed Rock or Class II Aggregate Base Rock Layer: Interior floors should be underlain by clean crushed rock. Crushed rock should be mechanically consolidated under the observation of NV5. 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 slab-on-grade concrete surfaces would include AB rock meeting the specification of Caltrans Class II AB. AB rock layers should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. Just prior to pouring the concrete slab, the rock layer should be moistened to a saturated surface dry (SSD) condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks.

   If the current property owner elects to eliminate the crushed rock or AB rock layer beneath the interior 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: All concrete slab-on-grade subgrade soil should be prepared and compacted consistent with the recommendations of Section 7.1. 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 a moisture content within ± 3 percentage points of the ASTM D1557 optimum moisture content.

6. Crack Control: 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; All concrete slab-on-grade surfaces and installed steel reinforcements should be observed and inspected by an NV5 construction monitor prior to pouring concrete.

8. Field Curing of Concrete: Prior to applying construction loads, all exposed concrete slab-on-grade floors should be moisture cured for a minimum of 7 days following placement of the concrete. If concrete is placed during the hot summer months when the ambient air temperatures may be as low as 50 to 60 degrees Fahrenheit (°F) in the early morning and in excess of 90 °F in the afternoon, then the contractor may need to implement special curing measures to reduce the development of shrinkage cracks. The concrete contractor is responsible for determining the appropriate curing process to be applied to the slab-on-grade floor.

7.2.5.2 Interior Floors with Vehicle Traffic

1. Minimum 6-Inch-Thick Concrete Slab: should be installed with a minimum 3,500 psi compressive strength after 28 days of curing. NV5 recommends that the concrete design uses a water to

cement ratio between 0.40 and 0.50 and should be placed with minimum and maximum slumps of 4 and 6 inches, respectively. The concrete mix design is the responsibility of the concrete supplier.

2. Concrete Slabs in Contact With Isolated Concrete Foundations: We do not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floors a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over the vapor-moisture retarder membrane and crushed rock of the slab support layers. The development of adverse thermal gradients may cause the development of significant orthogonal and/or circular shrinkage cracks around the isolated column foundations.

3. Steel Reinforcement: should be used to improve the load carrying capacity and 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.

   Steel Rebar: As a minimum, use No. 4 ribbed steel rebar (ASTM A615/A615M-18e1 Grade 60 deformed for reinforcement in concrete), tied and placed with 12-inch centers in both directions (perpendicular) and supported on concrete "dobies" to position the rebar in the center of the slab during concrete pouring.

4. Underslab Vapor-Moisture Retarder Membrane: should be placed 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 reduced water to content ratio can be incorporated into the concrete slab-on-grade floor mix design to reduce its permeability and water-vapor transmissivity properties. A waterproofing consultant should be contacted to provide detailed recommendations if moisture sensitive flooring materials will be installed on the concrete slab-on-grade floors.

5. Minimum 6-Inch-Thick Crushed Rock Layer or Class II Aggregate Base Rock Layer: Interior floors should be underlain by clean crushed rock. Crushed rock should be mechanically consolidated under the observation of NV5. 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 slab-

on-grade concrete surfaces would include AB rock meeting the specification of Caltrans Class II AB. AB rock layers should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. Just prior to pouring the concrete slab, the rock layer should be moistened to a SSD condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks.

If the current property owner elects to eliminate the crushed rock or AB rock layer beneath the interior 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.

6. Subgrade Soil Preparation: All concrete slab-on-grade subgrade soil should be prepared and compacted consistent with the recommendations of Section 7.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content within ± 3 percentage points of the ASTM D1557 optimum moisture content.
7. Crack Control: Crack control grooves should be installed during placement or saw cuts should be made in accordance with the ACI and 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).
8. Field Observations; All concrete slab-on-grade surfaces and installed steel reinforcements should be observed and inspected by an NV5 construction monitor prior to pouring concrete.
9. Field Curing of Concrete: Prior to applying construction loads, all exposed concrete slab-on-grade floors should be moisture cured for a minimum of 7 days following placement of the concrete. If concrete is placed during the hot summer months when the ambient air temperatures may be as low as 50 to 60 °F in the early morning and in excess of 90 °F in the afternoon, then the contractor may need to implement special curing measures to reduce the development of shrinkage cracks. The concrete contractor is responsible for determining the appropriate curing process to be applied to the slab-on-grade floor.

7.2.5.3 Exterior Sidewalks and Patios

1. Minimum 4-Inch-Thick Concrete Slab: should be installed with a minimum 2,500 psi compressive strength after 28 days of curing. NV5 recommends that the concrete design uses a water to cement ratio between 0.40 and 0.45 and should be placed with minimum and maximum slumps of 4 and 6 inches, respectively. The concrete mix design is the responsibility of the concrete supplier.
2. Concrete Slabs in Contact With Isolated Concrete Foundations: NV5 does not recommend that concrete slab-on-grade floors be placed in direct contact with the top surface of isolated column concrete foundations. Our experience is that during curing period of the concrete slab-on-grade floor a significant thermal gradient may develop between the portions of the slab placed directly on the typically more massive isolated column concrete foundations and the portions of the slab placed over a vapor-moisture retarder membrane and crushed rock layers. The development of adverse thermal gradients may cause the development of significant orthogonal and/or circular shrinkage cracks around the isolated column foundations.

3. Steel Reinforcement: should be used to improve the load carrying capacity and 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 or cured. Concrete slabs that will be subjected to heavy loads should be designed with steel reinforcements by a California licensed professional engineer.

   If the current property owner (developer) elects to eliminate the steel reinforcements from the 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.

4. Minimum 4-Inch-Thick Crushed Rock Layer: Exterior concrete slabs-on-grade should be underlain by clean crushed rock. Crushed rock should be mechanically consolidated under the observation of NV5. 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 slab-on-grade concrete surfaces would include AB rock meeting the specification of Caltrans Class II AB. AB rock layers should be placed and compacted to a minimum of 95 percent of the ASTM D1557 dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content. Just prior to pouring the concrete slab, the rock layer should be moistened to a SSD condition. This measure will reduce the potential for water to be withdrawn from the bottom of the concrete slab while it is curing and will help minimize the development of shrinkage cracks.

   If the current property owner elects to eliminate the crushed rock or AB rock layer beneath the interior 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: All concrete slab-on-grade subgrade soil should be prepared and compacted consistent with the recommendations of Section 7.1. 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 a moisture content within ± 3 percentage points of the ASTM D1557 optimum moisture content.

6. Crack Control: Crack control grooves should be installed during placement or saw cuts should be made in accordance with the ACI and 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; All concrete slab-on-grade surfaces and installed steel reinforcements should be observed and inspected by an NV5 construction monitor prior to pouring concrete.

7.2.6 Rigid Concrete Pavement for Heavy Truck Traffic Areas and Fire Lanes

The rigid concrete pavement 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. The recommended modulus of subgrade value of 150 kips/cubic foot should be used if the site subgrade is prepared in accordance with the recommendations presented in Section 7.1 above.
2. Minimum 6-Inch-Thick Concrete Slab: The rigid concrete pavement should be installed with a minimum 3,500 pounds psi compressive strength after 28 days of curing. NV5 recommends that the concrete design uses a water-to-cement ratio between 0.40 and 0.45 and should be placed with minimum and maximum slumps of 4 and 6 inches, respectively. The concrete mix design is the responsibility of the concrete supplier.
3. Steel Reinforcements: The rigid concrete pavement sections should include steel reinforcement to improve the load carrying capacity and to minimize 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. Rigid concrete pavement that will be subjected to heavy loads should be designed with steel reinforcements by a California-licensed professional engineer.

   If the owner elects to eliminate the steel reinforcements from the 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.
4. Steel Rebar: Use No. 4 steel rebar (ASTM A615/A615M-18e1 Grade 60 reinforcement), 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.
5. Minimum 6-Inch Caltrans Class II AB Layer: The rigid concrete pavement should be underlain by Class II AB 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.
6. Subgrade Soil Preparation: The subgrade soil below the rigid concrete pavement sections designed for vehicle traffic should be prepared and compacted consistent with the recommendations of Section 7.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 95 percent of the ASTM D1557 dry density with a relatively uniform moisture content of 0 to 4 percentage points greater than the ASTM D1557 optimum moisture content.
7. Crack Control Grooves: The rigid concrete pavement should include crack control and expansion joint grooves installed during placement or saw cuts should be made in accordance with the ACI and 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 no greater than 10-foot-centers in both directions (perpendicular).
8. Field Observations: Field observations should be made by an NV5 construction monitor of all concrete slab-on-grade subgrade surfaces and installed steel reinforcements prior to placing concrete.

7.2.7 Flexible Pavement

NV5 used the Caltrans Highway Design Manual to develop several AC and AB rock pavement design alternatives to allow for different traffic loading conditions. NV5 used a Traffic Index (TI) of 4 to 8 which represents typical vehicle traffic for residential streets, collector streets, industrial/commercial streets, minor arterial streets, major arterial streets, and truck route arterial streets. The actual TI for the project pavement areas should be determined in accordance with Chapter 600 of the Caltrans Highway Design Manual.

Laboratory test results performed on a representative sample of the anticipated pavement subgrade soils within the proposed pavement improvements indicate these materials generally possess an R-Value of 22. Based on the fair quality near-surface soils encountered an R-Value of 20 should be considered for design purposes. The actual subsurface soil conditions exposed at the finished subgrade surface of the proposed pavement areas may be different from this R-Value based on site grades, or the use of imported fill materials. The actual finished subgrade materials should be evaluated during construction to confirm the design recommendations below. Please note that the Caltrans design method requires that the maximum R-Value of the subgrade soil not exceed 50.

NV5 assumed that the pavement layers will be constructed with Class 2 Aggregate Base Rock (Minimum R-Value = 78) and Type A Asphalt Concrete in accordance with the requirements of Section 26 of the Caltrans Standard Specifications. Table 7.2.7-1 presents the AC pavement design sections for varying TI's. NV5 recommends that the AB rock layer be constructed with a minimum thickness of 6-inches for constructability issues and to achieve a higher level of confidence that the road will achieve the expected service life.

Table 7.2.7-1, Flexible Pavement Design

Parameters	Design Values				
Traffic Description (approximate)	Light Automobiles	Light to Medium Autos and Trucks	Medium to Heavy Trucks	Heavy Trucks	Very Heavy Trucks
Traffic Index (TI)	4	5	6	7	8
Design R-Values Class II AB Rock Subgrade Soil	78 20	78 20	78 20	78 20	78 20
AC Thickness (inch)[1]	2.5	3.0	3.5	4.0	5.0
AB Rock Thickness (inch)[2] (95% Relative Compaction)	6.0	8.0	10.0	12.0	14.0
Subgrade Soil Thickness (inch) (95% Relative Compaction)	12.0	12.0	12.0	12.0	12.0

Notes:
(1) The asphalt concrete thickness includes the Caltrans safety factor.
(2) NV5 recommends that the minimum thickness of AB rock should be 6 inches regardless of what the Caltrans design method indicates. This minimum thickness is necessary for constructability issues and will increase the level of confidence that the roads will achieve the expected service life.

The subgrade soil and AB rock should be placed and compacted as described below.

1. The subgrade soil to a depth of 12 inches from the finished grade surface should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density with a moisture content of 2 to 4 percentage points of the ASTM D1557 optimum moisture content. The compacted sub-grade soil shall be graded to achieve the design grades and tolerances.
2. The stability of the compacted subgrade soil should be evaluated by wheel rolling prior to placing the overlying AB rock layer. Wheel rolling should be performed with a fully loaded water truck with tire pressures between 60 and 95 psi. The subgrade soil surface should exhibit only minor deflections as the wheel load passes by. Any unstable areas should be reworked and then retested for percent relative compaction and percent moisture content and then proof rolled again. This process should be repeated until the area appears to be relatively stable.
3. The Caltrans Class II AB rock should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density with a moisture content of ± 3 percentage points of the ASTM D1557 optimum moisture content.
4. The stability of the compacted AB rock should be evaluated by wheel rolling prior to placing the overlying AC layer. Wheel rolling should be performed with a fully loaded water truck with tire pressures between 60 and 95 psi. The AB rock surface should exhibit only minor deflections as the wheel load passes by. Any unstable areas should be reworked and then retested for percent relative compaction and percent moisture content and then proof rolled again. This process should be repeated until the area appears to be relatively stable.
5. Concrete cut-off curbs should be constructed around all landscaped areas that are adjacent to AC paved driveways and parking areas. The curbs should extend to a minimum depth of 8 inches into the underlying subgrade soil. The extended curbs will reduce migration of irrigation and rain waters originating in the landscaped areas from entering the AB rock materials underlying the AC pavement material. This design is intended to minimize failures of the paved areas due to saturation of the underlying AB rock and subgrade soils.

8.0 REFERENCES

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9.0 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. However, additional fieldwork and laboratory testing may be required for NV5 to develop any modifications to the recommendations. The cost to review project changes and perform additional fieldwork and laboratory testing necessary to modify the recommendations is beyond the scope-of-services presented in this report. Any additional work will be performed only after receipt of an approved scope-of-work, budget and written authorization to proceed.
5. The analyses, conclusions and recommendations presented in this report are based on the site conditions as they existed at the time NV5 performed the surface and subsurface field investigations. NV5 has assumed that the subsurface soil and groundwater conditions encountered at the location of the exploratory borings are generally representative of the subsurface conditions throughout the entire project site; however, if the actual subsurface conditions encountered during construction are different than those described in this report, then NV5 should be notified immediately so that we can review these differences and, if necessary, modify the recommendations.
6. The elevation or depth to the groundwater table underlying the project site may differ with time and location; therefore, the depth to the groundwater table encountered in the exploratory borings is only representative of the specific time and location where it was observed.
7. The project site map shows approximate exploratory excavation locations as determined by pacing distances from identifiable site features; therefore, their locations should not be relied upon as being exact nor located with the accuracy of a California-licensed land surveyor.
8. NV5's geotechnical investigation scope-of-services did not include an evaluation of the project site for the presence of hazardous materials. Although NV5 did not observe the presence of

hazardous materials at the time of the field investigation, all project personnel should be careful and take the necessary precautions in the event hazardous materials are encountered during construction.

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

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, you can benefit from a lowered 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 techniques that can be of genuine benefit for 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 for Specific Purposes, Persons, and Projects, and At Specific Times

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 Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> 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 <u>Not</u> 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 Moisture Infiltration and Mold

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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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

APPENDIX B:

Exploratory Boring Logs

EXPLORATORY BORING LOG

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

Boring No. **B21-1**

Sheet: 1 **Of** 3

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-12-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1360.00		**Finish Date:** 1-12-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 51.5	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-12-21			
Time (24 Hour)	11:10			
Depth (Ft.)	23.0			

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
8:55			HSA			0	(SC) CLAYEY SAND WITH GRAVEL, Fld. Est.: 50% Fine to Medium Sand, 30% Gravel, and 20% Low Plastic Clay-Silt Fines; Dark Yellowish Brown (10YR, 4/4); Medium Dense; Damp.
					BK-1	1	
8:56		6	2.5SS			2	
		6			L1-2-2		
	3.0	10		0.75/1.5	L1-1-2	3	
			HSA			4	
9:04		7	2.5SS			5	
		14			L2-2-2		
	4.5	19		0.8/1.5	L2-1-2	6	Roots Encountered
			HSA			7	
						8	
						9	(CL) SANDY CLAY, FLD. EST: 60% Low Plastic Clay-Silt Fines, 30% Fine Sand, and 10% Gravel; Light Olive Brown (2.5YR, 5/4); Hard; Damp.
9:15		6	2.5SS			10	
		19			L3-2-2		
	4.5	29		0.9/1.5	L3-1-2	11	
			HSA			12	
						13	Increased Drill Effort
						14	
9:24		9	2.5SS			15	
		8			L4-2-2		
	2.75	8		1.2/1.5	L4-1-2	16	Yellowish Brown (10YR, 5/6); Stiff; Damp; Reddish Brown Mottling
			HSA			17	
						18	
						19	
						20	

NOTES: SPT - Standard Penetration Test
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. B21-1 | **Sheet:** 2 **Of** 3

Project Name: Proposed Burns Valley Development | **Project No.:** 71075.00.001 | **Task:** 001 | **Start Date:** 1-12-21

Location: Burns Valley Road, Clearlake, California | **Estimated Ground Surface Elevation (Ft. AMSL):** 1360.00 | **Finish Date:** 1-12-21

Logged By: Santiago Carrillo | **Drilling Cmpny:** Taber Drilling | **Drill Rig Type:** CME-55

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

Boring Dia. (In.): 8.00 | **Total Depth (Ft.):** 51.5 | **Backfill or Well Design:** Neat Cement Grout

Ground Water Information

Date	1-12-21			
Time (24 Hour)	11:10			
Depth (Ft.)	23.0			

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
9:39		2	2.5SS			20	(CL) SANDY CLAY, FLD. EST: 60% Low Plastic Clay-Silt Fines, 30% Fine Sand, and 10% Gravel; Light Olive Brown (2.5Y, 5/4); Hard; Moist; Weakly Cemented.
		23			L5-2-2	21	
	1.5	43		1.5/1.5	L5-1-2		
			HSA			22	
						23	Hard Drilling
						24	
9:58		15	SPT			25	(GM) SILTY GRAVEL WITH SAND, Fld. Est.: 60% Gravel; 20% Fine Sand; and 20% Low Plastic Clay-Silt Fines; Light Gray (10YR, 7/1); Medium Dense; Wet.
		10		0.3/1.5	B1-1-1	26	
		3					
			HSA			27	
						28	
						29	
10:11		2	SPT			30	(CH) FAT CLAY WITH SAND, FLD. EST: 85% High Plastic Clay-Silt Fines and 15% Fine Sand; Dark Greenish Gray (GLEY 1, 4/1); Firm; Wet.
		2				31	
	1.5	3		1.5/1.5	B2-1-1		
			HSA			32	
						33	
						34	
10:21		7	SPT			35	
		15				36	
	3.0	20		1.5/1.5	B3-1-1		Hard
			HSA			37	
						38	
						39	
						40	

NOTES: SPT - Standard Penetration Test
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. **B21-1**
Sheet: 3 **Of** 3

Project Name: Proposed Burns Valley Development | **Project No.:** 71075.00.001 | **Task:** 001 | **Start Date:** 1-12-21

Location: Burns Valley Road, Clearlake, California | **Estimated Ground Surface Elevation (Ft. AMSL):** 1360.00 | **Finish Date:** 1-12-21

Logged By: Santiago Carrillo | **Drilling Cmpny:** Taber Drilling | **Drill Rig Type:** CME-55

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

Boring Dia. (In.): 8.00 | **Total Depth (Ft.):** 51.5 | **Backfill or Well Design:** Neat Cement Grout

Ground Water Information

Date	1-12-21			
Time (24 Hour)	11:10			
Depth (Ft.)	23.0			

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
10:37		5	SPT			40	(CH) FAT CLAY WITH SAND, FLD. EST: 85% High Plastic Clay-Silt Fines and 15% Fine Sand; Dark Greenish Gray (GLEY 1, 4/1); Very Stiff; Wet.
		9					
	1.5	9		1.5/1.5	B4-1-1	41	
			HSA				
						42	
						43	
						44	
10:52		4	SPT			45	
		6					
	1.0	10		1.5/1.5	B5-1-1	46	Stiff
			HSA				
						47	
						48	
						49	
11:10		5	SPT			50	
		10					
11:10	2.0	12		1.0/1.5	B6-1-1	51	Increase in Sand Content; Very Stiff
						52	
						53	
						54	
						55	
						56	
						57	
						58	
						59	
						60	

NOTES: SPT - Standard Penetration Test
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. **B21-2**

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-12-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1352.00		**Finish Date:** 1-12-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling	**Drill Rig Type:** CME-55	**Sheet:** 1 **Of** 3
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)	**Hammer Type:** 140 Pound Auto Trip Hammer	
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 51.5	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-12-21			
Time (24 Hour)	15:38			
Depth (Ft.)	30.0			

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
13:56			HSA			0	(CL) SANDY CLAY, Fld. Est.: 60% Low Plastic Clay-Silt Fines, 30% Fine Sand, and 10% Gravel; Dark Yellowish Brown (10YR, 3/6); Very Stiff; Moist.
					BK-2	1	
14:04		7	2.5SS			2	
		11			L1-2-2	3	
	4.5	12		0.8/1.5	L1-1-2		
			HSA			4	
14:10		6	2.5SS			5	
		12			L2-2-2	6	
	4.5	16		1.0/1.5	L2-1-2		
			HSA			7	
						8	
						9	
14:17		5	2.5SS			10	
		11			L3-2-2	11	
	4.5	18		1.0/1.5	L3-1-2		Weakly Cemented
			HSA			12	
						13	
						14	
14:23		3	2.5SS			15	
		7			L4-2-2	16	
	2.0	11		1.2/1.5	L4-1-2		Black Mottling
			HSA			17	
						18	
						19	
						20	

NOTES: SPT - Standard Penetration Test
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. **B21-2**

Sheet: 2 **Of** 3

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-12-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1352.00		**Finish Date:** 1-12-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 51.5	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-12-21			
Time (24 Hour)	15:38			
Depth (Ft.)	30.0			

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.)
14:31		2	2.5SS			20
		4			L5-2-2	21
	0.5	6		1.4/1.5	L5-1-2	
			HSA			22
						23
						24
14:46		7	SPT			25
		9				26
	2.0	9		1.5/1.5	B1-1-1	
			HSA			27
						28
						29
14:49		2	SPT			30
		2				31
	.5	2		1.5/1.5	B2-1-1	
			HSA			32
						33
						34
15:00		6	SPT			35
		9				36
		10		1.5/1.5	B3-1-1	
			HSA			37
						38
						39
						40

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.

(CL) SANDY CLAY, Fld. Est.: 60% Low Plastic Clay-Silt Fines, 30% Fine Sand, and 10% Gravel; Dark Yellowish Brown (10YR, 3/6); Firm; Moist to Very Moist.

(GP) POORLY GRADED GRAVELS, Fld. Est.: 80% Gravel; 10% Fine Sand; and 10% Low Plastic Clay-Silt Fines; Gray (10YR, 5/1); Dense; Very Moist.

(CL) LEAN CLAY WITH SAND, FLD. EST: 85% Low Plastic Clay-Silt Fines and 15% Fine Sand; Dark Greenish Gray (GLEY 1, 4/1); Soft; Wet.

(SM) SILTY SAND, FLD. EST: 55% Fine Sand and 45% Low Plastic Clay-Silt Fines; Dark Grayish Brown (2.5YR, 4/2); Medium Dense; Wet.

(CH) FAT CLAY, FLD. EST: 95% High Plastic Clay-Silt Fines and 5% Fine Sand; Dark Greenish Gray (GLEY 1, 4/1); Stiff; Wet.

NOTES: SPT - Standard Penetration Test
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. B21-2

Sheet: 3 **Of** 3

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-12-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1352.00		**Finish Date:** 1-12-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 51.5	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-12-21			
Time (24 Hour)	15:38			
Depth (Ft.)	30.0			

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.)	Sample Interval And Symbol	Graphic Log	Soil And/Or Rock Material Descriptions
15:13		3	SPT			40			(CH) FAT CLAY, FLD. EST: 95% High Plastic Clay-Silt Fines and 5% Fine Sand; Dakr Greenish Gray (GLEY 1, 4/1); Stiff; Wet.
		6							
	1.75	8		1.5/1.5	B4-1-1	41			
			HSA						
						42			
						43			
						44			
15:25		3	SPT			45			
		5							
	1.0	7		1.5/1.5	B5-1-1	46			
			HSA						
						47			
						48			
						49			
15:37		6	SPT			50			
		6							
15:38	1.5	10		1.5/1.5	B6-1-1	51			Increase in Sand Content
						52			
						53			
						54			
						55			
						56			
						57			
						58			
						59			
						60			

NOTES: SPT - Standard Penetration Test
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. **B21-3**

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-13-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1352.00		**Finish Date:** 1-13-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling	**Drill Rig Type:** CME-55	**Sheet:** 1 **Of** 1
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)	**Hammer Type:** 140 Pound Auto Trip Hammer	
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 15.0	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-13-21			
Time (24 Hour)	9:00			
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
8:28			HSA			0	(SC) CLAYEY SAND WITH GRAVEL, Fld. Est.: 55% Fine Sand, 20% Low Plastic Clay-Silt Fines, and 25% Gravel; Dark Yellowish Brown (10YR, 4/6); Medium Dense; Moist to Damp.
8:31		5	2.5SS		BK-3	1	
		5			L1-2-2		
		8		0.9/1.5	L1-1-2	2	
			HSA				
8:40		5	2.5SS			3	
		7			L2-2-2	4	
		6		0.75/1.5	L2-1-2		
			HSA			5	
						7	(CL) LEAN CLAY WITH SAND, Fld. Est.: 90% Low Plastic Clay-Silt Fines and 10% Fine Sand; Dark Brown (10YR, 5/3); Very Stiff; Moist.
8:50		5	2.5SS			8	
		9			L3-2-2	9	
	4.25	11		1.25/1.5	L3-1-2		
			HSA			10	
9:00		3	2.5SS			13	
		7			L4-2-2	14	
9:00	2.25	9		1.5/1.5	L4-1-2	15	Increase in Sand Content; Stiff; Very Moist.

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. **B21-4**

Sheet: 1 **Of** 1

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-13-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1355.00		**Finish Date:** 1-13-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 21.5	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-13-21			
Time (24 Hour)	10:06			
Depth (Ft.)	20.0			

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
9:24			HSA			0	(CL) LEAN CLAY WITH SAND, Fld. Est.: 85% Low Plastic Clay-Silt Fines and 15% Fine Sand; Brown (10YR, 4/3); Stiff; Moist.
9:26		5	2.5SS			2	
		9			L1-2-2		
	4.5	10		1.2/1.5	L1-1-2	3	Very Stiff
			HSA				
9:31		14	2.5SS			5	
		27			L2-2-2		
	4.5+	38		1.3/1.5	L2-1-2	6	Hard
			HSA				
						8.5	(SC) CLAYEY SAND WITH GRAVEL, Fld. Est.: 65% Fine Sand, 25% Low Plastic Clay-Silt Fines, and 10% Gravel; Strong Brown (7.5YR, 4/6); Medium Dense; Moist.
9:41		10	2.5SS			10	
		12			L3-2-2		
	4.5+	17		.8/1.5	L3-1-2	11	
			HSA				
9:52		9	2.5SS			15	
		5			L4-2-2		
	1.5	7		1.1/1.5	L4-1-2	16	(CL) LEAN CLAY, Fld. Est.: 90% Low Plastic Clay-Silt Fines and 10% Fine Sand; Dark Grayish Brown (10YR, 4/2); Firm; Moist; Orange Mottling.
			HSA				
						19.5	(SM) SILTY SAND, Fld. Est.: 80% Fine Sand and 15% Low Plastic Clay-Silt Fines, and, 5% Gravel; Brown (10YR, 4/3); Dense; Wet.
10:04		16	2.5SS			20	
		29			L5-2-2		
10:06		16		.8/1.5	L5-1-2	21	

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. **B21-5** **Sheet:** 1 **Of** 1

Project Name: Proposed Burns Valley Development | **Project No.:** 71075.00.001 | **Task:** 001 | **Start Date:** 1-13-21

Location: Burns Valley Road, Clearlake, California | **Estimated Ground Surface Elevation (Ft. AMSL):** 1360.00 | **Finish Date:** 1-13-21

Logged By: Santiago Carrillo | **Drilling Cmpny:** Taber Drilling | **Drill Rig Type:** CME-55

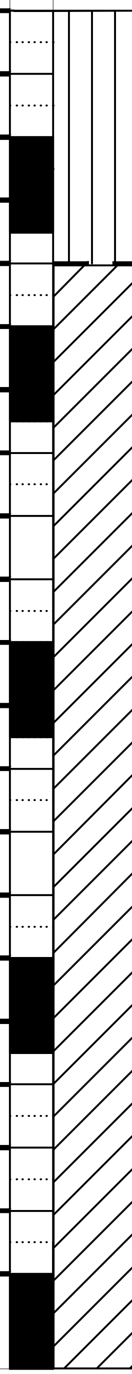
Driller: Toby Baldazo | **Drilling Method:** Hollow Stem Auger (HSA) | **Hammer Type:** 140 Pound Auto Trip Hammer

Boring Dia. (In.): 8.00 | **Total Depth (Ft.):** 21.5 | **Backfill or Well Design:** Neat Cement Grout

Ground Water Information

Date	1-13-21			
Time (24 Hour)	11:07			
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.	Depth B.G.S. (Ft.)	Soil And/Or Rock Material Descriptions
10:30			HSA			0	(ML) SANDY SILT, FLD. EST: 70% Low Plastic Clay-Silt Fines and 30% Fine Sand; Dark Yellowish Brown; (10YR, 4/4); Stiff; Damp.
					BK-4	1	
10:32		4	2.5SS			2	
		6			L1-2-2		
	4.25	8		.9/1.5	L1-1-2	3	
			HSA				
						4	(CL) LEAN CLAY WITH SAND, Fld. Est.: 80% Low Plastic Clay-Silt Fines and 20% Fine Sand; Brown (10YR, 4/4); Very Stiff; Damp to Moist.
10:36		8	2.5SS			5	
		10			L2-2-2		
	4.5+	17		.9/1.5	L2-1-2	6	
			HSA				
						7	
						8	
						9	
10:45		10	2.5SS			10	
		16			L3-2-2		
		19		.7/1.5	L3-1-2	11	
			HSA				
						12	
						13	
						14	
10:56		3	2.5SS			15	
		6			L4-2-2		
	.75	6		1.4/1.5	L4-1-2	16	Firm
			HSA				
						17	
						18	
						19	
11:07		4	2.5SS			20	
		6			L5-2-2		
11:07	1.75	8		1.5/1.5	L5-1-2		Firm to Stiff



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

EXPLORATORY BORING LOG

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

Boring No. **B21-6**

Sheet: 1 **Of** 2

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-13-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1356.00		**Finish Date:** 1-13-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 25.0	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-13-21			
Time (24 Hour)	12:10			
Depth (Ft.)	18.0			

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
11:30			HSA			0	(CL) LEAN CLAY WITH SAND, Fld. Est.: 90% Low Plastic Clay-Silt Fines and 10% Fine Sand; Dark Brown (10YR, 5/3); Very Stiff; Moist.
11:32		6	2.5SS			1	
		11			L1-2-2		
	4.5	12		1.2/1.5	L1-1-2	2	
			HSA			3	
11:36		8	2.5SS			4	
		13			L2-2-2		
	4.5	16		1.5/1.5	L2-1-2	5	
			HSA			6	
						7	
						8	
11:45		5	2.5SS			9	
		14			L3-2-2		
	4.5	16		1.4/1.5	L3-1-2	10	
			HSA			11	
						12	
						13	(SC) CLAYEY SAND WITH GRAVEL, Fld. Est.: 45% Fine Sand, 35% Low Plastic Clay-Silt Fines, and 20% Gravel; Brown (10YR, 4/3); Medium Dense; Moist.
11:52		16	2.5SS			14	
		16			L4-2-2		
	4.5+	12		1.0/1.5	L4-1-2	15	
			HSA			16	
						17	
						18	
11:59		5	2.5SS			19	
		26			L5-2-2		
		17		1.0/1.5	L5-1-2	20	Dense; Wet

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

EXPLORATORY BORING LOG

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

Boring No. B21-6

Sheet: 2 **Of** 2

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-13-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1357.00		**Finish Date:** 1-13-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 25.0	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-13-21			
Time (24 Hour)	12:10			
Depth (Ft.)	18.0			

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
			HSA			20	(SC) CLAYEY SAND WITH GRAVEL, Fld. Est.: 45% Fine Sand, 35% Low Plastic Clay-Silt Fines, and 20% Gravel; Brown (10YR, 4/3); Dense; Moist.
						21	
						22	(CH) FAT CLAY, FLD. EST: 90% High Plastic Clay-Silt Fines and 10% Fine Sand; Dark Greenish Gray (GLEY 1, 4/1); Stiff; Wet.
						23	
12:10		3	2.5SS				
		4			L6-2-2	24	
12:10	1.75	6		1.5/1.5	L6-1-2	25	
						26	
						27	
						28	
						29	
						30	
						31	
						32	
						33	
						34	
						35	
						36	
						37	
						38	
						39	
						40	

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. **B21-7**

Sheet: 1 **Of** 1

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-13-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1365.00		**Finish Date:** 1-13-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling		**Drill Rig Type:** CME-55
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)		**Hammer Type:** 140 Pound Auto Trip Hammer
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 21.5	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-13-21			
Time (24 Hour)	13:33			
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
12:56			HSA			0	(FILL) Undocumented Fill; Rocks;Garbage; Organics.
						3	(CL) LEAN CLAY WITH SAND, Fld. Est.: 80% Low Plastic Clay-Silt Fines and 20% Fine Sand; Dark Brown (10YR, 3/3); Stiff; Moist.
13:01		9	2.5SS			5	
		9			L1-2-2		
	4.5	8		.6/1.5	L1-1-2	6	
			HSA				
13:09		8	2.5SS			10	
		12			L2-2-2		
	4.5	15		.8/1.5	L2-1-2	11	Very Stiff
			HSA				
13:19		14	2.5SS			15	
		25			L3-2-2		
		30		.9/1.5	L3-1-2	16	Increase Gravels; Hard; Moist
			HSA				
13:32		4	2.5SS			20	
		7			L4-2-2		
13:33		10		1.5/1.5	L4-1-2	21	Light Olive Brown (2.5Y, 5/6); Stiff

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. **B21-8**

Sheet: 1 **Of** 1

Project Name: Proposed Burns Valley Development	**Project No.:** 71075.00.001	**Task:** 001	**Start Date:** 1-13-21
Location: Burns Valley Road, Clearlake, California	**Estimated Ground Surface Elevation (Ft. AMSL):** 1363.00		**Finish Date:** 1-13-21
Logged By: Santiago Carrillo	**Drilling Cmpny:** Taber Drilling	**Drill Rig Type:** CME-55	
Driller: Toby Baldazo	**Drilling Method:** Hollow Stem Auger (HSA)	**Hammer Type:** 140 Pound Auto Trip Hammer	
Boring Dia. (In.): 8.00	**Total Depth (Ft.):** 20.0	**Backfill or Well Design:** Neat Cement Grout	

Ground Water Information

Date	1-13-21			
Time (24 Hour)	14:41			
Depth (Ft.)	19.0			

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
14:05			HSA			0	(CL) LEAN CLAY WITH SAND, Fld. Est.:85% Low Plastic Clay-Silt Fines and 15% Fine Sand; Dark Yellowish Brown (10YR, 4/4); Stiff; Damp.
14:08		4	2.5SS		BK-5	1	
		9			L1-2-2		
	4.5+	14		1.1/1.5	L1-1-2	2	
			HSA				
						3	
14:11		13	2.5SS				
		17			L2-2-2	4	
	4.5+	21		1.5/1.5	L2-1-2		Hard
			HSA			5	
						6	
						7	
						8	
14:22		8	2.5SS				
		14			L3-2-2	9	
	4.25	18		1.5/1.5	L3-1-2		Dark Brown (10YR, 3/3); Very Stiff; White Mottling
			HSA			10	
						11	
						12	
						13	
14:32		4	2.5SS				
		7			L4-2-2	14	
	3.5+	8		1.5/1.5	L4-1-2		Stiff; Black Mottling
			HSA			15	
						16	
						17	
						18	
14:41		3	2.5SS				
		6			L5-2-2	19	
14:41		11		1.2/1.5	L5-1-2	20	Increase Sand Content; Brown (10YR, 4/4)

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

APPENDIX C:

Soil Laboratory Test Results

ATTERBERG INDICES

ASTM D4318

DSA LEA No. 284

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

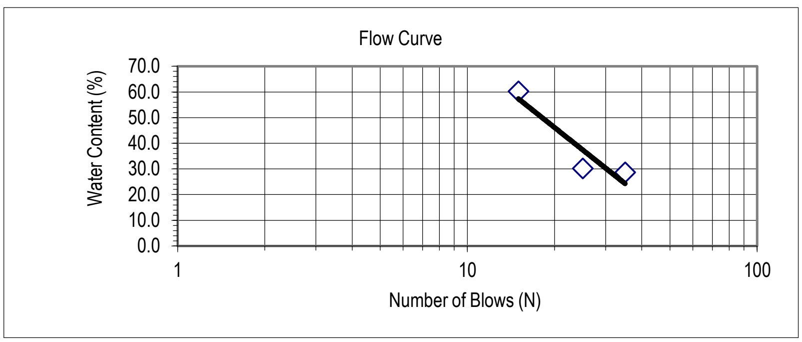
Project No.	71075.00.001	Project Name	City of Clearlake Burns Valley Development	Date:	01/20/21
Sample No.	BK-1	Boring/Trench	B21-1 Depth, (ft.): 0-3	Tested By:	LGH
Description:	(SC) CLAYEY SAND WITH GRAVEL; Yellowish Brown (10YR, 4/4)			Checked By:	DJP
Sample Location:				Lab. No.	C21-014

Estimated % of Sample Retained on No. 40 Sieve: Sample Air Dried:

Test Method A or B: A

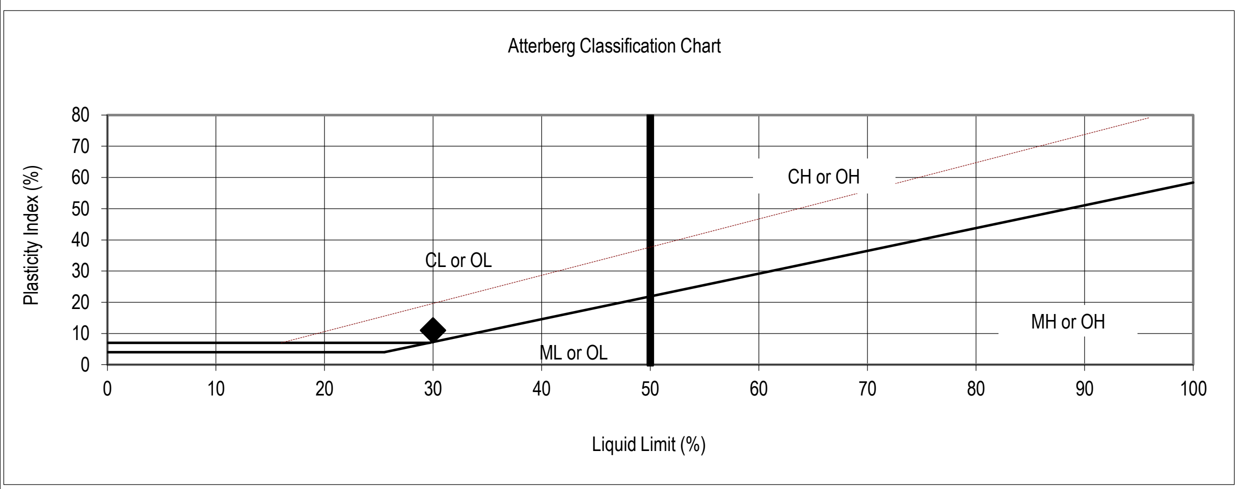
	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	C	X	E			Z	W	
Wt. Pan (gr)	38.48	38.20	36.46			37.46	37.79	
Wt. Wet Soil + Pan	46.96	49.42	48.06			46.75	45.34	
Wt. Dry Soil + Pan (	45.07	46.82	43.70			45.28	44.15	
Wt. Water (gr)	1.89	2.60	4.36			1.47	1.19	
Wt. Dry Soil (gr)	6.59	8.62	7.24			7.82	6.36	
Water Content (%)	28.7	30.2	60.2			18.8	18.7	
Number of Blows, N	35	25	15					

LIQUID LIMIT = 30 PLASTIC LIMIT = 19



Plasticity Index = 11

Group Symbol = CL



NV5

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.	71075.00.001	Project Name:	City of Clearlake Burns Valley Development	Date:	01/20/21
Sample No.	BK-1	Boring/Trench:	B21-1 Depth, (ft.): 0-3	Tested By:	LGH
Description:	(SC) CLAYEY SAND WITH GRAVEL; Yellowish Brown (10YR, 4/4)			Checked By:	DJP
Sample Location:				Lab. No.	C21-014

Moisture Content Data:			Total Material Sample Data:		
			Pan ID		
			Pan Weight		(gm)
Pan ID			Wet Soil + Pan Wt.	3,065.00	(gm)
Pan Weight		(gm)	Total Wet Weight	3,065.00	(gm)
Wet Soil + Pan		(gm)	Total Dry Weight	3,065.00	(gm)
Dry Soil + Pan		(gm)	Total Dry Wt. >#4 Sieve	1,183.20	(gm)
Water Weight	0.00	(gm)	Total Dry Wt.<#4 Sieve	1,881.80	(gm)
Dry Soil Weight	0.00	(gm)	Total Dry Wt. <#200 Sieve	614.73	(gm)
Moisture Content	0.0	(%)	Total Percent <#200 Sieve	20.06	(%)

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	3,065.00	100.0
3 Inch	3.0000	76.20		0.00	0.00	3,065.00	100.0
2 Inch	2.0000	50.80		0.00	0.00	3,065.00	100.0
1.5 Inch	1.5000	38.10		0.00	0.00	3,065.00	100.0
1.0 Inch	1.0000	25.40	26.10	26.10	26.10	3,038.90	99.1
3/4 Inch	0.7500	19.05	66.10	66.10	92.20	2,972.80	97.0
1/2 Inch	0.5000	12.70	239.00	239.00	331.20	2,733.80	89.2
3/8 Inch	0.3750	9.53	235.60	235.60	566.80	2,498.20	81.5
#4	0.1870	4.75	616.40	616.40	1,183.20	1,881.80	61.4
PAN			1,881.80	1,881.80			

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

Representative Sample Data:

			#200 Wash Data:		
Pan ID					
Pan Weight		(gm)	Portion >#200 Sieve:	222.40	(gm)
Wet Soil + Pan	330.30	(gm)	Portion <#200 Sieve:	107.90	(gm)
Wet Soil	330.30	(gm)	Percent <#200 Sieve	32.67	(%)
Dry Soil	330.30	(gm)	Total Wt. <#200 Sieve	614.73	(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	91.8	27.79	523.01	1,706.21	44.3
#20	0.033	0.850	48.50	14.68	276.32	1,982.52	35.3
#40	0.017	0.425	27.60	8.36	157.24	2,139.77	30.2
#60	0.010	0.250	16.50	5.00	94.00	2,233.77	27.1
#100	0.006	0.150	17.80	5.39	101.41	2,335.18	23.8
#200	0.003	0.075	20.20	6.12	115.08	2,450.27	20.1
PAN			Discard				

PARTICLE SIZE DISTRIBUTION

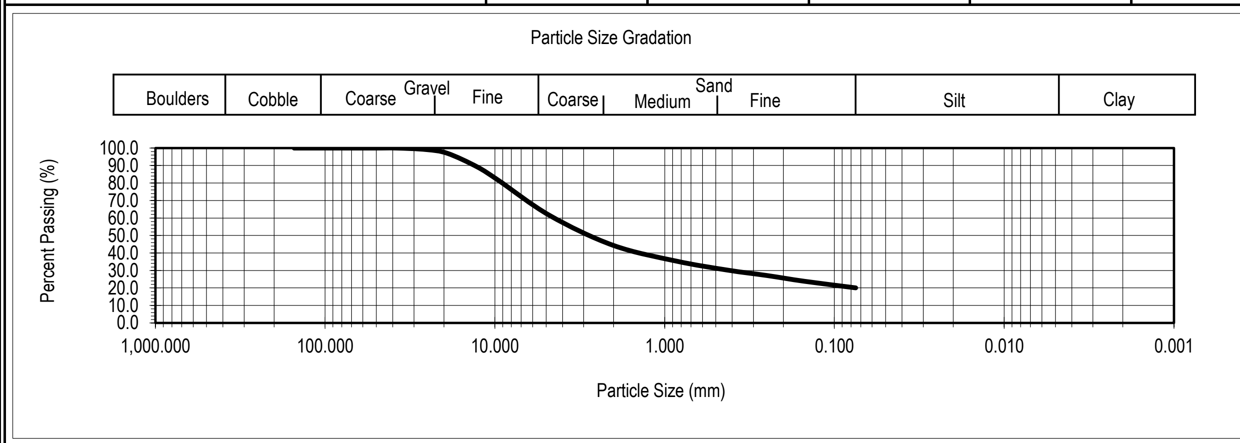
ASTM D422, C136

DSA LEA No.: 284

Project No.	**71075.00.001**	Project Name:	**City of Clearlake Burns Valley Development**	Date:	**1/20/2021**
Sample No.	**BK-1**	Boring/Trench:	**B21-1** Depth, (ft.): **0-3**	Tested By:	**LGH**
Description:	**(SC) CLAYEY SAND WITH GRAVEL; Yellowish Brown (10YR, 4/4)**			Checked By:	**DJP**
Sample Location:				Lab. No.	**C21-014**

Sieve Size	Particle Diameter		Dry Weight on Sieve			Percent
	Inches	Millimeter	Retained On Sieve	Accumulated On Sieve	Passing Sieve	Passing
(U.S. Standard)	(in.)	(mm)	(gm)	(gm)	(gm)	(%)
6 Inch	6.0000	152.4	0.00	0.0	3,065.0	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,065.0	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,065.0	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	3,065.0	**100.0**
1.0 Inch	1.0000	25.4	26.10	26.1	3,038.9	**99.1**
3/4 Inch	0.7500	19.1	66.10	92.2	2,972.8	**97.0**
1/2 Inch	0.5000	12.7	239.00	331.2	2,733.8	**89.2**
3/8 Inch	0.3750	9.5	235.60	566.8	2,498.2	**81.5**
#4	0.1870	4.7500	616.40	1,183.2	1,881.8	**61.4**
#10	0.0790	2.0066	523.01	1,706.2	1,358.8	**44.3**
#20	0.0335	0.8500	276.32	1,982.5	1,082.5	**35.3**
#40	0.0167	0.4250	157.24	2,139.8	925.2	**30.2**
#60	0.0098	0.2500	94.00	2,233.8	831.2	**27.1**
#100	0.0059	0.1500	101.41	2,335.2	729.8	**23.8**
#200	0.0030	0.0750	115.08	2,450.3	614.7	**20.1**

Hydrometer



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ATTERBERG INDICES

ASTM D4318

DSA LEA No. 284

DSA File No. N/A

DSA App No. N/A

Project No.	71075.00.001	Project Name	City of Clearlake Burns Valley Development			Date:	01/20/21
Sample No.	B2-1-1	Boring/Trench	B21-1	Depth, (ft.):	31.0	Tested By:	LGH
Description:	(CH) FAT CLAY, Dark Greenish Gray (GLEY 1, 4/1)					Checked By:	DJP
Sample Location:						Lab. No.	C21-014

Estimated % of Sample Retained on No. 40 Sieve:

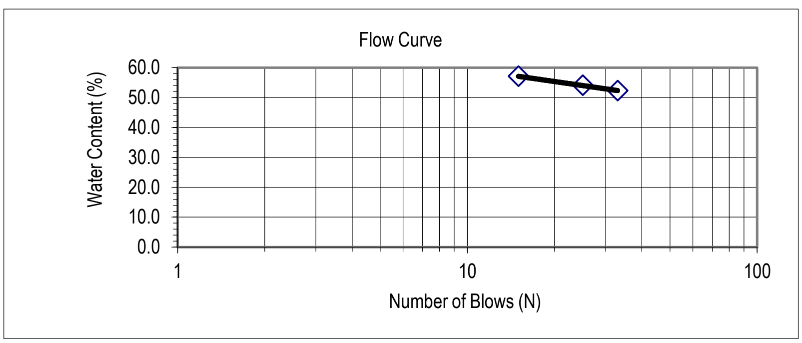
Sample Air Dried:

Test Method A or B: A

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	E	C	X			D	Z	
Wt. Pan (gr)	36.45	38.47	38.20			38.29	37.46	
Wt. Wet Soil + Pan	44.43	48.24	47.42			44.50	43.55	
Wt. Dry Soil + Pan (	41.69	44.81	44.07			43.31	42.43	
Wt. Water (gr)	2.74	3.43	3.35			1.19	1.12	
Wt. Dry Soil (gr)	5.24	6.34	5.87			5.02	4.97	
Water Content (%)	52.3	54.1	57.1			23.7	22.5	
Number of Blows, N	33	25	15					

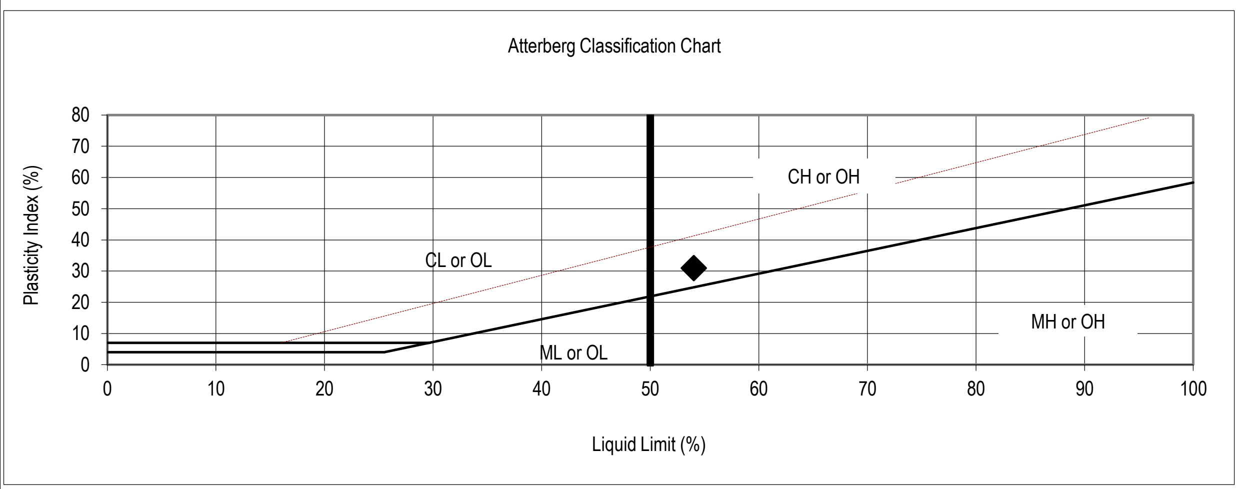
LIQUID LIMIT = 54

PLASTIC LIMIT = 23



Plasticity Index = 31

Group Symbol = CH



ATTERBERG INDICES

ASTM D4318

DSA LEA No. 284

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

Project No.	71075.00.001	Project Name	City of Clearlake Burns Valley Development			Date:	01/20/21
Sample No.	BK-2	Boring/Trench	B21-2	Depth, (ft.):	1-3	Tested By:	LGH
Description:	(CL) SANDY CLAY; Dark Yellowish Brown (10YR, 3/6)					Checked By:	0
Sample Location:						Lab. No.	C21-014

Estimated % of Sample Retained on No. 40 Sieve:

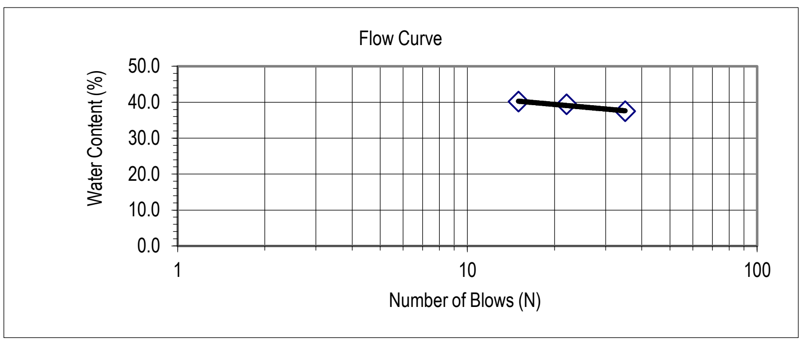
Sample Air Dried:

Test Method A or B: A

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	D	A	B			V	Y	
Wt. Pan (gr)	38.30	38.47	38.98			37.35	37.12	
Wt. Wet Soil + Pan	47.47	50.22	48.41			44.65	43.20	
Wt. Dry Soil + Pan (	44.97	46.90	45.71			43.37	42.17	
Wt. Water (gr)	2.50	3.32	2.70			1.28	1.03	
Wt. Dry Soil (gr)	6.67	8.43	6.73			6.02	5.05	
Water Content (%)	37.5	39.4	40.1			21.3	20.4	
Number of Blows, N	35	22	15					

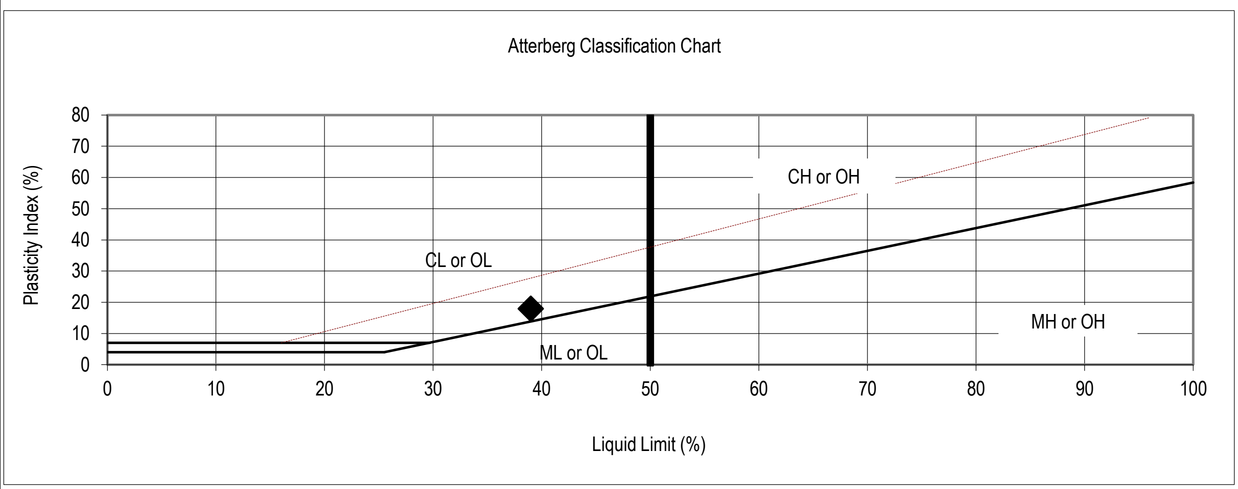
LIQUID LIMIT = 39

PLASTIC LIMIT = 21



Plasticity Index = 18

Group Symbol = CL



NV5

DSA LEA No. 284

PARTICLE SIZE DISTRIBUTION TEST WORK SHEET

ASTM D422, C136

DSA File No. N/A

DSA App No. N/A

Sieve Only Analysis Worksheet

Project No.	71075.00.001	Project Name:	City of Clearlake Burns Valley Development	Date:	01/20/21
Sample No.	BK-2	Boring/Trench:	B21-2 Depth, (ft.): 1-3	Tested By:	LGH
Description:	(CL) SANDY CLAY; Dark Yellowish Brown (10YR, 3/6)			Checked By:	0
Sample Location:				Lab. No.	C21-014

Moisture Content Data:			Total Material Sample Data:		
			Pan ID		
			Pan Weight		(gm)
Pan ID			Wet Soil + Pan Wt.	2,048.70	(gm)
Pan Weight		(gm)	Total Wet Weight	2,048.70	(gm)
Wet Soil + Pan		(gm)	Total Dry Weight	2,048.70	(gm)
Dry Soil + Pan		(gm)	Total Dry Wt. >#4 Sieve	224.20	(gm)
Water Weight	0.00	(gm)	Total Dry Wt.<#4 Sieve	1,824.50	(gm)
Dry Soil Weight	0.00	(gm)	Total Dry Wt. <#200 Sieve	1,169.67	(gm)
Moisture Content	0.0	(%)	Total Percent <#200 Sieve	57.09	(%)

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)	Percent Passing (%)
6 Inch	6.0000	152.40		0.00	0.00	2,048.70	100.0
3 Inch	3.0000	76.20		0.00	0.00	2,048.70	100.0
2 Inch	2.0000	50.80		0.00	0.00	2,048.70	100.0
1.5 Inch	1.5000	38.10		0.00	0.00	2,048.70	100.0
1.0 Inch	1.0000	25.40		0.00	0.00	2,048.70	100.0
3/4 Inch	0.7500	19.05		0.00	0.00	2,048.70	100.0
1/2 Inch	0.5000	12.70		0.00	0.00	2,048.70	100.0
3/8 Inch	0.3750	9.53	28.20	28.20	28.20	2,020.50	98.6
#4	0.1870	4.75	196.00	196.00	224.20	1,824.50	89.1
PAN			1,824.50	1,824.50			

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

Representative Sample Data:

			#200 Wash Data:		
Pan ID					
Pan Weight		(gm)	Portion >#200 Sieve:	117.40	(gm)
Wet Soil + Pan	327.10	(gm)	Portion <#200 Sieve:	209.70	(gm)
Wet Soil	327.10	(gm)	Percent <#200 Sieve	64.11	(%)
Dry Soil	327.10	(gm)	Total Wt. <#200 Sieve	1169.67	(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	23.8	7.28	132.75	356.95	82.6
#20	0.033	0.850	17.10	5.23	95.38	452.33	77.9
#40	0.017	0.425	15.50	4.74	86.46	538.79	73.7
#60	0.010	0.250	13.70	4.19	76.42	615.20	70.0
#100	0.006	0.150	19.10	5.84	106.54	721.74	64.8
#200	0.003	0.075	28.20	8.62	157.29	879.03	57.1
PAN			Discard				

PARTICLE SIZE DISTRIBUTION

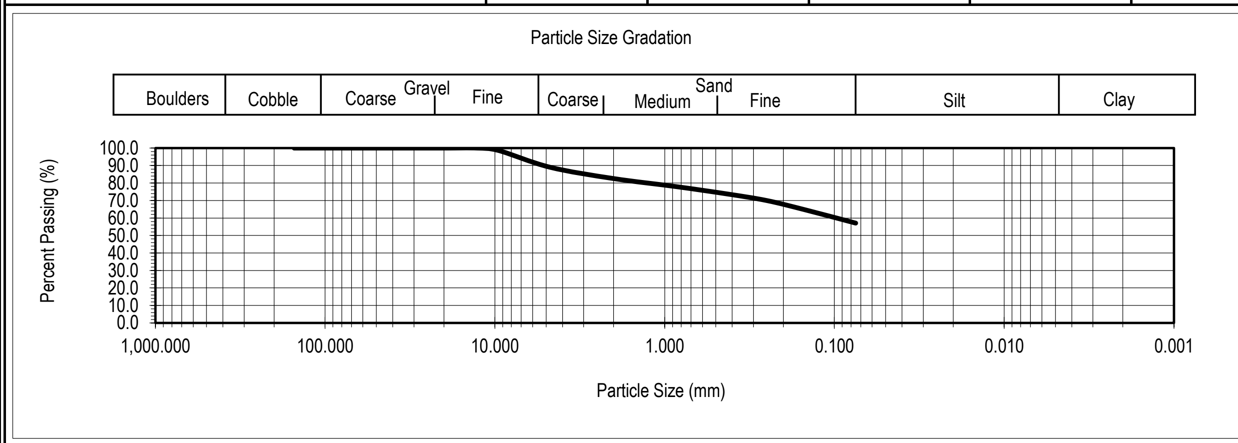
ASTM D422, C136

DSA LEA No.: 284

Project No.	**71075.00.001**	Project Name: **City of Clearlake Burns Valley Development**	Date: **1/20/2021**
Sample No.	**BK-2**	Boring/Trench: **B21-2** Depth, (ft.): **1-3**	Tested By: **LGH**
Description:	**(CL) SANDY CLAY; Dark Yellowish Brown (10YR, 3/6)**		Checked By: **0**
Sample Location:			Lab. No. **C21-014**

Sieve Size	Particle Diameter		Dry Weight on Sieve			Percent
	Inches	Millimeter	Retained On Sieve	Accumulated On Sieve	Passing Sieve	Passing
(U.S. Standard)	(in.)	(mm)	(gm)	(gm)	(gm)	(%)
6 Inch	6.0000	152.4	0.00	0.0	2,048.7	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	2,048.7	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	2,048.7	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	2,048.7	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	2,048.7	**100.0**
3/4 Inch	0.7500	19.1	0.00	0.0	2,048.7	**100.0**
1/2 Inch	0.5000	12.7	0.00	0.0	2,048.7	**100.0**
3/8 Inch	0.3750	9.5	28.20	28.2	2,020.5	**98.6**
#4	0.1870	4.7500	196.00	224.2	1,824.5	**89.1**
#10	0.0790	2.0066	132.75	357.0	1,691.7	**82.6**
#20	0.0335	0.8500	95.38	452.3	1,596.4	**77.9**
#40	0.0167	0.4250	86.46	538.8	1,509.9	**73.7**
#60	0.0098	0.2500	76.42	615.2	1,433.5	**70.0**
#100	0.0059	0.1500	106.54	721.7	1,327.0	**64.8**
#200	0.0030	0.0750	157.29	879.0	1,169.7	**57.1**

Hydrometer



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MOISTURE & DENSITY

ASTM D2216, D2937, C566

DSA LEA No. 284

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

Project No. 71075.00.001 Project Name: City of Clearlake Burns Valley Development Date: 01/20/21

Tested By: LGH
Checked By: DJP
Lab. No. C21-014

SAMPLE LOCATION DATA										
Boring/Trench No.	Units	B21-2								
Sample No.		L2-1-2								
Depth Interval	(ft.)	6.0								
Sample Description		(CL) Sandy Clay; Dark Yellowish Brown (10YR,3/6)								
USCS Symbol		CL								
SAMPLE DIMENSION AND WEIGHT DATA										
Sample Length	(in)	6.043								
Sample Diameter	(in)	2.367								
Sample Volume	(cf)	0.0154								
Wet Soil + Tube Wt.	(gr)	817.20								
Tube Wt.	(gr)	0.00								
Wet Soil Wt.	(gr)	817.20								
MOISTURE CONTENT DATA										
Tare No.		ZZ-2								
Tare Wt.	(gr)	0.00								
Wet Soil + Tare Wt.	(gr)	817.20								
Dry Soil + Tare Wt.	(gr)	703.70								
Water Wt.	(gr)	113.50								
Dry Soil Wt.	(gr)	703.70								
Moisture Content	(%)	16.1								
TEST RESULTS										
Wet Unit Wt.	(pcf)	117.1								
Moisture Content	(%)	16.1								
Dry Unit Wt.	(pcf)	100.8								
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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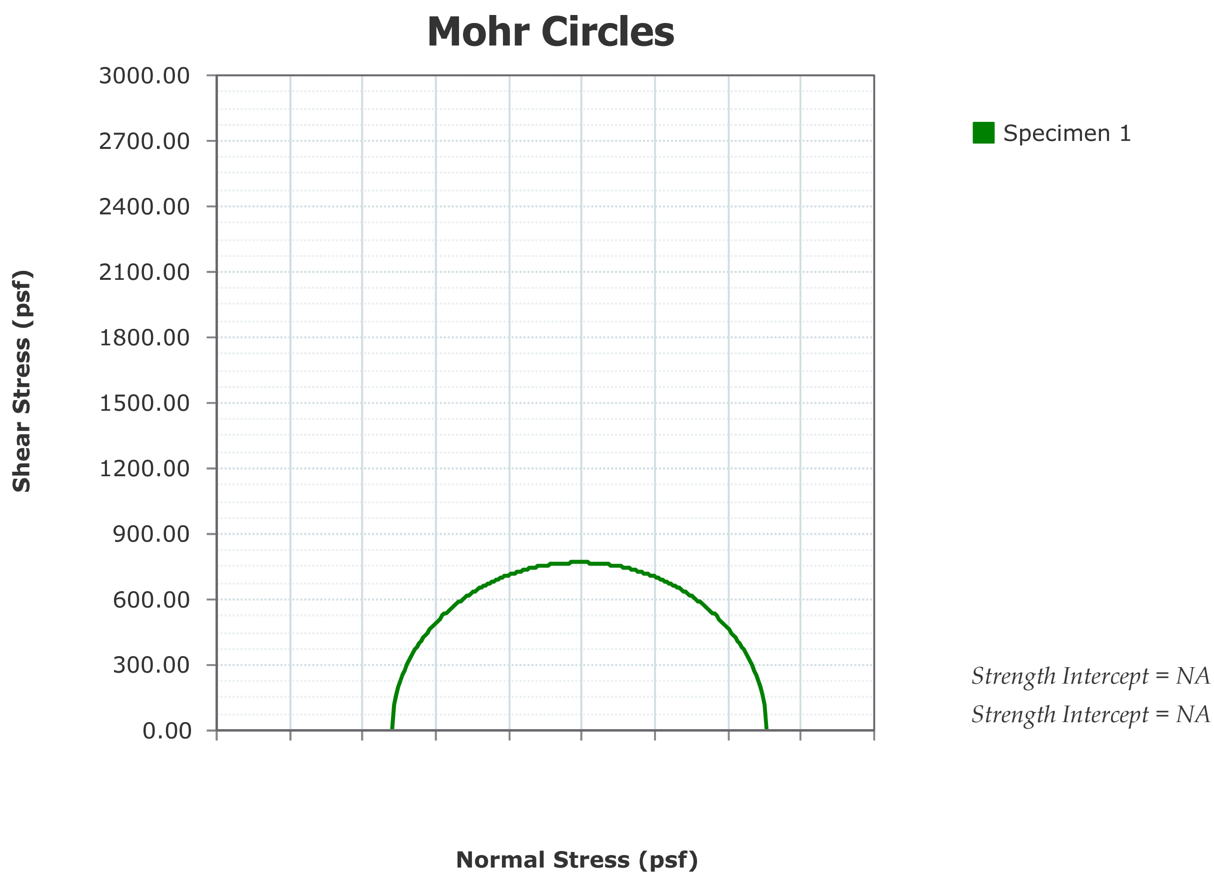
Unconsolidated Undrained Test

ASTM D2850



Project:	City of Clearlake Burns Valley Development
Project Number:	71075.00.001
Sampling Date:	
Sample Number:	L1-2-2
Sample Depth:	1.5 ft
Location:	B21-8
Client Name:	City of Clearlake
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)	5.680							
Diameter (in)	2.375							
Water Content (%)	18.5							
Wet Density (Units)	120.4							
Dry Density (pcf)	101.6							
Degree of Saturation (%)	78.0							
Void Ratio	0.628							
Height To Diameter Ratio	2.392							
Test Data	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Comp. Strength at Failure (psf)	1538.51							
σ1 at Failure (psf)	2258.51							
σ3 at Failure (psf)	720.00							
Rate of Strain (in/min)	0.085200							
Axial Strain at Failure (%)	20.44							
After Test	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Final Water Content (%)	22.3							

Project:	City of Clearlake Burns Valley Development
Project Number:	71075.00.001
Sampling Date:	
Sample Number:	L1-2-2
Sample Depth:	1.5 ft
Location:	B21-8
Client Name:	City of Clearlake
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:	D2850		
Other Associated Tests:			
Device Details:			
Test Specification:			
Test Time:	2/3/2021		
Technician:	DJP	Sampling Method:	
Specimen Code:		Specimen Lab #:	
Specimen Description:			
Specific Gravity:	2.650		
Plastic Limit:	0	Liquid Limit:	0
Height (in):	5.680	Diameter (in):	2.375
Area (in^2):	4.430	Volume (in^3):	25.16
Large Particle:			
Moisture Material:	Specimen		
Moist Weight (g):	795.4		
Test Remarks:			

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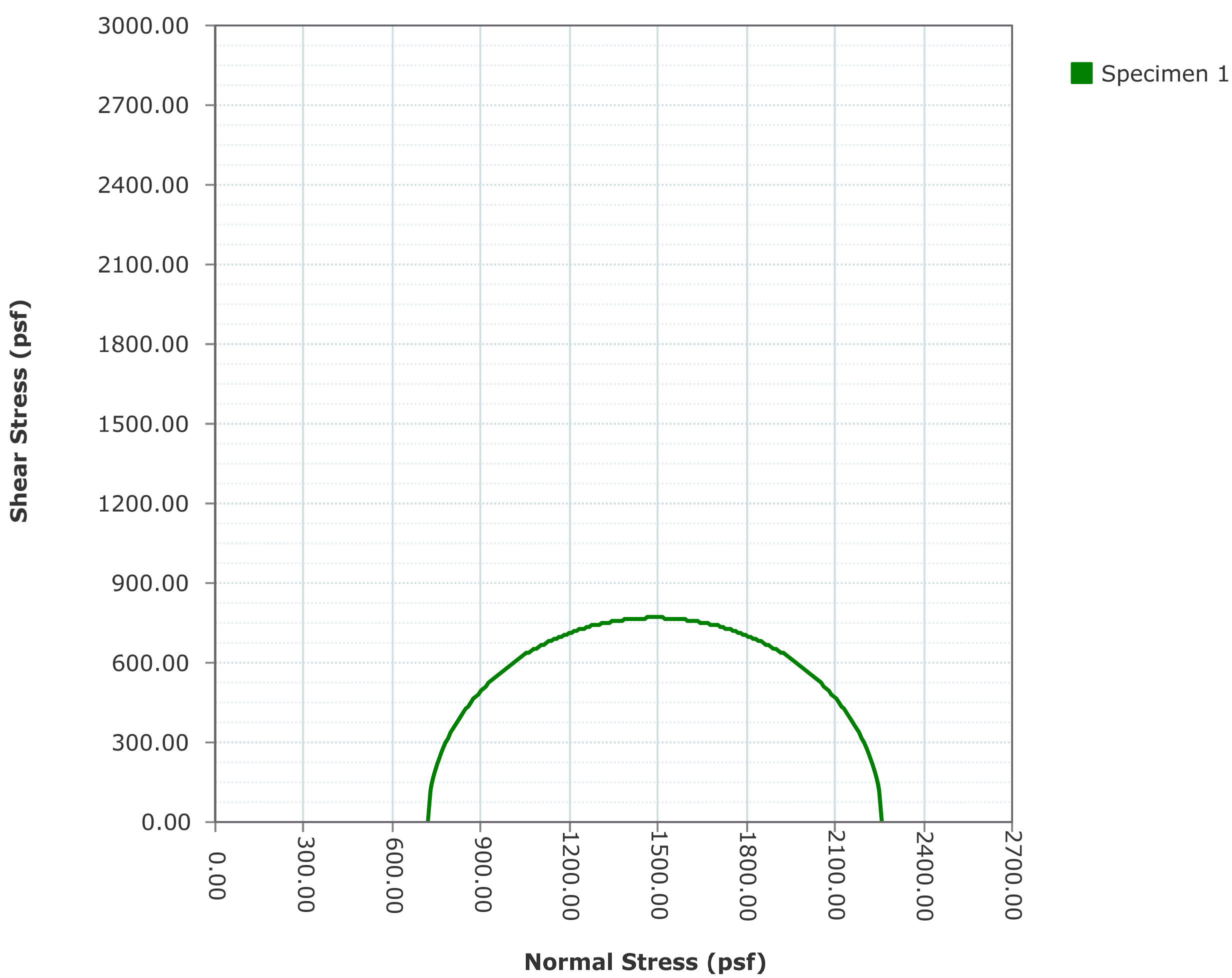
Mohr Circles (Total Stress) Graph

ASTM D2850



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

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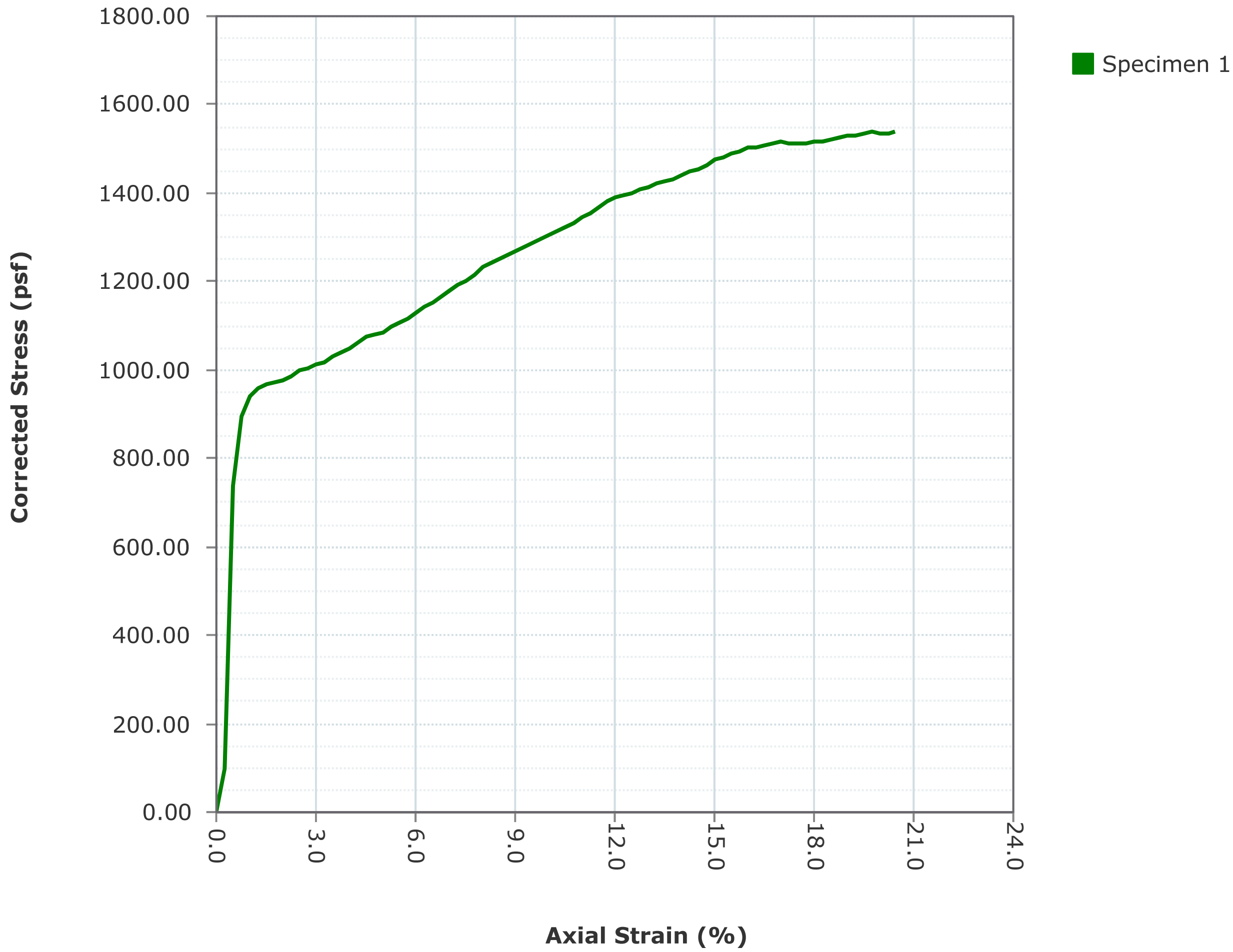
Stress-Strain Graph

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	2.1	0.0007	0.0	0.000	4.430	0.0	0.00	0.00	720.00	720.00	1.000	720.00	0.00
1	00:00:10	5.2	0.0151	3.0	0.014	4.441	0.3	97.84	97.47	817.47	720.00	1.135	768.74	48.74
2	00:00:20	24.9	0.0293	22.8	0.029	4.453	0.5	739.77	735.79	1,455.79	720.00	2.022	1,087.90	367.90
3	00:00:30	29.9	0.0439	27.7	0.043	4.464	0.8	901.70	894.48	1,614.48	720.00	2.242	1,167.24	447.24
4	00:00:40	31.4	0.0582	29.2	0.057	4.475	1.0	950.21	940.11	1,660.11	720.00	2.306	1,190.05	470.05
5	00:00:50	32.1	0.0723	29.9	0.072	4.487	1.3	972.86	960.59	1,680.59	720.00	2.334	1,200.30	480.30
6	00:01:00	32.4	0.0865	30.3	0.086	4.498	1.5	984.54	969.67	1,689.67	720.00	2.347	1,204.83	484.83
7	00:01:10	32.6	0.1006	30.4	0.100	4.509	1.8	988.69	971.30	1,691.30	720.00	2.349	1,205.65	485.65
8	00:01:20	32.8	0.1147	30.7	0.114	4.521	2.0	997.82	977.79	1,697.79	720.00	2.358	1,208.90	488.90
9	00:01:30	33.2	0.1288	31.1	0.128	4.532	2.3	1,009.64	986.88	1,706.88	720.00	2.371	1,213.44	493.44
10	00:01:40	33.6	0.1428	31.5	0.142	4.544	2.5	1,022.67	997.09	1,717.09	720.00	2.385	1,218.54	498.54
11	00:01:50	34.0	0.1569	31.8	0.156	4.555	2.8	1,034.12	1,005.68	1,725.68	720.00	2.397	1,222.84	502.84
12	00:02:00	34.2	0.1712	32.1	0.170	4.567	3.0	1,042.13	1,010.85	1,730.85	720.00	2.404	1,225.43	505.43
13	00:02:10	34.5	0.1854	32.4	0.185	4.579	3.3	1,052.95	1,018.71	1,738.71	720.00	2.415	1,229.35	509.35
14	00:02:20	35.0	0.1998	32.9	0.199	4.591	3.5	1,067.99	1,030.56	1,750.56	720.00	2.431	1,235.28	515.28
15	00:02:30	35.4	0.2141	33.2	0.213	4.603	3.8	1,079.56	1,039.01	1,759.01	720.00	2.443	1,239.50	519.50
16	00:02:40	35.7	0.2286	33.6	0.228	4.615	4.0	1,091.94	1,048.12	1,768.12	720.00	2.456	1,244.06	524.06
17	00:02:50	36.3	0.2431	34.1	0.242	4.628	4.3	1,109.38	1,062.04	1,782.04	720.00	2.475	1,251.02	531.02
18	00:03:00	36.7	0.2575	34.6	0.257	4.640	4.5	1,124.44	1,073.59	1,793.59	720.00	2.491	1,256.80	536.80
19	00:03:10	37.1	0.2716	34.9	0.271	4.652	4.8	1,135.74	1,081.57	1,801.57	720.00	2.502	1,260.78	540.78
20	00:03:20	37.3	0.2861	35.2	0.285	4.664	5.0	1,143.83	1,086.36	1,806.36	720.00	2.509	1,263.18	543.18
21	00:03:30	37.7	0.3001	35.6	0.299	4.677	5.3	1,156.95	1,095.97	1,815.97	720.00	2.522	1,267.98	547.98
22	00:03:40	38.1	0.3142	36.0	0.313	4.689	5.5	1,170.11	1,105.53	1,825.53	720.00	2.535	1,272.76	552.76
23	00:03:50	38.6	0.3283	36.5	0.328	4.701	5.8	1,186.37	1,117.94	1,837.94	720.00	2.553	1,278.97	558.97
24	00:04:00	39.2	0.3425	37.0	0.342	4.714	6.0	1,203.35	1,130.94	1,850.94	720.00	2.571	1,285.47	565.47
25	00:04:10	39.7	0.3567	37.5	0.356	4.726	6.3	1,219.44	1,143.02	1,863.02	720.00	2.588	1,291.51	571.51

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	40.1	0.3709	38.0	0.370	4.739	6.5	1,234.29	1,153.85	1,873.85	720.00	2.603	1,296.92	576.92
27	00:04:30	40.6	0.3850	38.5	0.384	4.752	6.8	1,249.92	1,165.35	1,885.35	720.00	2.619	1,302.67	582.67
28	00:04:40	41.1	0.3993	39.0	0.399	4.764	7.0	1,266.67	1,177.78	1,897.78	720.00	2.636	1,308.89	588.89
29	00:04:50	41.6	0.4137	39.5	0.413	4.778	7.3	1,283.95	1,190.58	1,910.58	720.00	2.654	1,315.29	595.29
30	00:05:00	42.2	0.4281	40.0	0.427	4.791	7.5	1,300.99	1,203.09	1,923.09	720.00	2.671	1,321.54	601.54
31	00:05:10	42.7	0.4424	40.6	0.442	4.804	7.8	1,319.49	1,216.87	1,936.87	720.00	2.690	1,328.43	608.43
32	00:05:20	43.4	0.4566	41.2	0.456	4.817	8.0	1,340.32	1,232.72	1,952.72	720.00	2.712	1,336.36	616.36
33	00:05:30	43.8	0.4707	41.6	0.470	4.830	8.3	1,353.01	1,241.05	1,961.05	720.00	2.724	1,340.53	620.53
34	00:05:40	44.2	0.4849	42.0	0.484	4.843	8.5	1,366.64	1,250.14	1,970.14	720.00	2.736	1,345.07	625.07
35	00:05:50	44.6	0.4989	42.5	0.498	4.856	8.8	1,380.45	1,259.36	1,979.36	720.00	2.749	1,349.68	629.68
36	00:06:00	45.0	0.5130	42.9	0.512	4.869	9.0	1,394.23	1,268.48	1,988.48	720.00	2.762	1,354.24	634.24
37	00:06:10	45.5	0.5270	43.3	0.526	4.883	9.3	1,408.09	1,277.61	1,997.61	720.00	2.774	1,358.80	638.80
38	00:06:20	45.9	0.5411	43.7	0.540	4.896	9.5	1,421.24	1,286.01	2,006.01	720.00	2.786	1,363.00	643.00
39	00:06:30	46.3	0.5552	44.1	0.554	4.909	9.8	1,434.51	1,294.48	2,014.48	720.00	2.798	1,367.24	647.24
40	00:06:40	46.7	0.5693	44.6	0.569	4.923	10.0	1,449.54	1,304.42	2,024.42	720.00	2.812	1,372.21	652.21
41	00:06:50	47.2	0.5835	45.1	0.583	4.937	10.3	1,465.35	1,315.00	2,035.00	720.00	2.826	1,377.50	657.50
42	00:07:00	47.6	0.5977	45.5	0.597	4.950	10.5	1,478.52	1,323.11	2,043.11	720.00	2.838	1,381.56	661.56
43	00:07:10	48.1	0.6122	45.9	0.611	4.965	10.8	1,493.14	1,332.40	2,052.40	720.00	2.851	1,386.20	666.20
44	00:07:20	48.6	0.6265	46.5	0.626	4.979	11.0	1,510.09	1,343.72	2,063.72	720.00	2.866	1,391.86	671.86
45	00:07:30	49.1	0.6410	47.0	0.640	4.993	11.3	1,527.66	1,355.46	2,075.46	720.00	2.883	1,397.73	677.73
46	00:07:40	49.8	0.6551	47.6	0.654	5.007	11.5	1,547.62	1,369.32	2,089.32	720.00	2.902	1,404.66	684.66
47	00:07:50	50.3	0.6693	48.1	0.669	5.021	11.8	1,564.00	1,379.91	2,099.91	720.00	2.917	1,409.95	689.95
48	00:08:00	50.7	0.6833	48.6	0.683	5.035	12.0	1,578.67	1,388.95	2,108.95	720.00	2.929	1,414.48	694.48
49	00:08:10	51.1	0.6974	48.9	0.697	5.050	12.3	1,589.90	1,394.88	2,114.88	720.00	2.937	1,417.44	697.44
50	00:08:20	51.4	0.7114	49.3	0.711	5.064	12.5	1,601.17	1,400.84	2,120.84	720.00	2.946	1,420.42	700.42
51	00:08:30	51.8	0.7256	49.6	0.725	5.078	12.8	1,613.14	1,407.27	2,127.27	720.00	2.955	1,423.64	703.64

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)
52	00:08:40	52.1	0.7396	50.0	0.739	5.093	13.0	1,625.06	1,413.65	2,133.65	720.00	2.963	1,426.82	706.82
53	00:08:50	52.5	0.7536	50.4	0.753	5.107	13.3	1,637.06	1,420.06	2,140.06	720.00	2.972	1,430.03	710.03
54	00:09:00	52.8	0.7676	50.7	0.767	5.122	13.5	1,647.90	1,425.40	2,145.40	720.00	2.980	1,432.70	712.70
55	00:09:10	53.2	0.7817	51.1	0.781	5.136	13.8	1,660.51	1,432.18	2,152.18	720.00	2.989	1,436.09	716.09
56	00:09:20	53.7	0.7959	51.5	0.795	5.151	14.0	1,674.91	1,440.42	2,160.42	720.00	3.001	1,440.21	720.21
57	00:09:30	54.1	0.8103	51.9	0.810	5.167	14.3	1,687.55	1,447.00	2,167.00	720.00	3.010	1,443.50	723.50
58	00:09:40	54.5	0.8249	52.3	0.824	5.182	14.5	1,701.37	1,454.47	2,174.48	720.00	3.020	1,447.24	727.24
59	00:09:50	54.9	0.8393	52.8	0.839	5.197	14.8	1,715.53	1,462.25	2,182.25	720.00	3.031	1,451.13	731.13
60	00:10:00	55.5	0.8536	53.4	0.853	5.213	15.0	1,734.54	1,474.08	2,194.08	720.00	3.047	1,457.04	737.04
61	00:10:10	56.0	0.8677	53.8	0.867	5.228	15.3	1,749.72	1,482.63	2,202.63	720.00	3.059	1,461.31	741.31
62	00:10:20	56.3	0.8818	54.2	0.881	5.243	15.5	1,760.92	1,487.77	2,207.77	720.00	3.066	1,463.89	743.89
63	00:10:30	56.7	0.8956	54.6	0.895	5.259	15.8	1,773.36	1,493.95	2,213.95	720.00	3.075	1,466.98	746.98
64	00:10:40	57.1	0.9096	55.0	0.909	5.274	16.0	1,787.19	1,501.20	2,221.20	720.00	3.085	1,470.60	750.60
65	00:10:50	57.4	0.9236	55.3	0.923	5.290	16.2	1,797.26	1,505.23	2,225.23	720.00	3.091	1,472.61	752.61
66	00:11:00	57.7	0.9378	55.6	0.937	5.305	16.5	1,806.11	1,508.13	2,228.13	720.00	3.095	1,474.07	754.07
67	00:11:10	58.0	0.9518	55.8	0.951	5.321	16.7	1,813.88	1,510.15	2,230.15	720.00	3.097	1,475.08	755.08
68	00:11:20	58.3	0.9660	56.1	0.965	5.337	17.0	1,824.36	1,514.30	2,234.30	720.00	3.103	1,477.15	757.15
69	00:11:30	58.3	0.9802	56.1	0.979	5.353	17.2	1,824.55	1,509.92	2,229.92	720.00	3.097	1,474.96	754.96
70	00:11:40	58.5	0.9945	56.3	0.994	5.370	17.5	1,830.85	1,510.50	2,230.50	720.00	3.098	1,475.25	755.25
71	00:11:50	58.7	1.0088	56.6	1.008	5.386	17.7	1,839.20	1,512.77	2,232.77	720.00	3.101	1,476.39	756.39
72	00:12:00	59.0	1.0232	56.9	1.023	5.403	18.0	1,849.09	1,516.20	2,236.20	720.00	3.106	1,478.10	758.10
73	00:12:10	59.2	1.0376	57.1	1.037	5.419	18.3	1,855.67	1,516.91	2,236.91	720.00	3.107	1,478.46	758.46
74	00:12:20	59.5	1.0519	57.4	1.051	5.436	18.5	1,865.59	1,520.31	2,240.31	720.00	3.112	1,480.16	760.16
75	00:12:30	59.9	1.0663	57.8	1.066	5.453	18.8	1,877.79	1,525.51	2,245.51	720.00	3.119	1,482.76	762.76
76	00:12:40	60.3	1.0804	58.1	1.080	5.470	19.0	1,888.87	1,529.81	2,249.81	720.00	3.125	1,484.91	764.90
77	00:12:50	60.5	1.0943	58.4	1.094	5.487	19.3	1,897.28	1,531.97	2,251.97	720.00	3.128	1,485.99	765.99

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 (in^2)	Axial Strain (%)	Stress (psf)	Corrected Compressive Stress (psf)	σ1 (psf)	σ3 (psf)	σ1 — σ3	p (psf)	q (psf)
78	00:13:00	60.8	1.1081	58.6	1.107	5.503	19.5	1,906.28	1,534.61	2,254.61	720.00	3.131	1,487.31	767.31
79	00:13:10	61.1	1.1223	58.9	1.122	5.520	19.7	1,915.23	1,537.04	2,257.04	720.00	3.135	1,488.52	768.52
80	00:13:20	61.2	1.1364	59.1	1.136	5.537	20.0	1,920.64	1,536.60	2,256.60	720.00	3.134	1,488.30	768.30
81	00:13:30	61.4	1.1505	59.3	1.150	5.555	20.2	1,926.08	1,536.16	2,256.16	720.00	3.134	1,488.08	768.08
82	00:13:37	61.6	1.1615	59.5	1.161	5.568	20.4	1,933.71	1,538.51	2,258.51	720.00	3.137	1,489.25	769.25

APPENDIX D:

Liquefaction Analysis Results

NV5

Appendix D: SPT-Based Liquefaction Triggering Analysis for a Single Boring

Project Name: Proposed Burns Valley Development

Project No.: 71075.00

Boring No.: B21-1

Input parameters:

Peak ground accel (g) =	0.628	PGA_M
Earthquake magnitude, M =	9	
Water table depth (m) =	3.048	
Average γ above water table (kN/m³) =	17.6	*multiply unit weight in pcf by 0.16026 to obtain metric units
Average γ below water table (kN/m³) =	16.0	
Borehole Diameter (mm) =	203.2	
Requires correction for sampler liners (YES/NO)	YES	

Rod lengths assumed equal to the depth plus 1.5m (for the above ground extension).

Liquefaction Potential and Triggering

SPT Sample Number	Depth (m)	Measured N	Soil Type (USCS)	Flag "Clay" "Unsaturated" "Unreliable"	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	α_{vc} (kPa)	α_{vc}' (kPa)	C_N	$(N_1)_{60}$	Δn for fines content	$(N_1)_{60\text{-}cs}$	Stress Reduct. Coefficient r_d	CSR	MSF for sand	K_σ for sand	crr FOR m=7.5 & α_{vc} '=1atm	CRR	Factor of Safety
1	1.524	23	SC	unsaturated	20	75	1.25	1.15	0.8	1.3	34.4	27	27	1.42	48.7	4.5	53.20	1.00	0.410	0.67	1.10	2.000	n.a.	n.a.
2	3.048	34	CL	unsaturated	60	75	1.25	1.15	0.85	1.3	54.0	54	54	1.18	63.8	5.6	69.37	1.00	0.410	0.67	1.10	2.000	n.a.	n.a.
3	4.572	11	CL	clay	60	75	1.25	1.15	0.95	1.3	19.5	78	63	1.13	n.a.	n.a.	n.a.	1.01	0.507	0.67	1.10	n.a.	n.a.	n.a.
4	6.096	46	CL	clay	90	75	1.25	1.15	0.95	1.3	81.7	103	73	1.09	n.a.	n.a.	n.a.	1.01	0.579	0.67	1.10	n.a.	n.a.	n.a.
5	7.620	13	GM		20	75	1.25	1.15	0.95	1.2374	22.0	127	82	1.08	23.7	4.5	28.22	1.01	0.635	0.67	1.04	0.393	0.273	0.43
6	9.144	5	CH	clay	85	75	1.25	1.15	1	1.3	9.3	151	92	1.03	n.a.	n.a.	n.a.	1.01	0.678	0.67	1.03	n.a.	n.a.	n.a.
7	10.668	35	CH	clay	85	75	1.25	1.15	1	1.3	65.4	176	101	1.00	n.a.	n.a.	n.a.	1.00	0.713	0.67	1.00	n.a.	n.a.	n.a.
8	12.192	18	CH	clay	85	75	1.25	1.15	1	1.3	33.6	200	111	0.98	n.a.	n.a.	n.a.	1.00	0.741	0.67	0.97	n.a.	n.a.	n.a.
9	13.716	16	CH	clay	85	75	1.25	1.15	1	1.3	29.9	225	120	0.96	n.a.	n.a.	n.a.	1.00	0.764	0.67	0.95	n.a.	n.a.	n.a.
10	15.240	22	CH	clay	85	75	1.25	1.15	1	1.3	41.1	249	130	0.94	n.a.	n.a.	n.a.	1.00	0.782	0.67	0.93	n.a.	n.a.	n.a.

Seismically Induced Settlement

SPT Sample Number	Depth (m)	Measured N	Soil Type (USCS)	Limiting shear strain γ_{lim}	Para-meter $F\alpha$	Maximum shear strain γ_{max}	Δh_i (m)	ΔLDI_i (m)	Vertical reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	1.524	23	SC	0.000	-1.851	0.000	1.524	0.000	0.000	0.000	0.000
2	3.048	34	CL	0.000	-3.239	0.000	1.524	0.000	0.000	0.000	0.000
3	4.572	11	CL	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
4	6.096	46	CL	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
5	7.620	13	GM	0.059	0.029	0.059	1.524	0.090	0.012	0.019	0.749
6	9.144	5	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
7	10.668	35	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
8	12.192	18	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
9	13.716	16	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
10	15.240	22	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
							LDI=	0.090	Total S=	0.019	0.749

Field Data for Conversion

Sample Number	*Sample Depth (ft)*	*Strata Δh*	*Depth to GW (ft)*	*Historic High Depth to GW (ft)*	*Ave. Unit Wt Above GW (pcf)*	*Ave. Unit Wt Below GW (pcf)*	*Borehole Dia. (in)*
1	*5*	1.524	*19*	*10*	*110*	*100*	*8*
2	*10*	1.524					
3	*15*	1.524					
4	*20*	1.524					
5	*25*	1.524					
6	*30*	1.524					
7	*35*	1.524					
8	*40*	1.524					
9	*45*	1.524					
10	*50*	1.524					

Appendix D: SPT-Based Liquefaction Triggering Analysis for a Single Boring

Project Name: Proposed Burns Valley Development
Project No.: 71075.00
Boring No.: B21-2

Input parameters:

Peak ground accel (g) = 0.628 PGA_M
Earthquake magnitude, M = 9
Water table depth (m) = 3.048
Average ϒ above water table (kN/m³) = 17.6 *multiply unit weight in pcf by 0.16026 to obtain metric units
Average ϒ below water table (kN/m³) = 16.0
Borehole Diameter (mm) = 203.2
Requires correction for sampler liners (YES/NO) Yes
Rod lengths assumed equal to the depth plus 1.5m (for the above ground extension).

Liquefaction Potential and Triggering

SPT Sample Number	Depth	Measured N	Soil Type	Flag "Clay" "Unsaturated" "Unreliable"	Fines Content	Energy Ratio, ER	C_E	C_B	C_R	C_S	N_{60}	σ_{vc}	σ_{vc}'	C_N	$(N_1)_{60}$	Δn for fines content	$(N_1)_{60-cs}$	Stress Reduct. Coefficient r_d	CSR	MSF for sand	K_σ for sand	crr FOR m=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety
	(m)		(USCS)		(%)	(%)						(kPa)	(kPa)											
1	1.524	28	SC	unsaturated	60	75	1.25	1.15	0.8	1.3	41.9	27	27	1.42	59.3	5.6	64.91	1.00	0.410	0.67	1.10	2.000	n.a.	n.a.
2	3.048	29	CL	unsaturated	60	75	1.25	1.15	0.85	1.3	46.1	54	54	1.18	54.4	5.6	59.99	1.00	0.410	0.67	1.10	2.000	n.a.	n.a.
3	4.572	18	CL	clay	60	75	1.25	1.15	0.95	1.3	32.0	78	63	1.13	n.a.	n.a.	n.a.	1.01	0.507	0.67	1.10	n.a.	n.a.	n.a.
4	6.096	10	CL	clay	60	75	1.25	1.15	0.95	1.3	17.8	103	73	1.09	n.a.	n.a.	n.a.	1.01	0.579	0.67	1.10	n.a.	n.a.	n.a.
5	7.620	18	CL	clay	60	75	1.25	1.15	0.95	1.3	32.0	127	82	1.06	n.a.	n.a.	n.a.	1.01	0.635	0.67	1.06	n.a.	n.a.	n.a.
6	9.144	4	CL	clay	85	75	1.25	1.15	1	1.3	7.5	151	92	1.03	n.a.	n.a.	n.a.	1.01	0.678	0.67	1.03	n.a.	n.a.	n.a.
7	10.668	19	SM		45	75	1.25	1.15	1	1.3	35.5	176	101	1.00	35.5	5.6	41.10	1.00	0.713	0.67	1.00	2.000	1.338	1.88
8	12.192	14	CH	clay	85	75	1.25	1.15	1	1.3	26.2	200	111	0.98	n.a.	n.a.	n.a.	1.00	0.741	0.67	0.97	n.a.	n.a.	n.a.
9	13.716	12	CH	clay	85	75	1.25	1.15	1	1.3	22.4	225	120	0.96	n.a.	n.a.	n.a.	1.00	0.764	0.67	0.95	n.a.	n.a.	n.a.
10	15.240	16	CH	clay	85	75	1.25	1.15	1	1.3	29.9	249	130	0.94	n.a.	n.a.	n.a.	1.00	0.782	0.67	0.93	n.a.	n.a.	n.a.

Seismically Induced Settlement

SPT Sample Number	Depth	Measured N	Soil Type	Limiting shear strain γ_{lim}	Para-meter $F\alpha$	Maximum shear strain γ_{max}	Δh_i	ΔLDI_i	Vertical reconsol. Strain ε_v	ΔS_i	ΔS_i
	(m)		(USCS)				(m)	(m)		(m)	(in)
1	1.524	28	SC	0.000	-2.847	0.000	1.524	0.000	0.000	0.000	0.000
2	3.048	29	CL	0.000	-2.422	0.000	1.524	0.000	0.000	0.000	0.000
3	4.572	18	CL	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
4	6.096	10	CL	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
5	7.620	18	CL	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
6	9.144	4	CL	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
7	10.668	19	SM	0.007	-0.888	0.000	1.524	0.000	0.000	0.000	0.000
8	12.192	14	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
9	13.716	12	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
10	15.240	16	CH	0.000	0.000	0.000	1.524	0.000	0.000	0.000	0.000
							LDI=	0.000	Total S=	0.000	0.000

Field Data for Conversion

Sample Number	*Sample Depth (ft)*	*Strata Δh*	*Depth to GW (ft)*	*Historic High Depth to GW (ft)*	*Ave. Unit Wt Above GW (pcf)*	*Ave. Unit Wt Below GW (pcf)*	*Borehole Dia. (in)*
1	*5*	1.524	*19*	*10*	*110*	*100*	*8*
2	*10*	1.524					
3	*15*	1.524					
4	*20*	1.524					
5	*25*	1.524					
6	*30*	1.524					
7	*35*	1.524					
8	*40*	1.524					
9	*45*	1.524					
10	*50*	1.524					

APPENDIX E:

Seismic Design Parameters

City of Clearlake - Burns Valley Development

Latitude, Longitude: 38.9638, -122.6349

Date	2/19/2021, 12:14:23 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.541	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.2	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.523	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.628	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.567	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.672	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.541	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.586	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.523	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.937	Mapped value of the risk coefficient at short periods
C_{R1}	0.923	Mapped value of the risk coefficient at a period of 1 s