

A P P E N D I X E

G E O T E C H N I C A L E N G I N E E R I N G
I N V E S T I G A T I O N R E P O R T



GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
THE MOSAIC PROJECT
17015 CULL CANYON ROAD
CASTRO VALLEY, CALIFORNIA
September 16, 2019

Prepared For:

THE MOSAIC PROJECT
478 Santa Clara Avenue, Suite 200
Oakland, California 94610

Mr. Brian Lowe, Chief Operating Officer



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125619-0070852.00.001



September 16, 2019
Project No. 70852.00.001

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Via PDF: Brian@mosaicproject.org

**Reference: Geotechnical Engineering Investigation Report
The Mosaic Project
17015 Cull Canyon Road
Castro Valley, Alameda County, California**

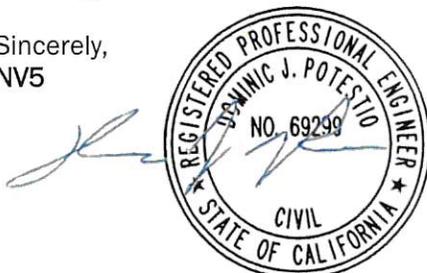
Dear Mr. Lowe,

NV5 conducted a geotechnical engineering investigation for the proposed Mosaic Project educational development located at 17015 Cull Canyon Road in Castro Valley, California. NV5's geotechnical engineering investigation of the site was performed consistent with the scope of services presented in the February 22, 2019 proposal (PC19.043).

The findings, conclusions and recommendations presented in this report are based on the following relevant information collected and evaluated by NV5: literature review, surface observations, subsurface exploration, laboratory test results, and experience with similar projects, sites and conditions in the area. There were no seismic hazards identified on the site or in the immediate area that require design mitigation. There were geotechnical engineering hazards consisting of loose fill materials containing concrete and asphalt concrete (AC) rubble within the subgrade soils of the site. However, 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 engineering geology 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



Dominic J. Potestio, PE 69299
Senior Engineer



Shane D. Cummings, CEG 2492
Associate Engineering Geologist

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B	Historical Aerial Photographs
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ACRONYMS

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
CQA	Construction Quality Assurance
DTSC	Department of Toxic Substances Control
EFP	equivalent fluid pressure
FS	factor of safety
ft/sec	feet per second
GBA	Geoprofessional Business Association
H:V	horizontal to vertical
IBC	International Building Code
MCE	maximum considered earthquake
msl	mean sea level
mybp	million years before present
OSHA	Occupational Safety and Hazards Administration
oz/sy	ounce per square yard
PCA	Portland Cement Association
PGA _M	peak ground acceleration
PI	plasticity index
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinylchloride
S-wave	shear-wave
SEAOC	Structural Engineers Association of California
sf	square feet
SRMS	Seismic Refraction Microtremor Survey
SSD	saturated surface dry
USCS	Unified Soils Classification System

1.0 INTRODUCTION

NV5 performed a geotechnical engineering investigation and prepared a geotechnical engineering investigation report for the proposed Mosaic Project educational development at 17015 Cull Canyon Road in Castro Valley, California, consistent with the scope of services presented in NV5's *Proposal for Geotechnical Engineering Services (PC19.043)*, dated February 22, 2019. 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.

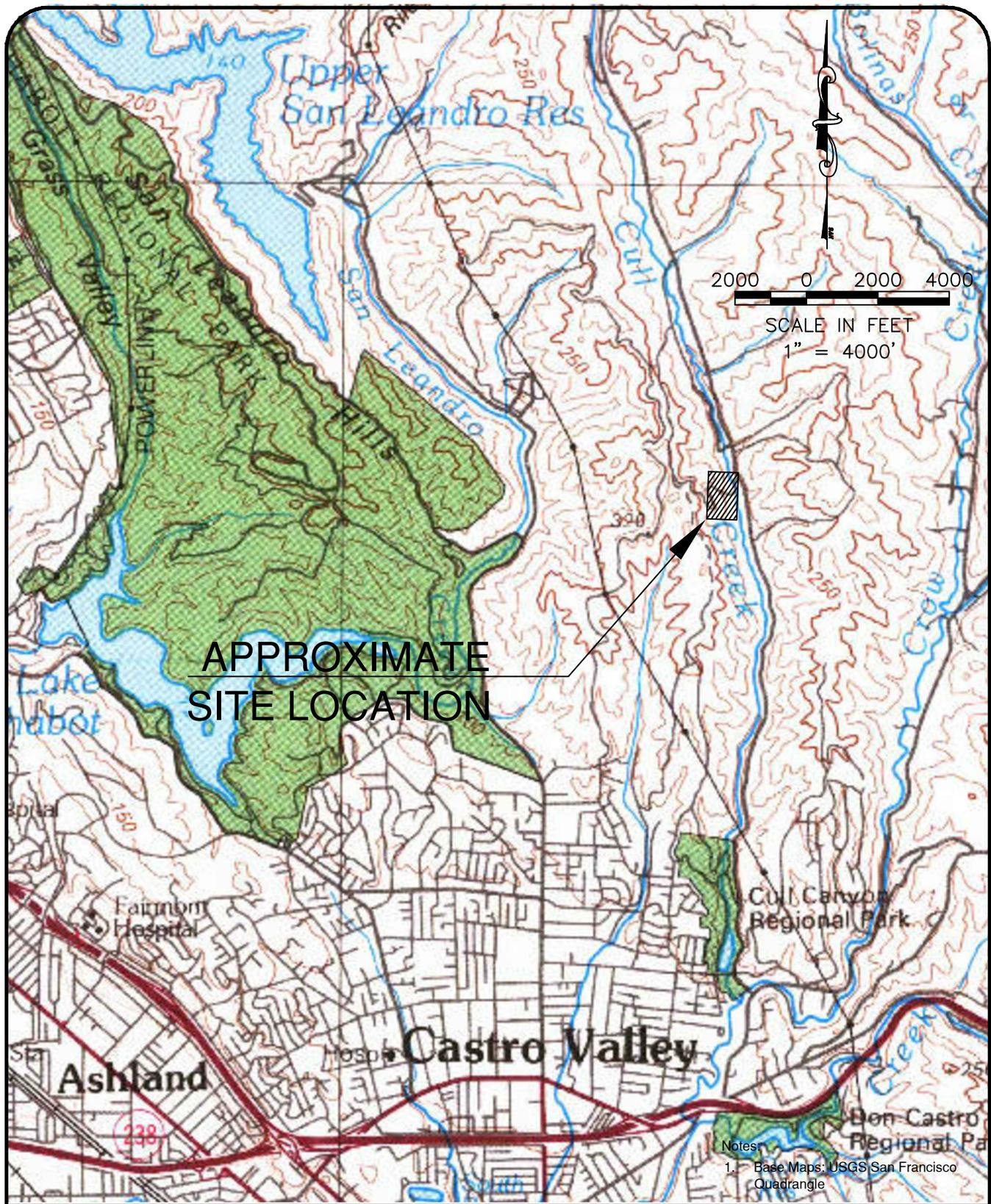
1.1 SCOPE-OF-SERVICES

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

- **Task 1 Site Investigation and Laboratory Testing:** 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, collected representative soil samples, conducted seismic refraction survey, 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 Mosaic Project educational development is located at 17015 Cull Canyon Road, north of Crow Canyon Road, in Castro Valley, California. The site is centered at about latitude 37.7418 North and longitude -122.0551 West. The property elevation ranges from approximately 72 feet above mean sea level (msl) along Cull Creek to approximately 150 feet above msl in the southwestern portion of the site, based on review of the topographical information presented on the *Topographic Survey* dated January 28, 2019, and prepared by Greenwood & Moore, Inc. Figure 1 shows the site location and vicinity.



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SITE LOCATION MAP
 THE MOSAIC PROJECT
 CASTRO VALLEY, ALAMEDA COUNTY, CALIFORNIA

PROJ NO.: 70852.00
 DATE: AUGUST, 2019
 FIGURE NO.: **1**



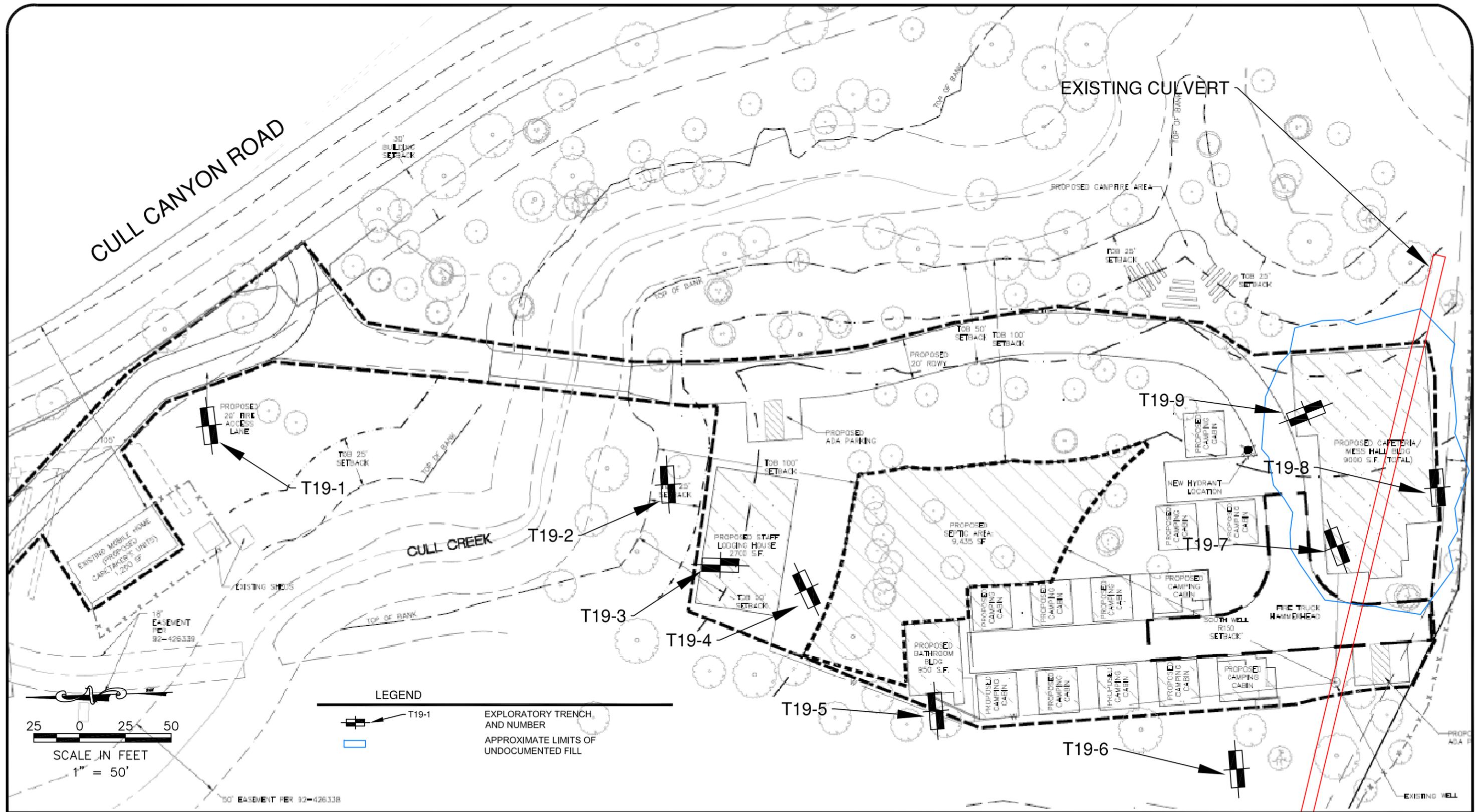
At the time of NV5's site investigation on June 6, 2019, the following site conditions were observed:

- The northeastern portion of the site supported an agricultural barn, a modular single-family residence, 2 outbuildings, and 2 metal storage containers. Numerous stockpiles of organics, primarily consisting of tree clippings, were observed to the south of the residence.
- Cull Creek was observed meandering through the eastern portion of the site. A concrete-decked bridge provided access to the portion of the site west of the creek. The creek had water in it at the time of NV5's site investigation.
- The western portion of the site supported a small wood-framed well house, a metal free-standing shade structure covering an outdoor kitchen area, and one tall, single-story masonry recreational building. The recreational building was partially built into the existing slope on the west side. The slope was retained by a masonry retaining wall. Access to the recreational building was provided by a concrete driveway extending from the bridge to the building.
- The ground surface of the site supported slight to moderate concentration of volunteer weeds and grasses. Mature trees were observed throughout the project site.

1.3 PROPOSED IMPROVEMENTS

Based on the project information provided by representatives of NorthStar, NV5 understands the proposed Mosaic project will include construction of an Outdoor Program overnight camp for 4th and 5th grade students. NV5's review of *Proposed Site Layout* dated July 2019, and prepared by

Northstar, indicates site development will include construction of 12 new dormitory cabins for students, a new restroom/shower building approximately 950 square feet (sf) in size, a new cafeteria/mess hall building approximately 9,000 sf in size, and a new staff lodging house approximately 2,700 sf in size. NV5 anticipates the proposed structures will be constructed with wood or steel framed walls and supported on continuous spread and isolated foundations with an interior slab-on-grade, concrete floor. Associated development is indicated to include construction of retaining walls, underground utilities, a campfire area, asphalt concrete/concrete pavements, and exterior concrete slab-on-grade flatwork. Earthwork grading may include general site preparation and minor to moderate cuts and fills required to balance the site to meet the proposed building grades. Figure 2 shows the proposed site development.



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SITE PLAN AND EXPLORATORY TRENCH LOCATIONS

THE MOSAIC PROJECT
17015 CULL CANYON ROAD
CASTRO VALLEY, ALAMEDA COUNTY, CALIFORNIA

DRAWN BY	SC
CHECKED BY	SDC
PROJECT NO.	70852.00
DATE	AUGUST, 2019

FIGURE NO.

2

1.4 INVESTIGATION PURPOSE

The purpose of NV5's investigation is to obtain sufficient on-site information about the soil, rock and groundwater conditions at the site to allow NV5 to prepare geotechnical engineering design recommendations for construction of the proposed earthwork and structural improvements described in the preceding. NV5 did not evaluate the site for the presence of hazardous waste, mold, asbestos and radon gas. Therefore, the presence and removal of these materials are not discussed in this report.

2.0 SITE INVESTIGATION

NV5 performed a site investigation to characterize the existing soil, rock and groundwater site conditions. The site investigation included a literature review of published and unpublished geologic documents and maps, review of historical aerial photographs, a surface reconnaissance investigation, and a subsurface exploratory investigation using seismic refraction survey equipment and a track-mounted excavator to excavate exploratory trenches. 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

NV5 reviewed the following design improvement plans:

- Existing Site Layout, prepared by NorthStar, 111 Mission Ranch Blvd., Suite 100, Chico, CA 95926, dated July 2019.
- Proposed Site Layout, prepared by NorthStar, 111 Mission Ranch Blvd., Suite 100, Chico, CA 95926, dated July 2019.
- Topographic Survey, prepared by Greenwood & Moore, Inc., 3111 Castro Valley Road, Suite 200, Castro Valley, CA 94546, dated January 28, 2019.

2.1.2 Previous Site Investigation Reports

NV5 was not provided previous geotechnical reports for review that may be associated with the existing site.

2.1.3 Historical Aerial Photographs

NV5 reviewed historical aerial photographs of the project site and its vicinity from 1946 through 2016. NV5 reviewed the photographs for evidence of past land use on and around the subject property and visible signs of previous landslide scarps. NV5's review of the historical aerial photographs is summarized below. Copies of the aerial photographs are included in Appendix B.

1946 through 1963: The subject property appears to support at least four to five structures in the western portion of the site, near the area that currently supports the large, masonry recreational building. An unpaved access road is visible extending from Cull Canyon Road across Cull Creek to the developed area in a similar alignment to the access road observed during NV5's site investigation. A second unpaved access road is observed extending westerly from the developed area up the hillside to the west on the adjacent property. More mature trees are visible within the developed area in the later photographs.

1982: The subject property appears to be relatively unchanged since 1963 with the exception of several structures previously visible in the western portion of the site that are no longer visible.

1993 and 1998: A new structure is first visible in 1993 in the western portion of the site, just west of the previously visible structures. This structure is similar in shape and location to the masonry recreational building we observed on-site during our site investigation. By 1998, two structures are visible in the northern portion of the site similar in shape and location as the modular residence and barn observed during the NV5 site visit.

2002 and Subsequent Photographs: No apparent changes can be seen to the subject property on the 2002 and subsequent photographs from that viewed on the 1998 photograph other than a general increase in vegetation.

2.2 REGIONAL GEOLOGY

The project site is situated in Diablo Range within the Coast Range geologic province west of the boundary with the Great Valley geologic province. This province is characterized as a geologically complex and seismically active region consisting of sub-parallel northwest-trending faults, mountain ranges and valleys. The oldest geologic units are the Jurassic-Cretaceous Franciscan Complex and Great Valley sequence which consist of sediments originally deposited in a marine environment. Subsequent younger rocks such as Tertiary-age volcanic and sedimentary rocks were deposited throughout the region. During the late Cretaceous through early Tertiary period, extensive folding and thrust faulting created complex geologic conditions across the region. In the valleys, bedrock is covered by thick alluvial soils and floodplain deposits that are incised by meandering river channels. In the mountains, colluvial soils and landslides cover a highly varied and discontinuous bedrock in conjunction with the regional faulting.

2.3 SITE GEOLOGY

Based on review of the *Geologic Map of the Hayward Quadrangle, Contra Costa and Alameda Counties, California*, published by the Dibblee Geological Foundation (Dibblee, T.W. and Minch, J.A., 2005), the geology underlying the subject site is comprised of Quaternary Holocene alluvial deposits, east of Cull Creek and the Monterey Formation west of Cull Creek.

The Holocene alluvial deposits generally consist of unweathered gravel, sand and silt deposited by present-day stream channels during the Holocene Epoch (approximately 11,700 years before present to present-day).

The Monterey Formation is characterized as marine clastic and biogenic sedimentary rock generally consisting of clay shale or claystone and siltstone and siliceous shale that is thin bedded to bedded formed during the middle to late Miocene Epoch (16 to 5 million years before present [mybp]).



2.4 REGIONAL FAULTING AND SEISMIC SOURCES

Regional faulting is primarily associated with the Hayward Fault Zone located west of the site and the Calaveras Fault Zone located to the east of the site. The southern extent of the Miller Creek Fault Zone is mapped in the vicinity of Cull Creek, which meanders through the project site. The Miller Creek Fault Zone is identified as an age undifferentiated Quarternary fault.

NV5 reviewed the Official Maps of Earthquake Fault Zones delineated by the California Geological Survey through December 2010, on the internet at <http://www.quake.ca.gov/gmaps/WH/regulatorymaps.htm>. These maps are updates to Special Publication 42, Interim Revision 2007 edition *Fault Rupture Hazard Zones in California*, which describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. Special Publication 42 and the 2010 online update indicate that the site is not located within an Alquist-Priolo active fault zone.

According to the 2010 Fault Activity Map of California by the California Geological Survey, (<http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#>), Geologic Data Map No. 6, the closest known active faults which have surface displacement within Holocene time (about the last 11,000 years) are the Hayward Fault and the Calaveras Fault. The mapped Hayward Fault Zone is located approximately 4 miles to the west of the subject site and the mapped Calaveras Fault Zone is located approximately 5 miles to the east of the subject site.

2.5 FIELD INVESTIGATION

NV5 performed a field investigation of the site on June 6, 2019. NV5's field engineer/geologist described the surface and subsurface soil, rock and groundwater conditions observed at the site using the procedures cited in the ASTM International (ASTM), Volume 04.08, *Soil and Rock; Dimension Stone; and Geosynthetics* as general guidelines. The field geologist described the soil color using the general guideline procedures presented in the Munsell® Soil Color Chart. Engineering judgment was used to extrapolate the observed surface and subsurface soil, rock and groundwater conditions to areas located between and beyond the subsurface exploratory locations. The surface, subsurface and groundwater conditions observed during the field investigation are summarized below.

2.5.1 Surface Conditions

NV5 observed the following surface conditions during the field investigation of the property. Figure 2 shows the project site boundaries and the approximate exploratory trench locations. At the time of NV5's site investigation, the site supported a single-family residence and outbuildings in the northeastern portion of the site and a large recreational building in the western portion of the site. Cull Creek meandered through the center of the site in a northwest to southeast trend. Mature trees also were observed throughout the project site.



2.5.2 Subsurface Conditions

The subsurface soil, rock and groundwater conditions were investigated by excavating exploratory trenches across the project site. The subsurface information obtained from this investigation method is described in the following subsections.

2.5.2.1 Exploratory Trench Information

NV5 provided engineering oversight for the excavation of 9 exploratory soil trenches at the project site. The trenches were advanced with a Kubota U35-4 equipped with a 24-inch bucket. Figure 2 shows the approximate locations of the subsurface exploratory excavations. The trenches were excavated to depths ranging from 4 to 7 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 trench using the ASTM D2487 Unified Soils Classification System (USCS) as guidelines for soil descriptions and the American Geophysical Union guidelines for rock descriptions. Representative bulk samples of the near-surface soil materials excavated from the exploratory trenches were sampled and placed in labeled sample bags.

Representative relatively undisturbed soil samples were collected from the exploratory trenches with a 2-inch, inside-diameter, split- spoon sampler equipped with steel liner sample tubes. The samples were collected from descript locations within the trenches. The sampler was driven into the soil using a 10-pound hand-operated hammer with an 18-inch drop. The steel liner tube samples were sealed with end-caps and labeled. The soil samples collected in the exploratory trenches 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 trench 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/rock material encountered.

A generalized profile of the soil, rock and groundwater conditions encountered to the maximum depth excavated (7 feet) for the project site is presented below. The soil and/or rock units encountered in the subsurface exploratory excavations were generally stratigraphically continuous across the site with some variations in gradations and thicknesses. The units encountered in general stratigraphic sequence during the subsurface investigation of the site are described below.

- **CL, Low Plasticity Clay Soil:** This soil is considered to be a native soil consisting of the following field estimated particle size percentages: 65 percent low plasticity silt and clay fines and 35 percent fine sand. This soil is predominantly dark brown with a Munsell® Soil Color Chart designation of (7.5YR 3/2). This soil was medium stiff to hard and slightly moist to moist 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 very dark grayish brown with a Munsell® Soil Color Chart designation of (2.5Y 3/2). This soil was stiff to hard and slightly moist to moist at the time of the subsurface investigation.

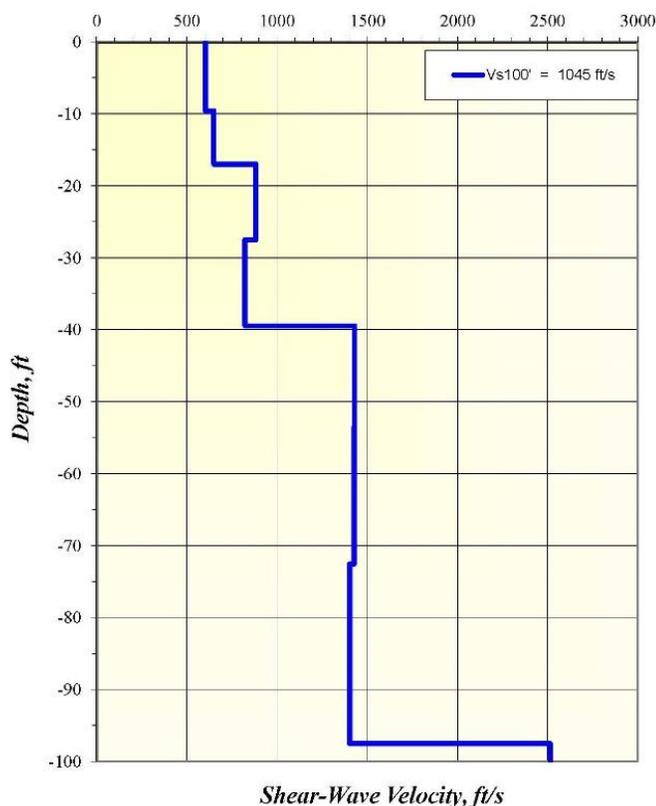


2.5.2.2 Seismic Refraction Microtremor Survey

The Seismic Refraction Microtremor Survey (SRMS) performed at the site used the SeisOpt® ReMi™ Vs30 method to determine the in-situ shear-wave (S-wave) velocity profile (Vs Model) of the uppermost 100 feet (30 meters) of soil beneath the site. The measured S-wave profile is used to determine the 2016 California Building Code (CBC) Site Class in accordance with Chapter 16, Section 1613.3.2 and Chapter 20 of ASCE 7-10.

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

70852.00.001 Mosaic Project, Castro Valley, CA: Vs Model



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 1045 feet per second (ft/s). This weighted shear wave velocity corresponds to the upper range of Site Class D (Stiff Soil Profile), as described in Chapter 20, Table 20.3-1 Site Classification of ASCE 7-10.

2.5.2.3 Groundwater Conditions

The groundwater table was not encountered in the exploratory trenches excavated during this site investigation. The moisture content of each soil unit described on the exploratory trench logs is considered the natural moisture within the vadose soil zone (soil situated above the groundwater table). However, fluctuations in soil moisture content and groundwater levels should be anticipated depending on precipitation, irrigation, runoff conditions and other factors. Based on our experience in the project area, seasonal saturation of near-surface soil should be anticipated, especially during and immediately after seasonal prolonged rainstorms.

NV5 reviewed available groundwater data within the Department of Water Resources Sustainable Groundwater Management Program's database (<http://sgma.water.ca.gov/webgis/?appid=SGMADataViewer>). Based on review of well completion reports completed for wells located on the project site and within approximately 2 miles of the site, the approximate depth to groundwater is 30 to 40 feet below ground surface. Therefore, NV5 does not anticipate a permanent groundwater table being encountered at the minimum elevations achieved in the site excavations. Seasonal infiltration into the shallow subsurface may create perched water conditions at contacts between soil and less weathered rock or competent rock. Perched groundwater may cause moisture intrusion into foundation crawl spaces or through concrete slab-on-grade floors, degradation of asphalt concrete (AC) pavements, and other adverse conditions. Mitigation measures such as gravel underdrains, elevated building pads, trench drains, vertical water barriers, or other methods may be required to intercept shallow groundwater or reduce potential adverse effects on project features.

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 D2166 Unconfined Compressive Strength of Cohesive Soil
- ASTM D2216 Soil Moisture Content
- ASTM D2487 Soil Classification by the USCS
- ASTM D2850 Unconsolidated-Undrained Triaxial Compression Test
- ASTM D2937 Density of Soil In-Place by Drive-Cylinder Method
- ASTM D4318 Atterberg Limits (Dry Method)

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

Table 3.0-1, Laboratory Test Results

Trench No.	Sample		ASTM Test Results ⁽¹⁾								
	No.	Depth	D2487 D2488	D2216	D2937	D422		D4318		D2166	D2850
		(ft)	USCS (sym)	Moisture Content (%)	Dry Density (pcf)	Passing No. 4 Mesh Sieve (%)	Passing No. 200 Mesh Sieve (%)	Plasticity Index (%)	Liquid Limit (%)	Unconfined Compressive Strength (psf)	UU Compressive Strength (psf)
T19-1	B1	1 - 3	CL			96.4	64.9	13	36		
T19-2	B1	2 - 3	CL					27	39		
T19-4	B1	1 - 2	CL					13	32		
T19-4	L1	1	CH	17.7	89.3					886.2	
T19-5	B1	3	CL					34	50		
T19-5	L1	6.5	CL	23.2	98.6						4,643.2

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

4.0 GEOLOGIC HAZARDS

NV5's evaluation of geologic hazards for the site was based on review of geologic maps and literature, regional aerial photographs, a site reconnaissance, and analysis of the soil and rock conditions encountered during the June 6, 2019 site investigation. This section provides additional information to meet the conditions of the 2016 CBC.

Portions of the property are mapped as being located within or adjacent to special geologic hazard zones designated by the California Geologic Survey or local building departments for liquefaction and landslides. The following presents NV5's evaluation of pertinent geologic hazards and their potential to negatively impact the site.

4.1 LIQUEFACTION

Soil liquefaction results when the shear strength of a saturated soil decreases to zero during cyclic loading that is generally caused by machine vibrations or earthquake shaking. Generally, saturated, clean, loose, uniformly graded sand and loose, silty sand soils of Holocene age are the most prone to undergo liquefaction, however, saturated, gravelly soil, and some clay-rich soil may be prone to liquefaction under certain conditions.

NV5's evaluation of the liquefaction potential for the site was based on literature review, the probabilistic seismic peak ground acceleration, and our site specific field data, which included exploratory trenches and a shear-wave velocity analysis. According to CGS Special Publication 117-A (2008), geophysical measurements of shear-wave velocities are appropriate methods to satisfy the screening evaluation of the resistance of soils to liquefaction. Accordingly, conservative "preliminary evaluation" methods (e.g., geologic assessments and/or shear-wave velocity measurements) often suffice for evaluation of their liquefaction potential. Based on the shear-wave velocity data measured at the site, NV5 determined that a more extensive study of the deeper subsurface was not warranted. NV5 respectfully requests the following factors be considered:

The site specific geology in the area of the proposed improvements consists of Monterey Formation formed during the middle to late Miocene Epoch (16 to 5 mybp). The Monterey Formation is characterized as marine clastic and biogenic sedimentary rock generally consisting of clay shale or claystone and siltstone and siliceous shale that is thin bedded to bedded. The soil generated from the Monterey Formation is predominately clay that varies from low plasticity to high plasticity. Soils with clay and silt contents greater than 30 percent typically are not prone to liquefaction.

Groundwater was not encountered in the trenches to the maximum depth explored of 7 feet bgs. However, groundwater data from nearby water wells indicates historically high groundwater levels are approximately 30 to 40 feet bgs and located within fractured rock of the Monterey Formation. Due to the predominant clay content of the soil and the weathered to slightly weathered rock of the Monterey Formation, it is NV5's opinion that the site subsurface conditions below the proposed building footprint make the probability of liquefaction occurring during ground shaking caused by a maximum considered earthquake (MCE) to be very low. Based on this information, NV5 believes that the age of the site geology, the groundwater conditions, and high clay content soil conditions above

the slightly to moderately weathered rock make the probability of liquefaction occurring during a nearby earthquake to be very low.

NV5 conducted a seismic refraction microtremor survey across the proposed building footprint. The seismic refraction survey used the SeisOpt® ReMi™ Vs30 method to determine the in-situ shear-wave (S-wave) velocity profile of the first 100-feet of soil beneath the site, which is an indication of density and shear strength of the soil deposits. This method is used for earthquake site response and liquefaction analysis and is particularly advantageous for use in areas with shallow soil profiles underlain by competent rock. A mean shear-wave velocity of the upper 100 ft of sediments (Vs100) of 1,045 ft/s, equal to 318 meters per second (m/s), was calculated from the seismic refraction shear wave profile data. The shear-wave profile for the soil beneath the proposed building indicates show the lowest shear-wave velocity of approximately 600 ft/s (182 m/s) indicative of the lowest density and shear strength soil is from the ground surface to a depth of approximately 10 ft bgs. The shear-wave velocity increases at depth to 650 ft/s (192 m/s) from 10 to 17 feet bgs, increases to over 800 ft/s (243 m/s) from 17 to 40 ft bgs, and increased to 1400 ft.s (426 m/s) and higher below 40 feet. The shear-wave velocity profile of the subsurface is presented on page 11 of this report. . These Vs values indicated still to dense soil and soft rock, which are not typically prone to liquefaction under strong ground shaking conditions.

NV5 concludes that the subsurface lacks the conditions to promote liquefaction based on the estimated PGA at the site, the seismic shear-wave velocity values for the subsurface, the age of the geology underlying the building, and the very deep groundwater conditions.

4.2 SEISMIC SETTLEMENT AND LATERAL SPREADING

Because the potential for liquefaction of the soil beneath the site is considered low, and the relatively shallow depth to bedrock, NV5 considers there to be a low probability for the occurrence of post-liquefaction settlement and lateral spreading that would be detrimental to the proposed site improvements.

4.3 LANDSLIDES

The existing topography at the site and near vicinity consists of moderate to steep sloping terrain. The project site is located in a region of known historical landslides; however, there were no mapped or observed indications of historic landslides, including rock falls, debris flows or deep and shallow failure on the site. Therefore, the potential for the occurrence or reoccurrence of a landslide hazard within the proposed building areas is considered to be low.

5.0 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 construction improvements provided that the geotechnical engineering design recommendations presented in this report are incorporated into the earthwork and structural improvement project plans. Prior to construction, NV5 should be allowed to review the proposed final earthwork grading plan and structural improvement plans to determine if the geotechnical engineering recommendations have been properly incorporated, are still applicable or need modifications.
2. NV5's primary concern is the presence of undocumented fills that were encountered in Trenches T19-7 through T19-9 that extended from approximately 3 to at least 5 feet below existing site grades. 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 6.1 of this report.
3. Based on the SeisOpt ReMi Vs30 shear-wave profile analysis, the site geology, and the observations within the exploratory trenches, the site soil profile can be modeled, according to the 2016 CBC, Chapter 16, and ASCE 7-10, 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 site geology, other field data, and literature review, NV5 believes that the site soil and groundwater conditions make the probability of liquefaction occurring during a nearby earthquake to be low.
5. Based on the site geology, other field data, and literature review, NV5 believes that the site soil and groundwater conditions make the probability of landslides occurring on the site to be low.
6. At the time of NV5's site investigation, the site supported a single-family residence and outbuildings in the northeastern portion of the site and a large recreational building in the western portion of the site. Cull Creek meandered through the center of the site in a northwest to southeast trend. Mature trees also were observed throughout the project site.
7. The native soil conditions observed to a maximum depth of 7 feet below the existing ground surface in our subsurface exploratory excavations (described relative to the existing ground surface) generally consisted of dark grayish brown silty clay (CH) underlain by brown silty clay (CL).
8. NV5's field and laboratory test data indicate that the silty clay (CH)/(CL) soil units encountered beneath the site has the following general geotechnical engineering properties: medium stiff to hard, low to high plasticity, and a low to moderate bearing capacity that is suitable for supporting shallow foundations.
9. Groundwater was not encountered in the exploratory trenches at the time of this subsurface investigation. 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 is located approximately 30 to 40 feet bgs. However, perched groundwater could be encountered depending on the time of year construction takes place.

6.0 RECOMMENDATIONS

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

6.1 EARTHWORK GRADING

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

6.1.1 Import Fill Soil

Import fill soil should meet the geotechnical engineering material properties described in Section 6.1.5.1 (Engineered Fill Construction with Non-Expansive Soil) of this report. Prior to importation to the site, the source generator should document that the import fill meets the guidelines set forth by the California Environmental Protection Agency (CalEPA) Department of Toxic Substances Control (DTSC) in their 2001 "Information Advisory, Clean Imported Fill Material." This advisory represents the best practice for characterization of soil prior to import for use as engineered fill. The NV5 project engineer should approve all proposed import fill soil for use in constructing engineered fills at the site.

6.1.2 Temporary Excavations

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

6.1.3 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 soil containing shallow vegetation roots and other deleterious materials. This highly organic topsoil can be stockpiled on-site and used for surface landscaping but should not be used for constructing compacted engineered fills. Grub the underlying 8 to 10 inches of soil to remove any large vegetation roots or other deleterious

material while leaving the soil in place. The NV5 project engineer or his/her representative should approve the use of any soil materials generated from the clearing and grubbing activities.

2. Remove all large shrub and tree roots and tree stumps. 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. Sufficient backfill soil should be placed and compacted in order to match the surrounding elevations and grades. The NV5 project engineer or his/her representative should observe and approve the preparation of the cavities and holes prior to placing and compacting engineered fill soil in the cavities and holes.
3. Completely remove all undocumented fill materials, as exposed in our exploratory excavations. Rocks and rubble with a greatest dimension larger than 6 inches will be referred to in this report as “oversized” materials. Oversized rock materials can be stockpiled on-site and used to construct engineered fills, however, they must be blended with on-site or imported soils and placed at or near the bottom of deep fills but not shallower than 2 feet from the finished subgrade surface. Oversized rubble materials also can be broken down into pieces 6 inches or smaller, blended with on-site or imported soils and placed at or near the bottom of deep fills but not shallower than 2 feet from the finished subgrade surface. The oversized rocks should be placed with enough space between them to avoid clustering and the creation of void space. The NV5 project engineer or his/her representative should approve the use and placement of all oversized rock materials prior to constructing compacted engineered fills.
4. Excessively large amounts of vegetation, other deleterious materials and oversized rock materials should be removed from the site.

6.1.4 Native Soil Preparation for Engineered Fill Placement

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

1. The native soil should be scarified to a minimum depth of 8 inches below the existing land surface or stripped and grubbed surface and then uniformly moisture conditioned. If the soil is classified as a coarse-grained soil by the USCS (i.e., GP, GW, GC, GM, SP, SW, SC or SM) then it should be moisture conditioned to within ± 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 should be prepared as specified in Section 6.1.5.2 (Engineered Fill Construction with Expansive Soils).
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 NV5 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 NV5 project engineer or his/her field representative by excavating test pads with the on-site earth moving equipment. Native soil preparation beneath concrete slab-on-grade structures (i.e., floors, sidewalks, patios, etc.) should be prepared as specified in Section 6.2 (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 NV5 project engineer or his/her field representative to be firm, competent and relatively unyielding. The NV5 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 6.1.4-1 or as modified by the NV5 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 6.1.4-1, Minimum Testing Frequencies

ASTM No.	Test Description	Minimum Test Frequency ⁽¹⁾
D1557	Modified Proctor Compaction Curve	1 per 1,500 CY or Material Change ⁽²⁾
D6938	Nuclear Density and Nuclear Moisture Content	1 per 250 CY
Notes: (1) These are minimum testing frequencies that may be increased or decreased at the NV5 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		

6.1.5 Engineered Fill Construction with Testable Earth Materials

Engineered fills are constructed to support structural improvements. Engineered fills should be constructed using non-expansive soil as described in Section 6.1.5.1. If possible, the use of expansive soil for constructing engineered fills should be avoided. If the use of expansive soil cannot be avoided, then engineered fills should be constructed as described in Section 6.1.5.2 or as modified by the NV5 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.

6.1.5.1 Engineered Fill Construction with Non-Expansive Soil

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

1. Non-expansive soil used to construct engineered fills should consist predominantly of materials less than ½-inch in greatest dimension and should not contain rocks greater than 6 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 NV5 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 12-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 6.1.5.1-1 or as modified by the NV5 project engineer to better suit the site conditions.

Table 6.1.5.1-1, Minimum Testing Frequencies for Non-Expansive Soil

ASTM No.	Test Description	Minimum Test Frequency ⁽¹⁾
D1557	Modified Proctor Compaction Curve	1 per 1,500 CY or Material Change ⁽²⁾
D6983	Nuclear Moisture and Density	1 per 250 CY
Notes:		
(1)	These are minimum testing frequencies that may be increased or decreased at the NV5 project engineer’s discretion on the basis of the site conditions encountered during grading.	
(2)	Whichever criteria provide the greatest number of tests.	
ASTM	= ASTM International	
CY	= cubic yards	
No.	= number	

7. The moisture content, density and relative percent compaction of all engineered fills should be tested by the NV5 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 NV5 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 6.1.4, Paragraph 3.

6.1.5.2 Engineered Fill Construction with Expansive Soil

NV5 did encounter potentially expansive soil within the shallow soil, or zones within the foundation loads and slab-on-grade floors that would be influenced by shrinking or swelling conditions. NV5 recommends removing expansive soil from below the building footprints to avoid the potential damage that may be caused by soil heave.

If expansive soils are encountered during grading of the site and if the property owner desires to use the expansive soil to construct engineered fills, or have the building foundations or concrete-slab-on-grade floors bear directly upon the expansive soil, then the following three options should be considered. Each option has inherent risks and associated costs relative to future problems associated with expansive soil including shrinking and settlement (downward movement) and/or and swell and heave (upward movement) of foundations and concrete slab-on-grade floors. The options are presented in the general order of decreasing cost but increasing risk with regards to future problems related to soil shrink-swell behavior. Prior to implementing any of these options, NV5 should be notified so that further evaluation of the potentially expansive soil can be completed and these recommendations confirmed or modify NV5's recommendations as appropriate, if necessary.

Option 1 Remove and Replace with Non-Expansive Soil (NV5 Preferred Option-Lowest Shrink-Swell Behavior Risk):

This mitigation option has the lowest inherent risk of incurring future problems regarding settlement and/or heave of foundations and concrete slab-on-grade floors. This option consists of removing the expansive soil to a depth to be determined by the project geotechnical engineer. NV5 estimates that expansive soil, if encountered at the site, should be removed completely or removed to a minimum depth of 2 feet below the bottom of the building foundations and concrete slab-on-grade floors, whichever creates the greater depth below the adjacent finished grade surface. The actual removal depth or depths should be evaluated by the project geotechnical engineer's field representative during grading and may be either increased or decreased depending upon the site conditions observed.

Non-expansive soil should then be placed, moisture conditioned and compacted to achieve the finished grades as described in Section 6.1.5.1 of this report. This option, when compared to the other two options, generally incurs the greatest upfront costs to the project but has the least risk for future problems arising from the high shrink-swell behavior of the soil. Repair of future problems due to soil shrink-swell behavior is generally from 10 to 100 times costlier than the cost of removing and replacing with non-expansive soil during initial grading.

Option 2 Expansive Soil Treatment with High Calcium Lime and Fly-Ash (Moderate Shrink-Swell Behavior Risk):

This mitigation option has an intermediate (moderate) inherent risk of incurring future problems regarding settlement and/or heave of foundations and concrete slab-on-grade floors. This option consists of mixing high calcium lime and fly-ash with the on-site expansive soil to reduce the expansive shrink-swell behavior of the soil. This option, when compared to the other two options, generally incurs an intermediate upfront cost to the project with an intermediate risk for future problems arising from the high shrink-swell behavior of the soil. NV5 did not evaluate the percentages of high calcium lime and fly-ash to be mixed with on-site expansive soil as part of the

geotechnical engineering investigation work scope. If this option is selected by the owner to mitigate the on-site expansive soils, then NV5 should be consulted to prepare a proposed work scope to evaluate and develop construction specifications for lime and fly-ash treatment of the onsite expansive soil.

Option 3 Reworking Expansive Soil With Use of Post-Tensioned Concrete Slab-On-Ground Surface Reinforced Floors (Highest Shrink-Swell Behavior Risk):

This mitigation option has the highest inherent risk of incurring future problems with settlement and/or heave of foundations and concrete slab-on-grade floors. This option consists of reworking the existing on-site expansive soil to reduce its expansive shrink-swell behavior and the construction of post-tensioned concrete slab-on-ground surface reinforced floors. The post-tensioned reinforced slab-on-ground surface floors should be designed by a California-licensed civil engineer. This option, when compared to the other two options, generally incurs the lowest upfront cost to the project with the highest risk for future problems arising from the high shrink-swell behavior of the soil. Construction of engineered fills with expansive soil should be performed as described below; however, these recommendations may need to be revised by the project geotechnical engineer during grading depending upon the actual site conditions encountered. The project geotechnical engineer should be notified prior to implementing this expansive soil mitigation approach to determine if alternative foundation and concrete slab-on-grade floor designs will be necessary (i.e., pier and grade-beams, post-tension slabs, among others).

1. Expansive soil used to construct engineered fills should consist predominantly of materials less than 1-inch in greatest dimension and should not contain rocks greater than 3 inches in greatest dimension (oversized material). Expansive soil will have a PI greater than $PI > 20$ as determined by ASTM D4318 Atterberg Indices test. Oversized materials can be placed at or near the bottom of deep fills, but not within 3.0 feet of the finished subgrade surface or within 2.0 feet of the foundation bottom. Deep fills are defined as fills that are greater than 10 feet in vertical thickness. Oversized materials should be spread apart to prevent clustering so that void spaces are not created. The project geotechnical engineer or project geotechnical engineer's field representative should approve the use of over sized materials for constructing engineered fills.
2. Expansive soil used to construct engineered fills should be uniformly moisture conditioned to within 2 to 4 percentage points greater than the ASTM D1557 optimum moisture content. The actual moisture content should be reviewed by the project geotechnical engineer to determine if this preliminary moisture content range is appropriate or should be modified.
3. Engineered fills should be constructed by placing uniformly moisture-conditioned expansive soil in maximum 12-inch-thick loose lifts (layers) prior to compacting.
4. The expansive soil should then be compacted to achieve a minimum relative compaction of 88 percent and a maximum relative compaction of 92 percent of the ASTM D1557 maximum dry density. The actual percent relative compaction should be reviewed by the project geotechnical engineer to determine if this preliminary relative percent compaction range is appropriate or should be modified.
5. Field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 6.1.5.2-1 or as modified by the project geotechnical engineer to better suit the site conditions.

Table 6.1.5.2-1, Minimum Testing Frequencies for 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 100 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

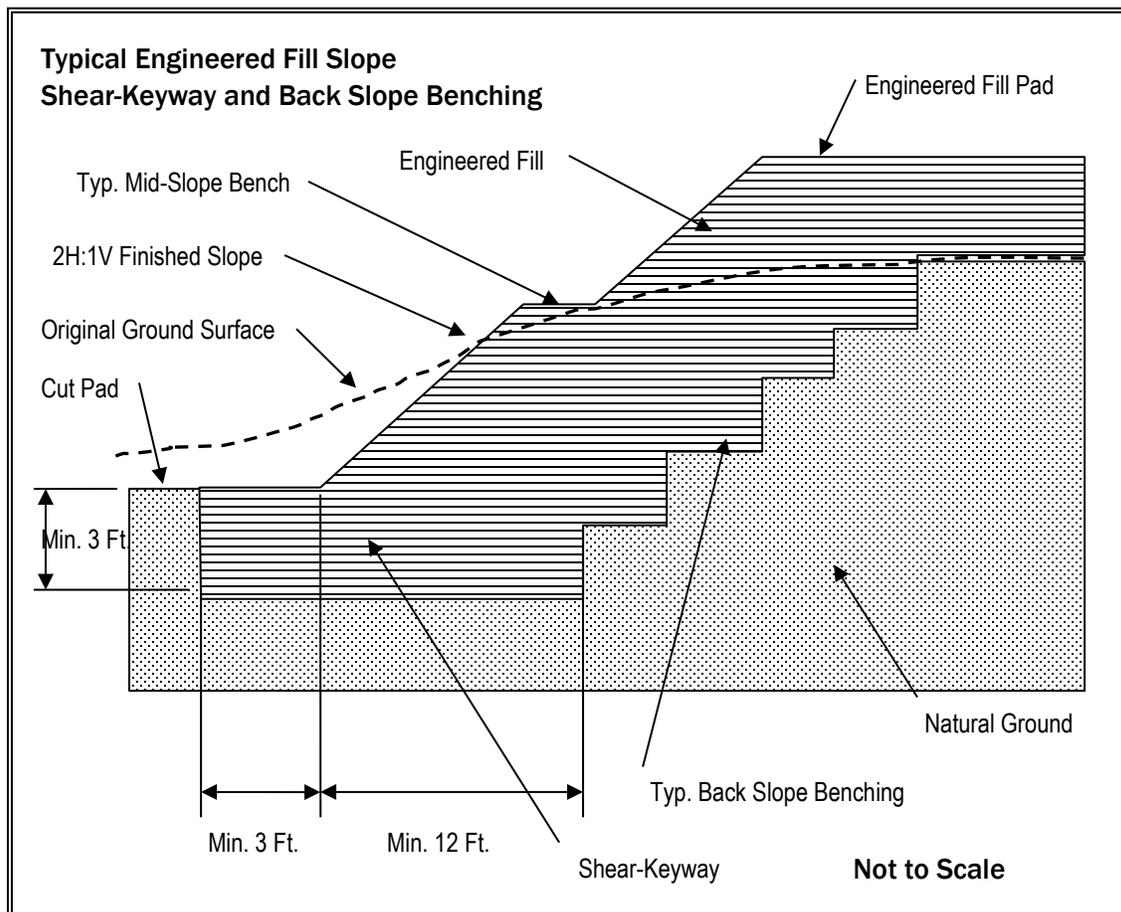
- The earthwork contractor should compact each loose soil lift with a tamping foot compactor such as a CAT 815 Compactor or equivalent as approved by the project geotechnical engineer. A smooth steel drum roller compactor should not be used to compact loose soil lifts of engineered fills with expansive soil, however, it may be used at the finished subgrade to finish the surface following the completion of compaction.
- The moisture content, density and relative percent compaction of all engineered fills constructed with expansive soil should be tested by the project geotechnical engineer's field representative during construction to evaluate whether the compacted soil meets or exceeds the minimum compaction and moisture content requirements. The earthwork contractor shall assist the project geotechnical engineer's field representative by excavating test pads with the onsite earth moving equipment.
- The prepared finished grade or finished subgrade soil surface constructed with expansive soil 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 minimum tire pressure should be 65 pounds per square inch (psi). The proof-rolled surface should be visually observed by the project geotechnical engineer or the project geotechnical engineer's field representative to be firm, competent and relatively unyielding. The project geotechnical engineer or the project geotechnical engineer's field representative may also evaluate the surface material by hand probing with a 1/4-inch-diameter steel probe; however, this evaluation method should not be performed in place of proof-rolling as described in the preceding.

6.1.6 Fill Slope Grading

Fill slopes should be graded as described below.

- Fill slopes should be graded with a maximum slope gradient of horizontal to vertical ratio (H:V) 2 H:1V, and with a maximum vertical height of 20 feet. If fill slopes are to be graded steeper than 2 H:1V and/or with a vertical height greater than 20 feet, then NV5 should be notified so that slope stability analysis of the proposed slope configuration can be performed, and revised recommendations provided.
- A shear-keyway should be graded at the base of the fill slope prior to constructing the fill slope. The shear-keyway should be a minimum of 15-feet wide and extend to a minimum depth of 3 feet below the finished subgrade surface, or deeper as determined by the project engineer

during grading. The shear-keyway base should be graded with a minimum slope gradient of 2 percent toward the inside fill slope surface.



3. Fill slopes should be graded in horizontal lifts to the lines and grades shown on the grading plans. The design-finished grade of a fill slope should be achieved by overbuilding the slope face and then cutting it back to the design-finished grade. Fill slopes should not be graded (extended horizontally) by compacting moisture conditioned, loose soil lifts on the slope face as thin veneer layers. In other words, do not construct engineered fill slopes by placing and compacting successive thin layers (veneers) of soil over the fill slope face at an inclination that is roughly coincident with the final fill slope horizontal to vertical slope ratio. The in-slope edge of each horizontal lift should be benched into the firm, competent and relatively unyielding soil of the natural ground slope.
4. If groundwater seepage from the slope and/or shear-keyway areas is encountered during grading, or if the site conditions indicate that groundwater seepage does occur during the wet winter season, then NV5 should be notified so that NV5 can assess the conditions and provide a design for installation of permanent dewatering subdrains.

5. Surface benches should be graded into the finished fill slope with a minimum width of 10 feet and with maximum vertical intervals of 15 feet between benches, or at mid-slope height if the total vertical slope height is between 15 feet and 30 feet.
6. Benches should be graded with a minimum slope gradient of 2 percent toward the inside fill slope surface. In other words, the bench slope gradient should cause surface water to drain toward the fill slope side of the bench (not over and down the fill slope face).
7. Fill soils used to construct slopes should be uniformly moisture conditioned, placed in loose lifts, and compacted as described in Sections 5.1.5 and 5.1.6.

6.1.7 Cut Slope Grading

Cut slopes should be graded as described below.

1. Cut slopes should be graded with a maximum slope gradient of 2H:1V and with a maximum vertical height of 20 feet. If cut slopes are to be graded steeper than 2H:1V and/or with a vertical height greater than 20 feet, then NV5 should be notified so that NV5 can perform a slope stability analysis of the proposed slope configurations and provide revised recommendations, if necessary.
2. Surface benches should be graded into the finished cut slope with a minimum width of 10 feet and with maximum vertical intervals of 20 feet between benches, or at the mid-slope height if the total vertical slope height is greater than 20 feet but less than 30 feet.
3. The benches should be graded with a minimum slope gradient of 2 percent toward the cut. In other words, the bench slope gradient should cause surface water to drain toward the cut slope side of the bench (not over and down the cut slope face).

6.1.8 Erosion Controls

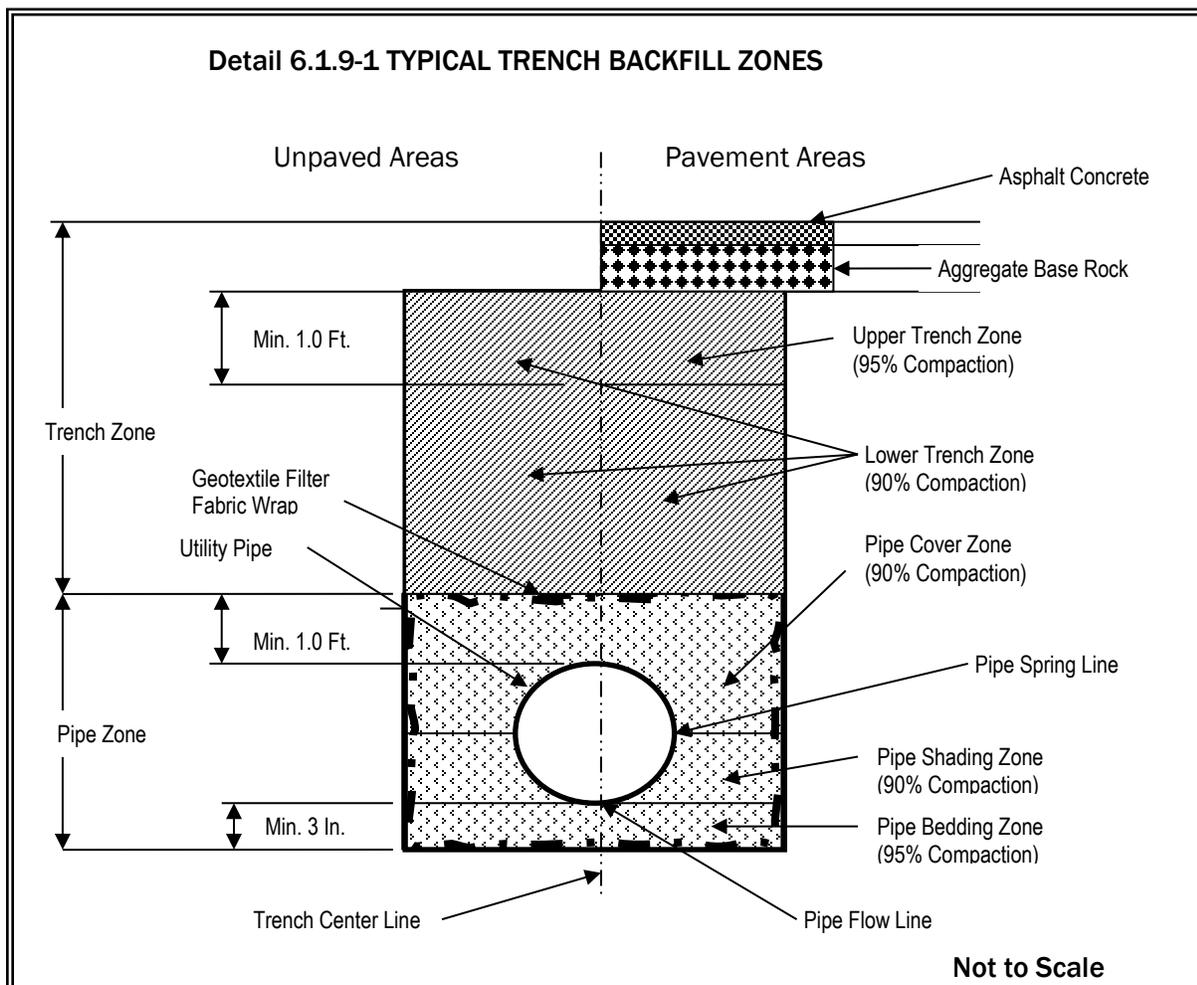
Erosion controls should be installed as described below.

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.

6.1.9 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.
- 2. Trench Shoring:** All utility trenches that are excavated deeper than 4 feet bgs are required by California OSHA to be shored with bracing equipment or sloped back to an appropriate slope gradient prior to being entered by any individuals.
- 3. Trench Dewatering:** NV5 does not anticipate that the proposed underground utility trenches will encounter shallow groundwater. However, if the utility trenches are excavated during the winter rainy season, then shallow or perched groundwater may be encountered. The earthwork contractor may need to employ dewatering methods as discussed in Section 6.1.10 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 6.1.9-1 below.



- **Pipe Zone Backfill Material Type:** Trench backfill used within the pipe zone, which includes the bedding zone, the shading zone and the cover zone, should consist of ¾-inch-minus, washed, crushed rock. The crushed rock particle size gradation should meet the following requirements (percentages are expressed as dry weights using ASTM D422 test method): 100 percent passing the ¾-inch sieve, 80 to 100 percent passing the ½-inch sieve, 60 to 100 percent passing the 3/8-inch sieve, 0 to 30 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve. If groundwater is encountered within the trench during construction, perched water is anticipated in the trench, or if groundwater is expected to rise during the rainy season to an elevation that will infiltrate the pipe zone within the trench, then the pipe zone material should be wrapped with a minimum 6 ounce per square yard, non-woven geotextile filter fabric such as TenCate® Mirifi N140 or an approved equivalent. The geotextile seam should be located along the trench centerline and have a minimum 1-foot overlap. If the utility pipes are coated with a corrosion protection material, then the pipes should be wrapped with a minimum 6 ounce per square yard, non-woven, geotextile cushion fabric such as TenCate® Mirifi N140 or an approved equivalent. The geotextile cushion fabric should have a minimum 6-inch seam overlap. The geotextile cushion fabric will protect the pipe from being scratched by the crushed rock backfill material.
 - **Pipe Bedding Zone Compaction:** Trench backfill soil placed in the pipe bedding zone (beneath the utilities) should be a minimum of 3 inches thick, moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
 - **Pipe Shading Zone Compaction:** Trench backfill soil placed within the pipe shading zone (above the bedding zone and to a height of one pipe radius above the pipe spring line) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The pipe shading zone backfill material should be shovel-sliced to remove voids and to promote compaction.
 - **Pipe Cover Zone Compaction:** Trench backfill soil placed within the pipe cover zone (above the pipe shading zone to 1 foot over the pipe top surface) should be moisture conditioned to within ± 3 percentage points of the ASTM D1557 optimum moisture content and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
5. **Trench Zone Backfill and Compaction Requirements:** The trench zone backfill materials consist of both lower and upper zones, as discussed below.
- **Trench Zone Backfill Material Type:** Soil used as trench backfill within the lower and upper intermediate zones, as shown on the preceding figure, should consist of non-expansive soil with a PI of less than or equal to 15 (based on ASTM D4318) and should not contain rocks greater than 3 inches in greatest dimension.
 - **Lower Trench Zone Compaction:** Soil used to construct the lower trench zone backfills should be uniformly moisture conditioned to within 0 and 4 percentage points of the ASTM D1557 optimum moisture content, placed in maximum 12-inch-thick loose lifts prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

- Upper Trench Zone Compaction (Pavement Areas):** Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 and 4 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 8-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
 - Upper Trench Zone Compaction (Non-Pavement Areas):** Soil used to construct the upper trench zone backfills should be uniformly moisture conditioned to within 0 and 2 percentage points greater than the ASTM D1557 optimum moisture content, placed in maximum 6-inch-thick loose lifts (layers) prior to compacting and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
- 6. CQA Testing and Observation Engineering Services:** The moisture content, dry density and relative percent compaction of all engineered utility trench backfills should be tested by the NV5 project engineer’s field representative during construction to evaluate whether the compacted trench backfill materials meet or exceed the minimum compaction and moisture content requirements presented in this report. The earthwork contractor shall assist the NV5 project engineer’s field representative by excavating test pads with the on-site earth moving equipment.
- Compaction Testing Frequencies:** The field and laboratory CQA tests should be performed consistent with the testing frequencies presented in Table 6.1.9-1 or as modified by the NV5 project engineer to better suit the site conditions.

Table 6.1.9-1, 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 NV5 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 aggregate base (AB) rock surface and/or finished subgrade soil surface of utility trench backfill should be proof-rolled, as mentioned above in Section 6.1.4, Paragraph 3.

6.1.10 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® Mirifi® N140 or an approved equivalent. The geotextile filter fabric should have minimum 1-foot overlapped seams. The granular rock should meet or exceed the following gradation specifications (all percentages are expressed as dry weights using ASTM D422 test method): 100 percent passing the 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.
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.

6.1.11 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 Castro Valley area is that their corrosion potential is relatively low. 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.

6.1.12 Subsurface Groundwater Drainage

Due to the near-surface cohesive soils and relatively shallow depth to sedimentary rock, 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.

6.1.13 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.

6.1.14 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 have been implemented and, if necessary, to provide additional and/or modified recommendations.
2. NV5 should be allowed to perform CQA monitoring of all earthwork grading performed by the contractor to determine whether the recommendations have been implemented and, if necessary, to provide additional and/or modified recommendations.
3. NV5's experience, and that of the engineering profession, clearly indicate 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.

6.2 STRUCTURAL IMPROVEMENTS

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

6.2.1 Seismic Design Parameters

NV5 developed the code-based seismic design parameters in accordance with Section 1613 of the 2016 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 2010 ASCE-7 Standard (erratum released March 2013) and the 2015 International Building Code (2015 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 6.2.1-1 . The Seismic Design Maps report from the SEAOC analysis is provided in Appendix D.

6.2.2 Seismic Design Category

Based on the short period response acceleration ground motion parameters above ($S_{DS} = 1.10$) and the Risk Category of I or II, and III, the Seismic Design Category is D. Based on the 1-S period response acceleration ground motion parameters above ($S_{D1} = 0.649$) and the Risk Category of I or II, and III, the Seismic Design Category is D. Therefore, the Seismic Design Category for the site is D.

6.2.3 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 22-7 from ASCE 7-10. The PGA_M was calculated using the following equation:

$$PGA_M = F_{PGA}PGA = 1.00 \times 0.638 = 0.638 \text{ g}$$

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

6.2.4 Shallow Foundations

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

1. The base of all shallow foundations should bear on firm, competent non-expansive native soil, or non-expansive engineered fill compacted consistent with the earthwork recommendations of Section 6.1.
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 6.2.4-1.
3. The bearing capacities to be used for structural design of shallow foundations embedded in either non-expansive native soil or non-expansive engineered fill are presented in Table 6.2.4-1.
 - The calculated factor of safety (FS) for allowable bearing pressures including live plus dead loads is 3.0 for all foundation embedment depths.
 - The allowable bearing pressure capacities were increased by a factor of 1.33 to include wind or seismic short-term loads.
 - The project structural engineer of record should review the factor of safety and confirm that it is not less than the over-strength factor for this structure.

Table 6.2.4-1, Foundation Bearing Pressures for Shallow Continuous Strip and Isolated Spread Foundations

Minimum Foundation Embedment Depth	Maximum Ultimate Bearing Pressures For Live + Dead Loads	Maximum Allowable Bearing Pressures For Live + Dead Loads	Maximum Allowable Bearing Pressures For Live + Dead + Wind or Seismic Loads	Allowable Safety Factor (Ultimate/Total)
(in)	(psf)	(psf)	(psf)	(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 = 300 (H), pounds per square foot (psf), where H = foundation embedment depth (feet) below lowest adjacent soil surface.
 - b. Foundation bottom sliding friction coefficient = 0.30 (dimensionless).
5. Minimum steel reinforcement for continuous strip foundations should consist of four No. 4 bars with two bars placed near the top and two bars 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 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 inch. Differential

settlement between similarly loaded, adjacent foundations is expected to be about 1/2 inch, provided the foundations are founded into similar materials (e.g., all on competent and firm engineered fill, native soil or rock).

11. Prior to placing concrete in any foundation excavation, the project geotechnical engineer or his/her field representative should observe the excavations to document that the following requirements have been achieved: minimum foundation dimensions, minimum reinforcement steel placement and dimensions, removal of all loose soil, rock, wood debris or other deleterious materials, and that firm and competent native or engineered fill soil is exposed along the entire foundation excavation bottom. Strict adherence to these requirements is paramount to the satisfactory behavior of a building foundation. Minor deviations from these requirements can cause the foundations to undergo minor to severe amounts of settlement which can result in cracks developing in the foundation and adjacent structural members, such as concrete slab-on-grade floors.

6.2.5 Retaining Walls Entirely Above the Groundwater Table

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

1. The retaining wall recommendations for static loading conditions are based on Rankine earth pressure theory published by W.J.M. Rankine (1857). The retaining wall recommendations for seismic loading conditions are based on the published work by Mononobe, N. and Matsuo, H. (1929).
2. Retaining walls should be founded on firm competent bedrock or engineered fill consistent with the requirements of Section 6.1.
3. The retaining wall should be designed using the geotechnical engineering design parameters presented in Table 6.2.5-1.
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 6.2.6.
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 6.2.5-1, 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) ⁽¹⁾	50 (H) ⁽⁵⁾	9 (H)
Wall Passive Condition Pressures (psf) ⁽²⁾	300 (H)	9 (H)
Wall At-Rest Condition Pressure (psf) ⁽³⁾	70 (H)	9 (H)
P _{active} Force Located Above Foundation Base	0.33 (H)	Not Applicable
P _{passive} Force Located Above Foundation Base	0.33 (H)	Not Applicable
P _{at-rest} Force Located Above Foundation Base	0.33 (H)	Not Applicable
P _{earthquake} Force Located Above Foundation Base	Not Applicable	0.60(H)
Maximum Allowable Foundation Bearing Capacity (psf), (Live + Dead Loads)	2,500	2,500
Maximum Allowable Foundation Bearing Capacity (psf) (Live + Dead + Wind or Seismic Loads)	3,325	3,325
Minimum Foundation Embedment Depth (in)	18	18
Foundation Bottom Friction Coefficient (dim.) ⁽⁴⁾	0.30	0.30

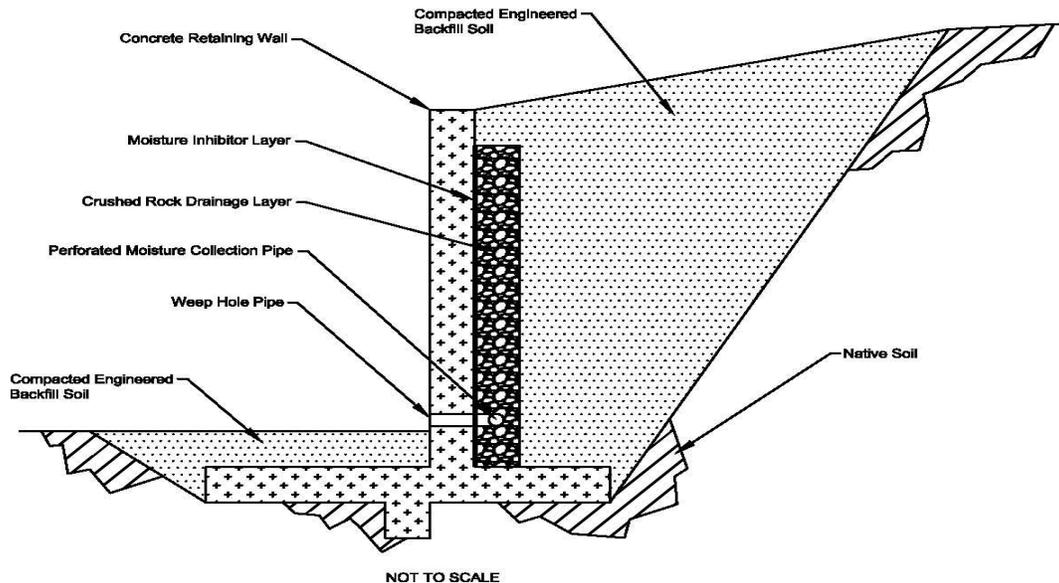
Notes:

- (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).

6.2.6 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, elevator shafts, 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



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 in order to prevent build up 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 water proofing 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 water proofing 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 water proofing system (as described below) to separate the backfill soil from the wall. The drain rock should extend from the wall bottom to within 12 inches of the wall top. A minimum 6-ounce per square yard (oz/sy) non-woven geotextile fabric, such as Amoco 4506 manufactured by Amoco Fabrics and Fibers Company or equivalent should be used.
 - c. **Geosynthetic Composite Drainnet (Geonet) Method:** Place a geosynthetic composite drain-net (geonet) directly against the wall or water proofing 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 orientated 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.

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

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

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

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

- ACI 302.1R-15, Guide for Concrete Floor and Slab Construction, reported by ACI Committee 302.

- ASTM E1643-18a, Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.
- ASTM E1745-17, Standard Specifications for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs.
- ASTM F710-19, Standard Practice for Preparing Concrete Floors to Receive Resilient Flooring.

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

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

Rebar: As a minimum, use No. 3 rebar (ASTM A615/A615M-18e1 Grade 60), tied and placed with 18-inch centers in both directions (perpendicular) and supported on concrete “dobies” to position the rebar in the center of the slab during concrete pouring. NV5 does not recommend that the steel reinforcements of the concrete slab-on-grade floor be tied into the perimeter or interior continuous strip foundations or interior isolated column foundations. In other words, we recommend that the concrete slab-on-grade floors be constructed as independent structural members so that they can move (float) independently from the foundation structures.

3. **Underslab Vapor-Moisture Retarder Membrane:** The underslab retarder membrane should be placed in areas with moisture sensitive floor coverings as a floor component that will minimize transmission of both liquid water and water vapor transmission through the concrete slab-on-grade floor. NV5 recommends using at a minimum a Class A (ASTM E1745-17), minimum 10-mil-thick, plastic, vapor-moisture, retarder membrane material such as Stego Wrap® underslab vapor retarder membranes or equivalents. Additionally, the following materials are recommended: Stego® Tape and Stego® Mastic or equivalents to seal membrane joints and any utility penetrations.

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

properties. A waterproofing consultant should be contacted to provide detailed recommendations if moisture sensitive flooring materials will be installed on the concrete slab-on-grade floors.

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

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

5. **Subgrade Soil Preparation:** The subgrade soil should be prepared and compacted consistent with the recommendations of Section 6.1. The top 12 inches of the non-expansive soil should be compacted to a minimum of 90 percent of the ASTM D1557 dry density with relatively uniform moisture content within ± 3 percentage points of the ASTM D1557 optimum moisture content.
6. **Crack Control Grooves:** Crack control grooves should be installed during placement or saw cuts should be made in accordance with the ACI and Portland Cement Association (PCA) specifications. Generally, NV5 recommends that expansion joints be provided between the slab and perimeter footings, and that crack control grooves or saw cuts are installed on 10-foot-centers in both directions (perpendicular).
7. **Field Observations:** Field observations should be made by an NV5 construction monitor of all concrete slab-on-grade surfaces and installed steel reinforcements prior to pouring concrete.

6.2.8 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 structural engineer should design the necessary concrete slab-on-grade floor thickness and steel reinforcements.

1. The recommended modulus of subgrade value of 200 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 5-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 structural 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/A 615M-04 Grade 60 reinforcement), 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.
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 6.1. 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.

7.0 REFERENCES

- American Society of Civil Engineers, 2010. *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10).
- ASTM International (ASTM), 1992. *Soil and Rock; Dimension Stone; and Geosynthetics*. Volume 04.08.
- CGS, 1997. *Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps*. Special Publication 42. Interim Revision.
- Dibblee, T.W. and Minch, J.A., 2005, *Geologic Map of the Hayward Quadrangle, Contra Costa and Alameda Counties, California*, Dibblee Geological Foundation.
- GBA 2016. *Important Information about Your Geotechnical Engineering Report*.
- Mononobe, N. and Matsuo, H., 1929, *Determination of Earth Pressures During Earthquakes*, Proceedings of the World Engineering Congress, Tokyo, Japan, Vol. 9, paper 388.
- Rankine, W.J.M., 1857, *On the Stability of Loose Earth*, Philosophical Transactions of the Royal Society, London, Vol. 147.

8.0 LIMITATIONS

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

1. This report should not be relied upon without review by NV5 if a period of 24 months elapses between the issuance report date shown above and the date when construction commences.
2. NV5's professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in Northern California. No warranties are either expressed or implied.
3. NV5 provided engineering services for the site project consistent with the work scope and contract agreement presented in the proposal and agreed to by the client. The findings, conclusions and recommendations presented in this report apply to the conditions existing when NV5 performed the services and are intended only for the client, purposes, locations, timeframes and project parameters described herein. NV5 is not responsible for the impacts of any changes in environmental standards, practices or regulations subsequent to completing the services. NV5 does not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of the client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.
4. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. The validity of the conclusions and recommendations presented in this report can only be made by NV5; therefore, NV5 should be allowed to review all project changes and prepare written responses with regards to their impacts on the conclusions and recommendations. Additional fieldwork and laboratory testing may be required for NV5 to develop any modifications to the recommendations. The cost to review project changes and perform additional fieldwork and laboratory testing necessary to modify the recommendations is beyond the scope-of-services presented in this report. Any additional work will be performed only after receipt of an approved scope-of-work, budget and written authorization to proceed.
5. The analyses, conclusions and recommendations presented in this report are based on the site conditions as they existed at the time NV5 performed the surface and subsurface field investigations. NV5 has assumed that the subsurface soil and groundwater conditions encountered at the location of the exploratory trenches 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 trenches 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 2016)

Important Information about This

Geotechnical-Engineering Report

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may

perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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



Telephone: 301/565-2733

e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B:

Historical Aerial Photographs



Mosaic Project

17031 Cull Canyon Road

Castro Valley, CA 94552

Inquiry Number: 5677202.1

June 10, 2019

The EDR Aerial Photo Decade Package



6 Armstrong Road, 4th floor
Shelton, CT 06484
Toll Free: 800.352.0050
www.edrnet.com

EDR Aerial Photo Decade Package

06/10/19

Site Name:

Mosaic Project
17031 Cull Canyon Road
Castro Valley, CA 94552
EDR Inquiry # 5677202.1

Client Name:

Holdrege & Kull Consultants
792 Searls Avenue
Nevada City, CA 95959
Contact: Dominic Potestio



Environmental Data Resources, Inc. (EDR) Aerial Photo Decade Package is a screening tool designed to assist environmental professionals in evaluating potential liability on a target property resulting from past activities. EDR's professional researchers provide digitally reproduced historical aerial photographs, and when available, provide one photo per decade.

Search Results:

<u>Year</u>	<u>Scale</u>	<u>Details</u>	<u>Source</u>
2016	1"=500'	Flight Year: 2016	USDA/NAIP
2012	1"=500'	Flight Year: 2012	USDA/NAIP
2009	1"=500'	Flight Year: 2009	USDA/NAIP
2006	1"=500'	Flight Year: 2006	USDA/NAIP
1998	1"=500'	Flight Date: August 27, 1998	USDA
1993	1"=500'	Acquisition Date: July 10, 1993	USGS/DOQQ
1982	1"=500'	Flight Date: July 05, 1982	USDA
1979	1"=500'	Flight Date: August 16, 1979	USDA
1968	1"=500'	Flight Date: April 22, 1968	USGS
1963	1"=500'	Flight Date: July 18, 1963	EDR Proprietary Aerial Viewpoint
1958	1"=500'	Flight Date: July 21, 1958	USGS
1950	1"=500'	Flight Date: March 13, 1950	USDA
1949	1"=500'	Flight Date: October 13, 1949	USGS
1946	1"=500'	Flight Date: July 26, 1946	USGS

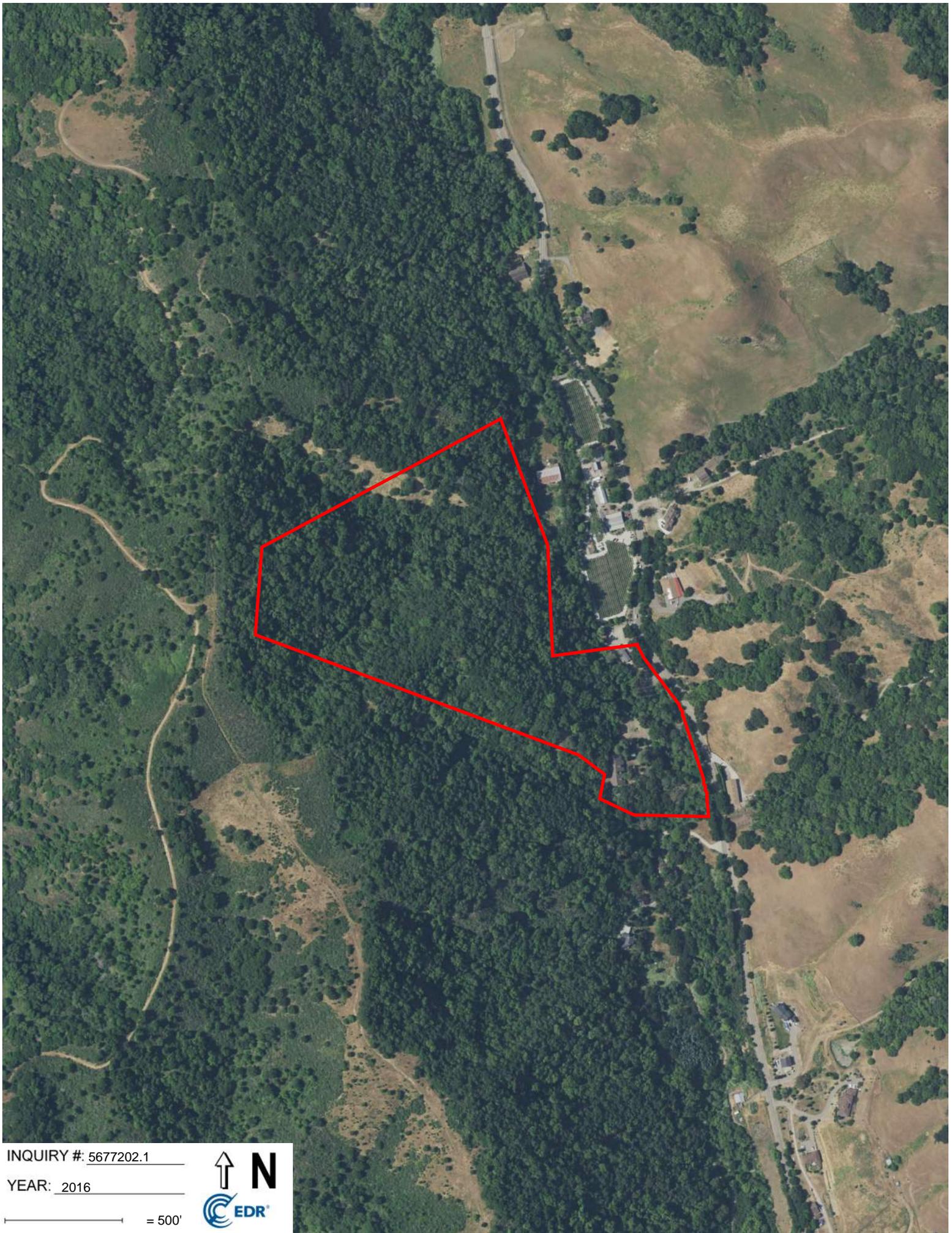
When delivered electronically by EDR, the aerial photo images included with this report are for ONE TIME USE ONLY. Further reproduction of these aerial photo images is prohibited without permission from EDR. For more information contact your EDR Account Executive.

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INQUIRY #: 5677202.1

YEAR: 2016

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INQUIRY #: 5677202.1

YEAR: 2012

— = 500'





INQUIRY #: 5677202.1

YEAR: 2009

— = 500'



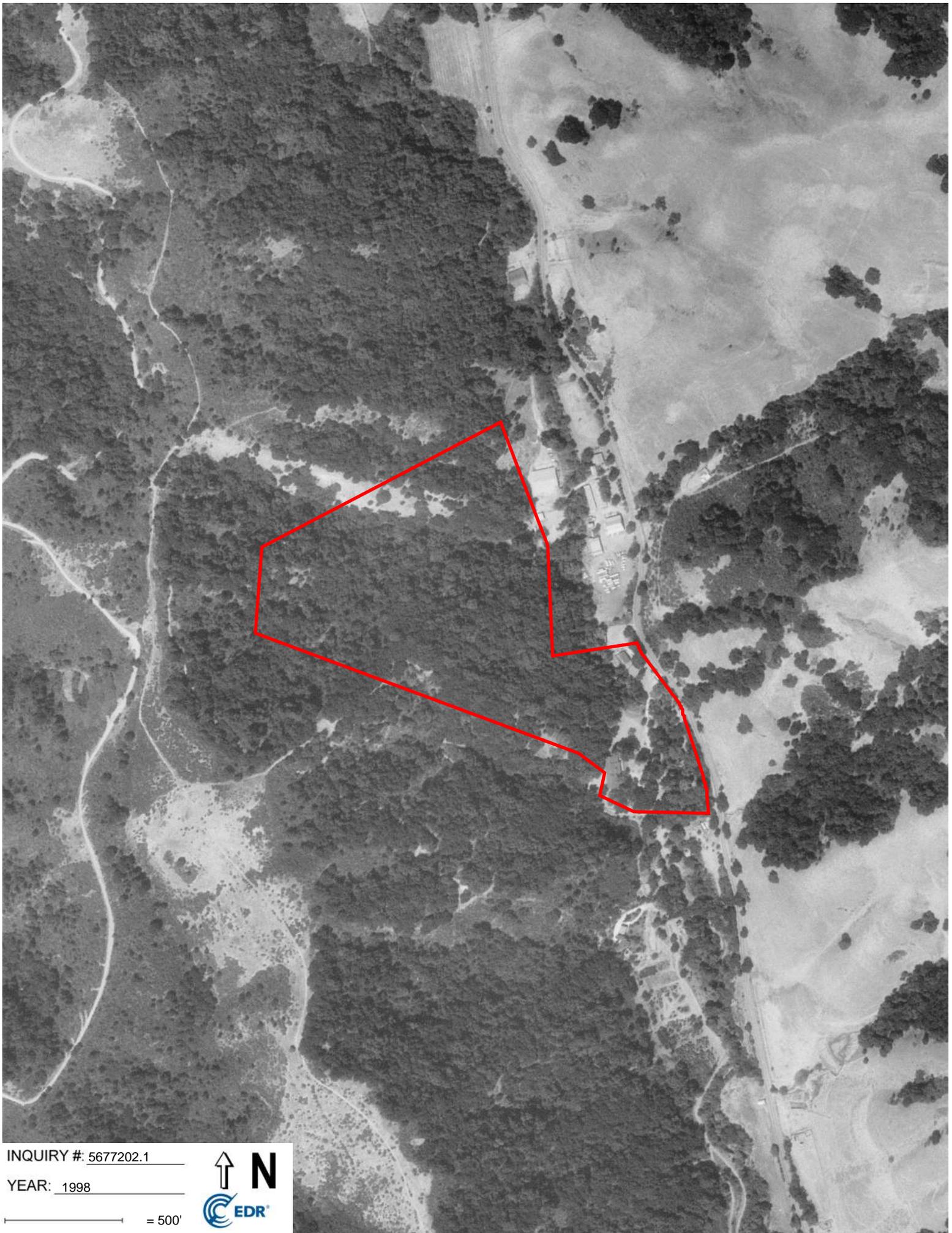


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YEAR: 2006

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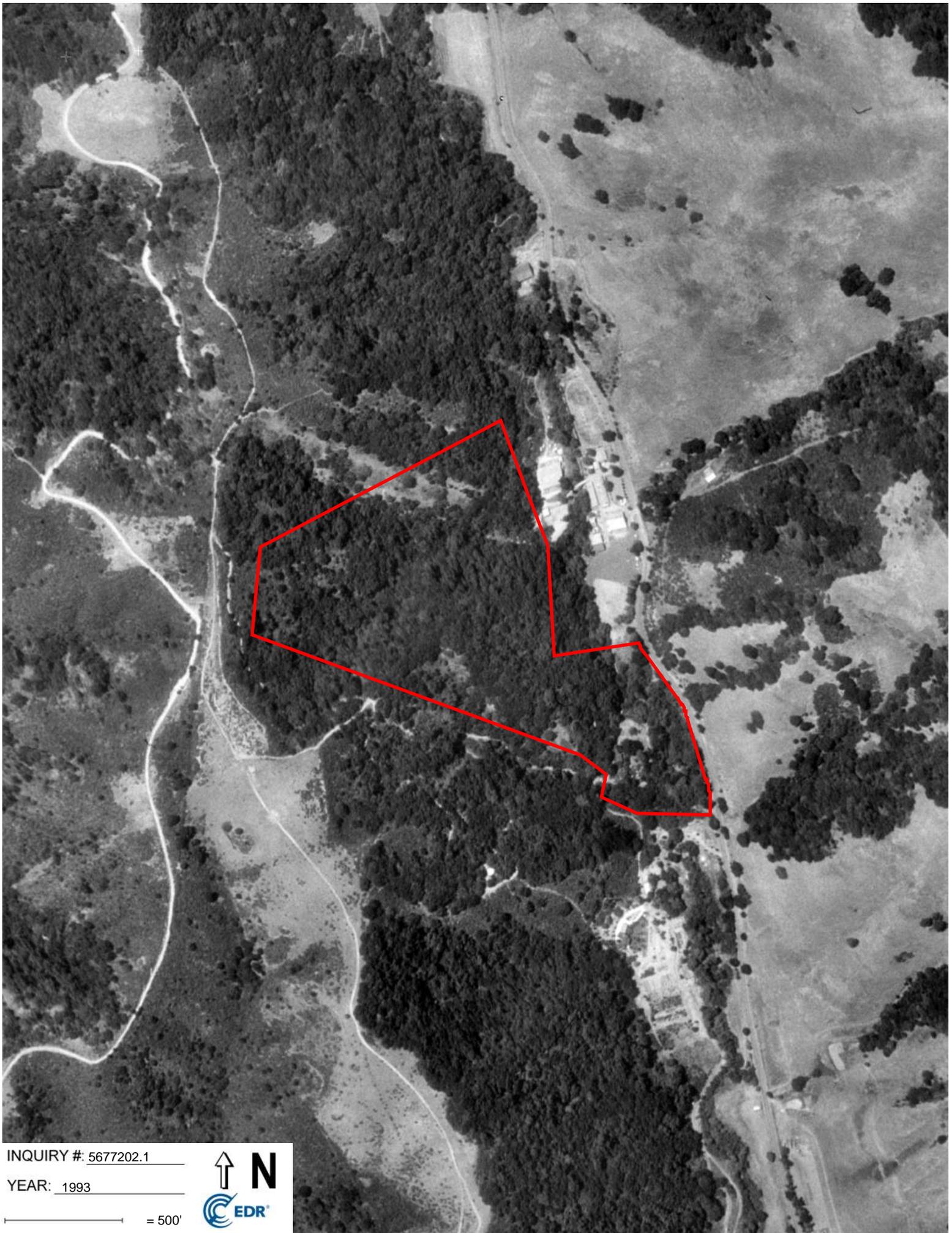


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INQUIRY #: 5677202.1

YEAR: 1993

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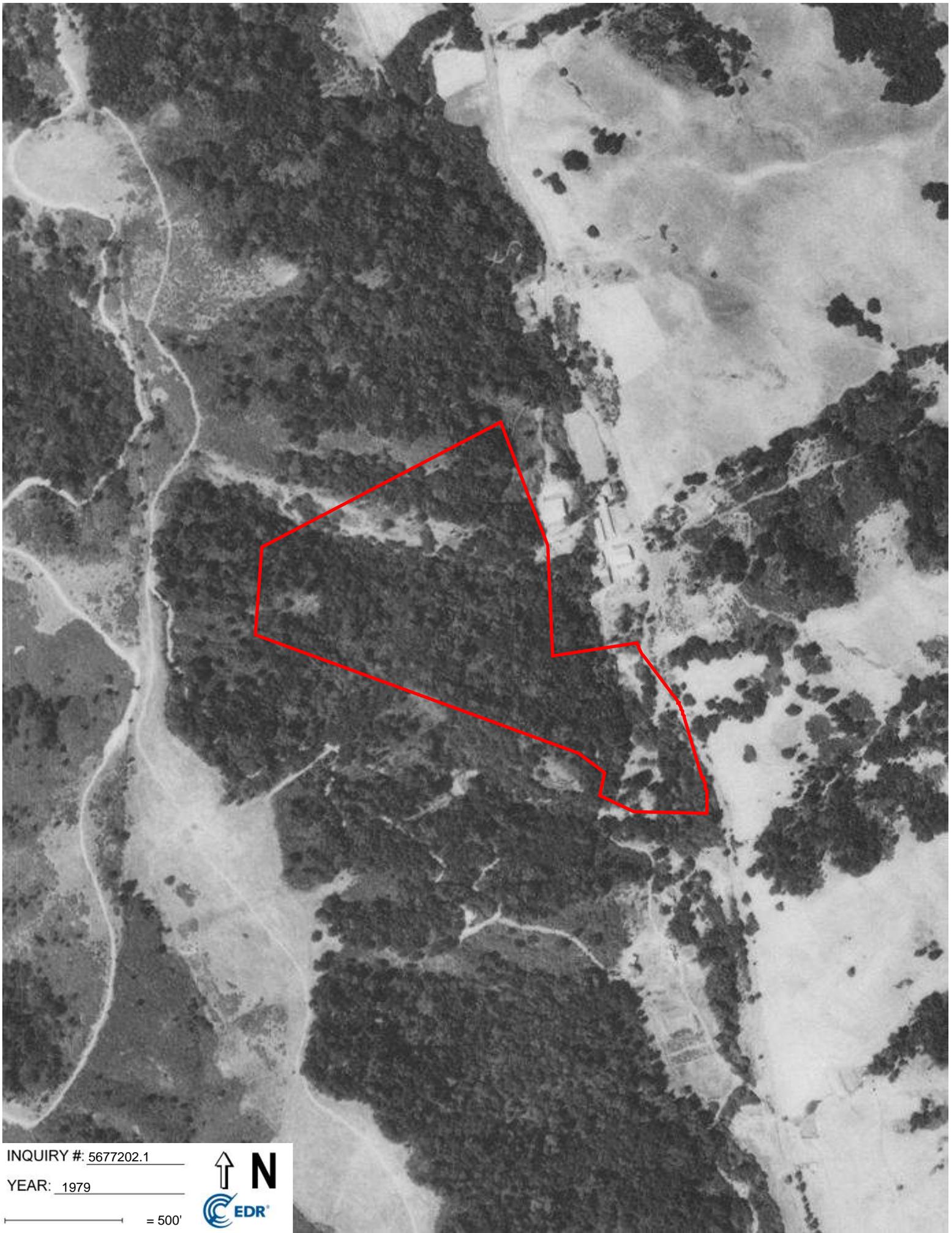


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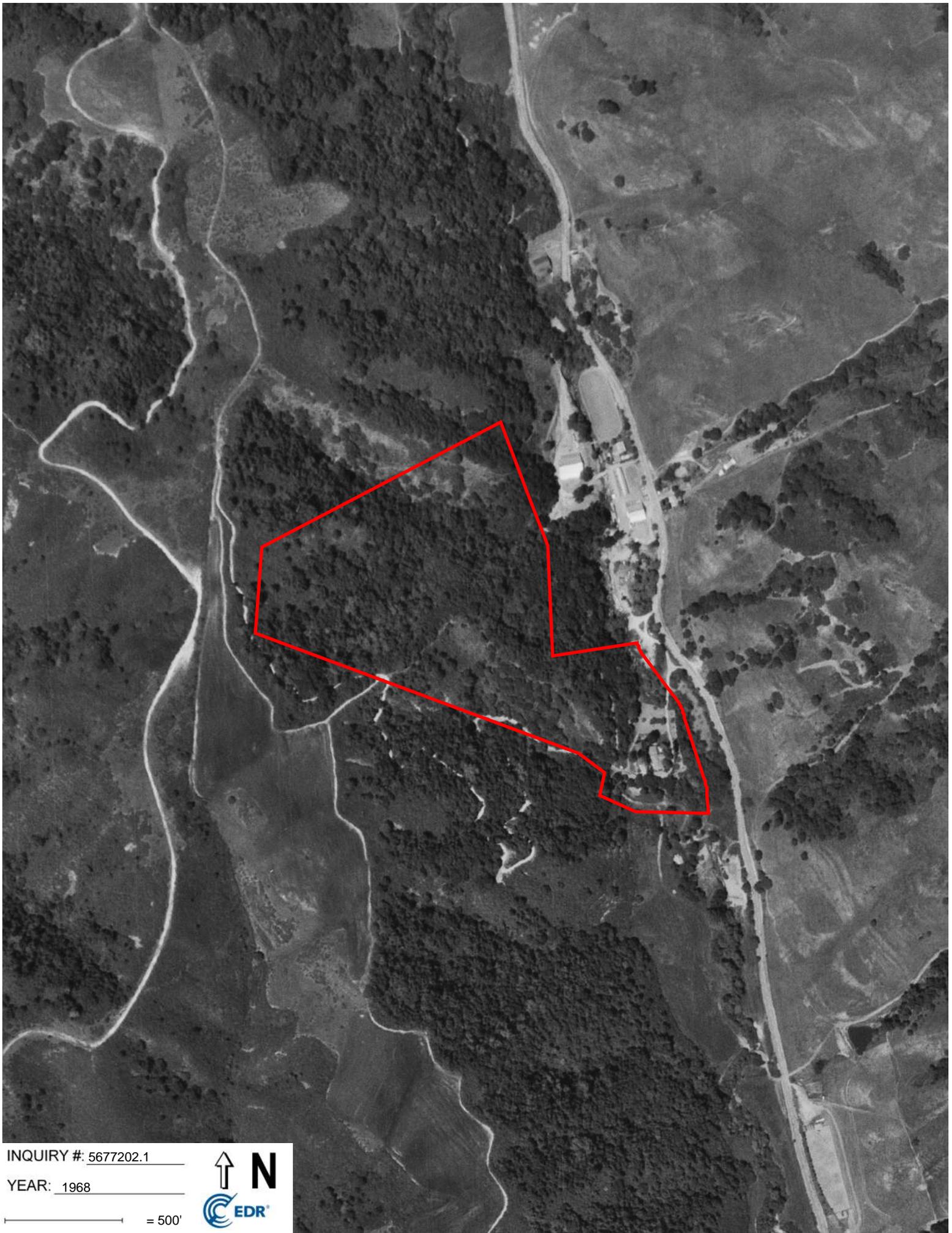


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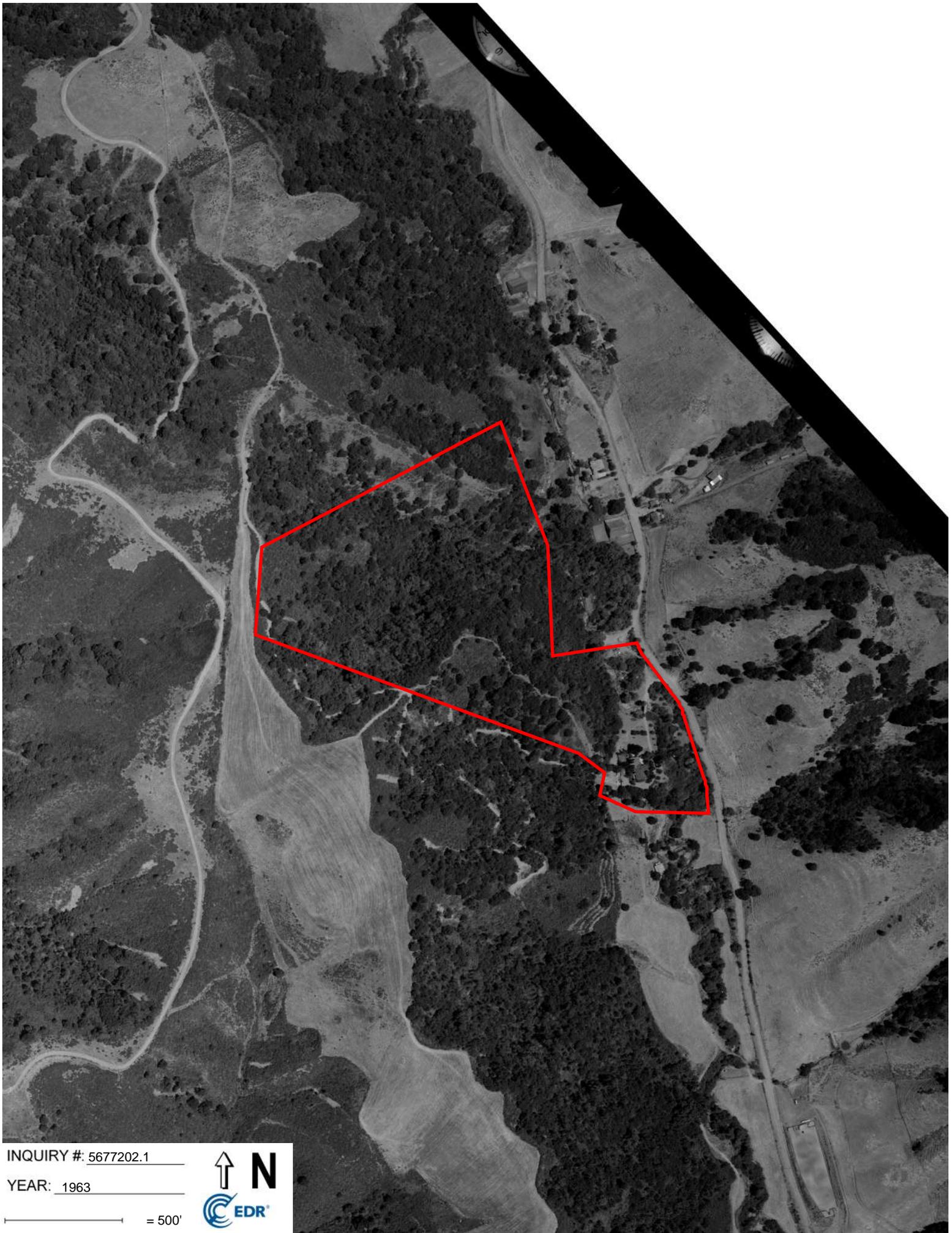


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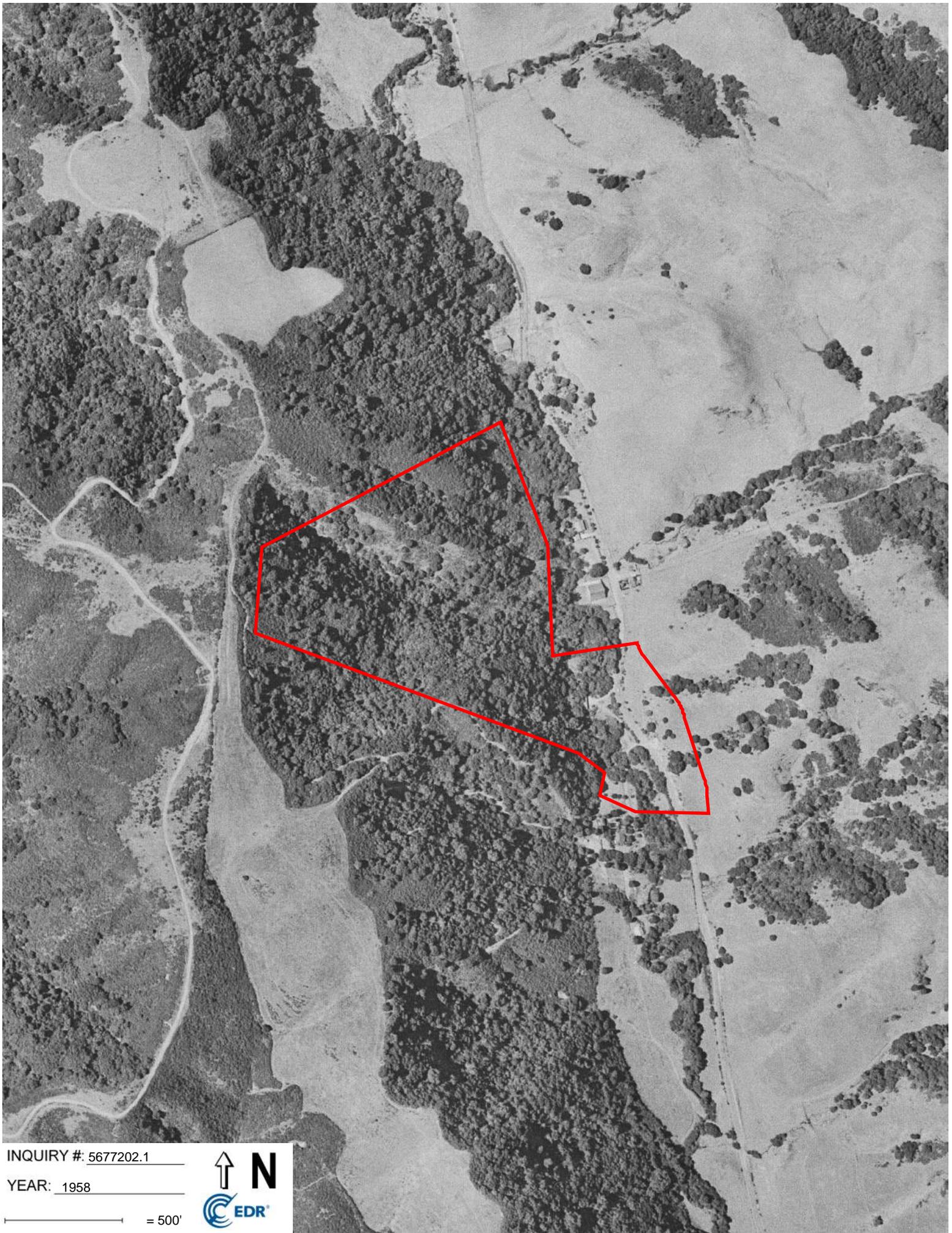


INQUIRY # 5677202.1

YEAR: 1963

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INQUIRY #: 5677202.1

YEAR: 1958

— = 500'



INQUIRY #: 5677202.1

YEAR: 1950

— = 500'



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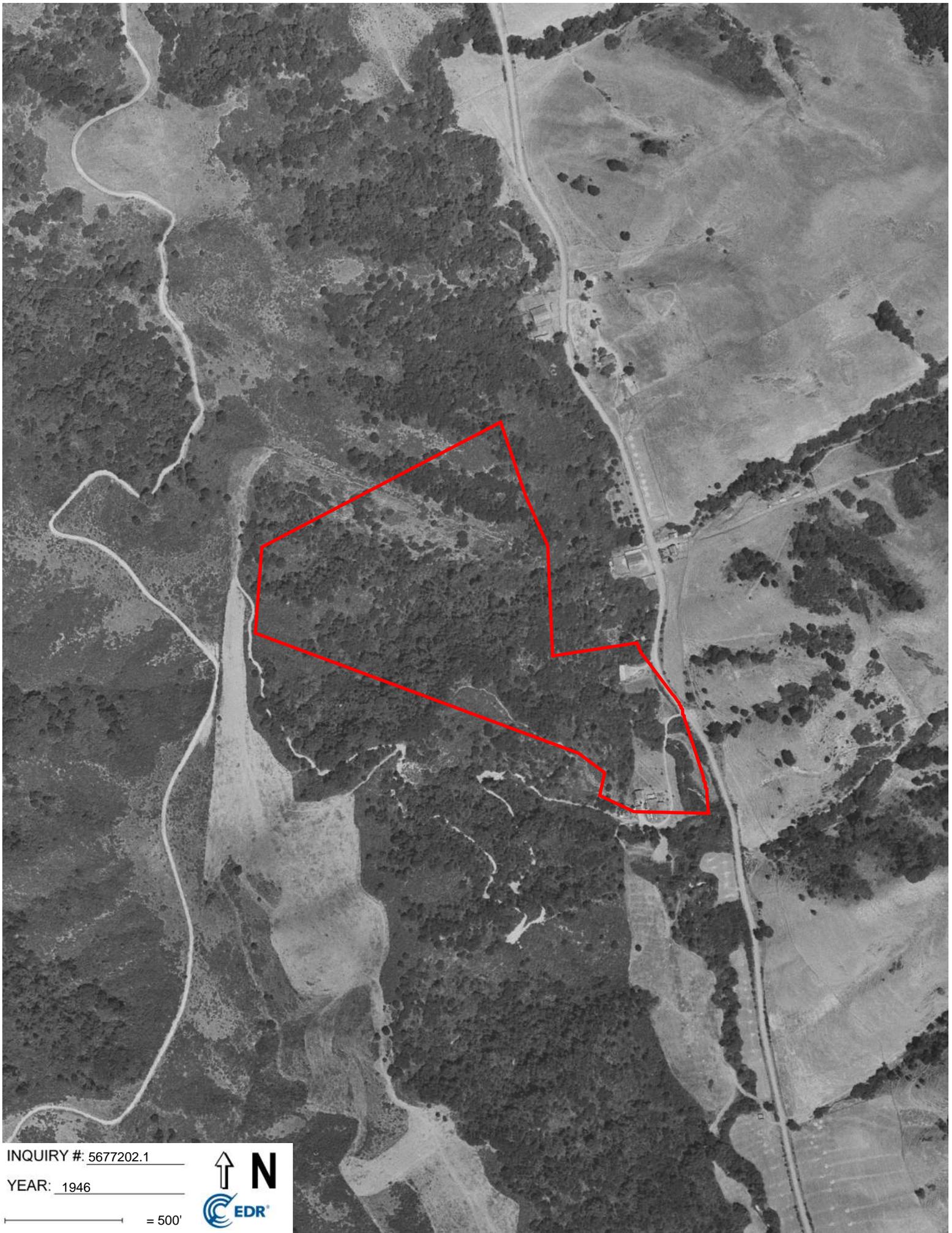
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YEAR: 1949

— = 500'



Subject boundary not shown because it exceeds image extent or image is not georeferenced.



INQUIRY #: 5677202.1

YEAR: 1946

— = 500'



APPENDIX C:

Exploratory Trench Logs

APPENDIX D:

Soil Laboratory Test Results

DSA LEA No. 284

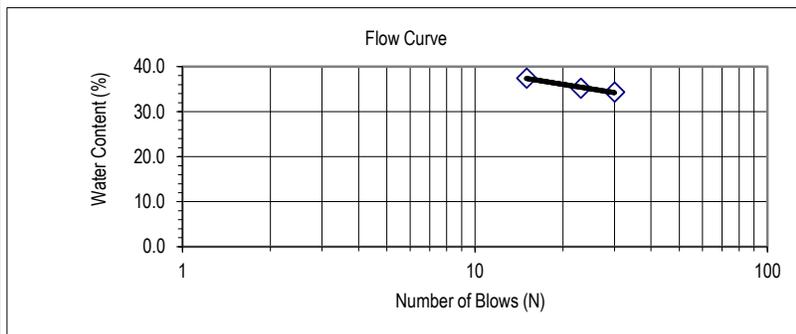
DSA File No. 0
 DSA App No. 0

Project No.	70852.00.001	Project Name	Mosaic	Date:	07/11/19	
Sample No.	T19-1	Boring/Trench	B-1	Depth, (ft.):	1-3'	
Description:	(CL) sandy lean clay, dark brown (10YR 3/3)				Tested By:	LGH
Sample Location:	0				Checked By:	SDC
					Lab. No.	C19-089

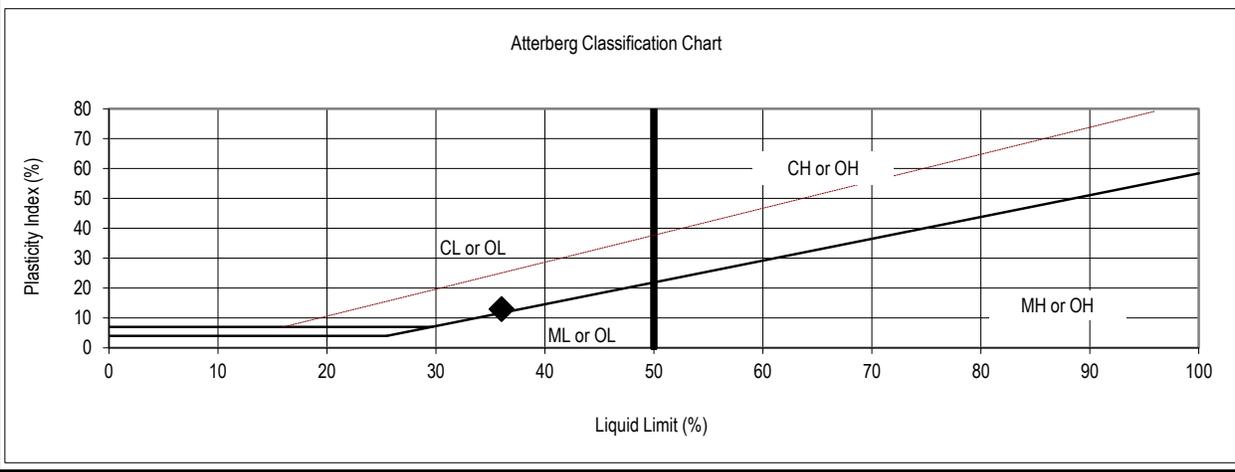
Estimated % of Sample Retained on No. 40 Sieve: _____ Sample Air Dried: yes
 Test Method A or B: A

Sample No.:	LIQUID LIMIT:					PLASTIC LIMIT:		
	1	2	3	4	5	1	2	3
Pan ID:	M-1	J-1	L			H	N-1	
Wt. Pan (gr)	60.89	63.31	61.22			61.61	64.90	
Wt. Wet Soil + Pan	72.78	76.14	70.47			70.58	73.30	
Wt. Dry Soil + Pan (69.74	72.80	67.95			68.89	71.71	
Wt. Water (gr)	3.04	3.34	2.52			1.69	1.59	
Wt. Dry Soil (gr)	8.85	9.49	6.73			7.28	6.81	
Water Content (%)	34.4	35.2	37.4			23.2	23.3	
Number of Blows, N	30	23	15					

LIQUID LIMIT = 36 PLASTIC LIMIT = 23



Plasticity Index = 13
 Group Symbol = CL





PARTICLE SIZE DISTRIBUTION TEST WORK SHEET

ASTM D422, C136

DSA LEA No. 284

DSA File No. 0

DSA App No. 0

Sieve Only Analysis Worksheet			
Project No.	70852.00.001	Project Name:	Mosaic
Sample No.	T19-1	Boring/Trench:	B-1
		Depth, (ft.):	1-3'
Description:	(CL) sandy lean clay, dark brown (10YR 3/3)		Date:
Sample Location:	0		Tested By:
			Checked By:
			Lab. No.

Moisture Content Data:		Total Material Sample Data:	
Pan ID	0	Pan ID	0
Pan Weight	0.00 (gm)	Pan Weight	0.00 (gm)
Wet Soil + Pan	0.00 (gm)	Wet Soil + Pan Wt.	7,534.20 (gm)
Dry Soil + Pan	0.00 (gm)	Total Wet Weight	7,534.20 (gm)
Water Weight	0.00 (gm)	Total Dry Weight	7,534.20 (gm)
Dry Soil Weight	0.00 (gm)	Total Dry Wt. >#4 Sieve	265.20 (gm)
Moisture Content	0.0 (%)	Total Dry Wt. <#4 Sieve	7,269.00 (gm)
		Total Dry Wt. <#200 Sieve	4,886.95 (gm)
		Total Percent <#200 Sieve	64.86 (%)

GRAVEL PORTION SIEVE ANALYSIS (Portion Retained On > #4 Sieve)							
Sieve Size	Particle Diameter		Wet Weight Retained On Sieve (gm)	Dry Weight			
	Inches (in.)	Millimeter (mm)		Retained On Sieve (gm)	Accum. On Sieve (gm)	Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.40	0.00	0.00	0.00	7,534.20	100.0
3 Inch	3.0000	76.20	0.00	0.00	0.00	7,534.20	100.0
2 Inch	2.0000	50.80	0.00	0.00	0.00	7,534.20	100.0
1.5 Inch	1.5000	38.10	0.00	0.00	0.00	7,534.20	100.0
1.0 Inch	1.0000	25.40	41.70	41.70	41.70	7,492.50	99.4
3/4 Inch	0.7500	19.05	25.80	25.80	67.50	7,466.70	99.1
1/2 Inch	0.5000	12.70	36.70	36.70	104.20	7,430.00	98.6
3/8 Inch	0.3750	9.53	25.50	25.50	129.70	7,404.50	98.3
#4	0.1875	4.75	135.50	135.50	265.20	7,269.00	96.5
PAN			7,269.00	7,269.00			

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

Representative Sample Data:			
Pan ID	0	#200 Wash Data:	
Pan Weight	0.00 (gm)	Portion >#200 Sieve:	124.10 (gm)
Wet Soil + Pan	378.70 (gm)	Portion <#200 Sieve:	254.60 (gm)
Wet Soil	378.70 (gm)	Percent <#200 Sieve	67.23 (%)
Dry Soil	378.70 (gm)	Total Wt. <#200 Sieve	4886.95 (gm)

Sieve Size	Particle Diameter		Dry Weight Rep. Sample		Total Sample Weight Retained (gm)	Accum. Grand Total On Sieve (gm)	Total Percent Passing (%)
	Inches (in.)	Millimeter (mm)	Retained On Sieve (gm)	Percent Retained (%)			
#10	0.079	2.000	10.3	2.72	197.70	462.90	93.9
#20	0.033	0.850	11.30	2.98	216.90	679.80	91.0
#40	0.017	0.425	12.40	3.27	238.01	917.82	87.8
#60	0.010	0.250	17.50	4.62	335.91	1,253.72	83.4
#100	0.006	0.150	25.50	6.73	489.46	1,743.19	76.9
#200	0.003	0.075	47.10	12.44	904.07	2,647.25	64.9
PAN			Discard				

DSA LEA No. 284

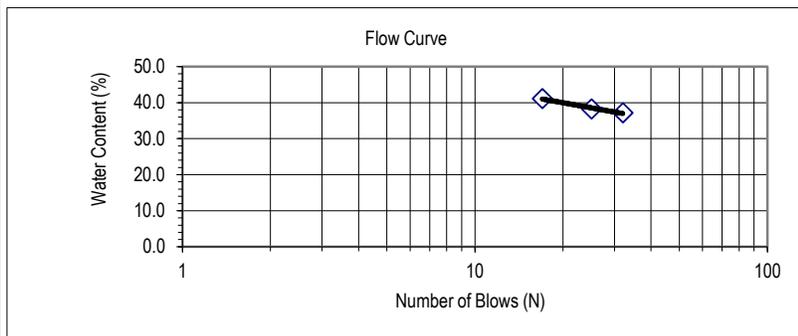
DSA File No. 0
 DSA App No. 0

Project No.	70852.00.001	Project Name	Mosaic	Date:	07/11/19
Sample No.	T19-2	Boring/Trench	B-1	Depth, (ft.):	2-3'
Description:	(CL) Sandy lean clay, dark brown (10YR 3/3)			Tested By:	LGH
Sample Location:	0			Checked By:	SDC
				Lab. No.	C19-089

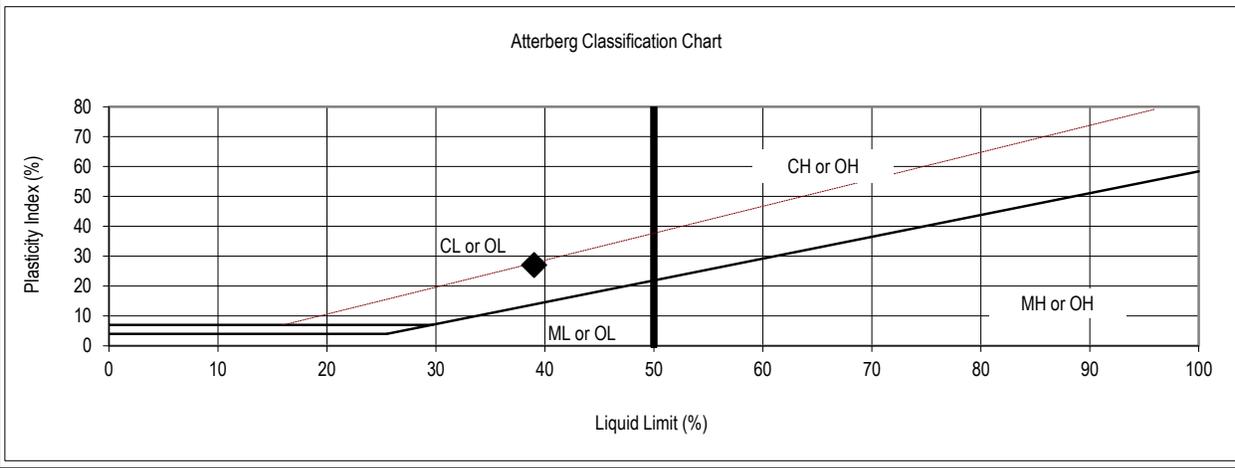
Estimated % of Sample Retained on No. 40 Sieve: _____ Sample Air Dried: yes
 Test Method A or B: A

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	A	C	Z			Y	V	
Wt. Pan (gr)	38.46	38.47	37.43			37.10	37.34	
Wt. Wet Soil + Pan	48.39	48.44	47.83			45.07	45.32	
Wt. Dry Soil + Pan (g)	45.70	45.68	44.80			44.19	44.46	
Wt. Water (gr)	2.69	2.76	3.03			0.88	0.86	
Wt. Dry Soil (gr)	7.24	7.21	7.37			7.09	7.12	
Water Content (%)	37.2	38.3	41.1			12.4	12.1	
Number of Blows, N	32	25	17					

LIQUID LIMIT = 39 PLASTIC LIMIT = 12



Plasticity Index = 27
 Group Symbol = CL



DSA LEA No. 284

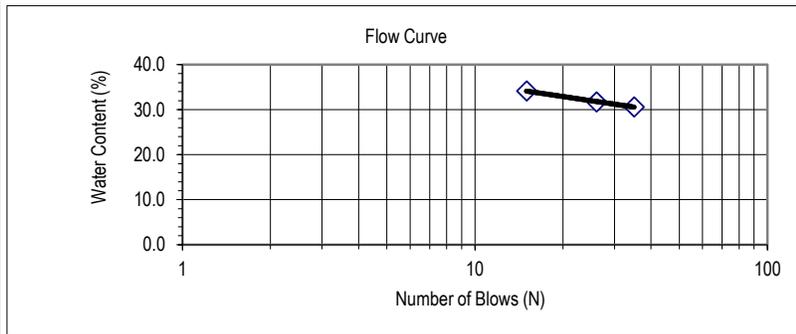
DSA File No. 0
 DSA App No. 0

Project No.	70852.00.001	Project Name	Mosaic	Date:	07/11/19	
Sample No.	T19-4	Boring/Trench	B-1	Depth, (ft.):	1-2'	
Description:	(CL) lean clay with sand, dark brown (10YR 3/3)				Tested By:	LGH
Sample Location:	0				Checked By:	SDC
					Lab. No.	C19-089

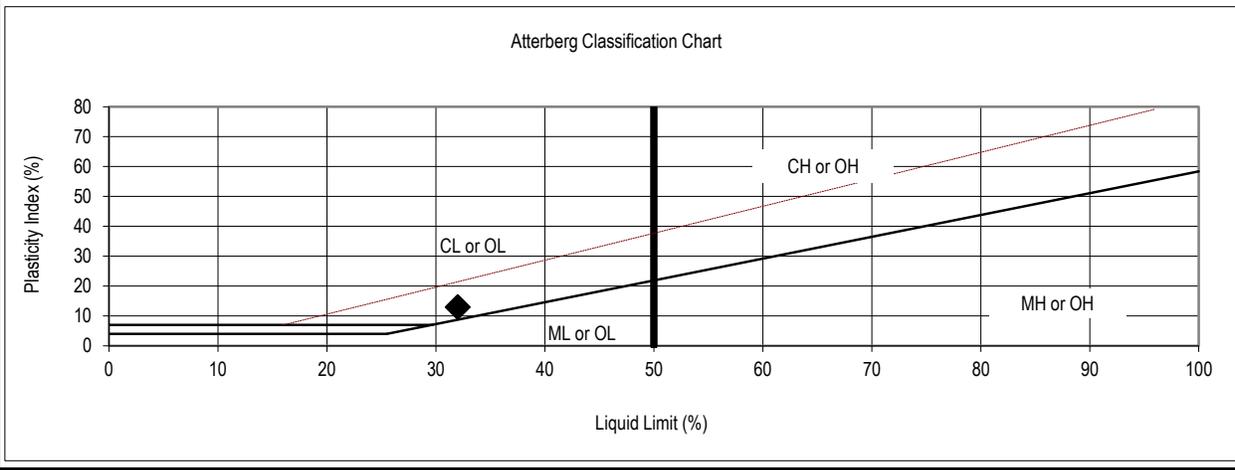
Estimated % of Sample Retained on No. 40 Sieve: _____ Sample Air Dried: yes
 Test Method A or B: A

LIQUID LIMIT:					PLASTIC LIMIT:			
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	A	V	Z			C	Y	
Wt. Pan (gr)	38.47	37.34	37.43			38.47	37.11	
Wt. Wet Soil + Pan	46.96	45.85	46.86			47.34	45.83	
Wt. Dry Soil + Pan (44.97	43.80	44.46			45.90	44.41	
Wt. Water (gr)	1.99	2.05	2.40			1.44	1.42	
Wt. Dry Soil (gr)	6.50	6.46	7.03			7.43	7.30	
Water Content (%)	30.6	31.7	34.1			19.4	19.5	
Number of Blows, N	35	26	15					

LIQUID LIMIT = 32 PLASTIC LIMIT = 19



Plasticity Index = 13
 Group Symbol = CL



DSA LEA No. 284

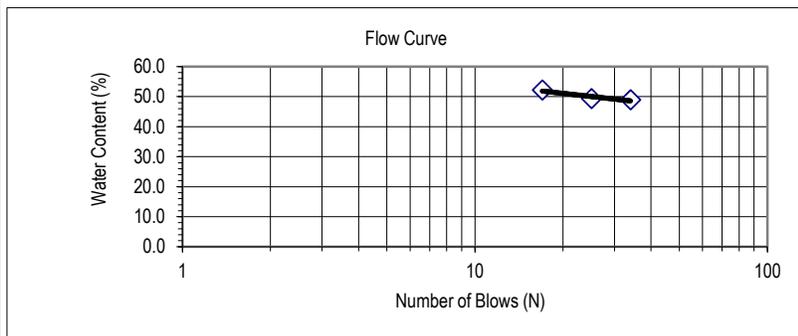
DSA File No. 0
 DSA App No. 0

Project No.	70852.00.001	Project Name	Mosaic	Date:	07/11/19	
Sample No.	T19-5	Boring/Trench	B-1	Depth, (ft.):	3'	
Description:	(CH) Fat clay with sand, dark brown (10YR 3/3)				Tested By:	LGH
Sample Location:	0				Checked By:	SDC
					Lab. No.	C19-089

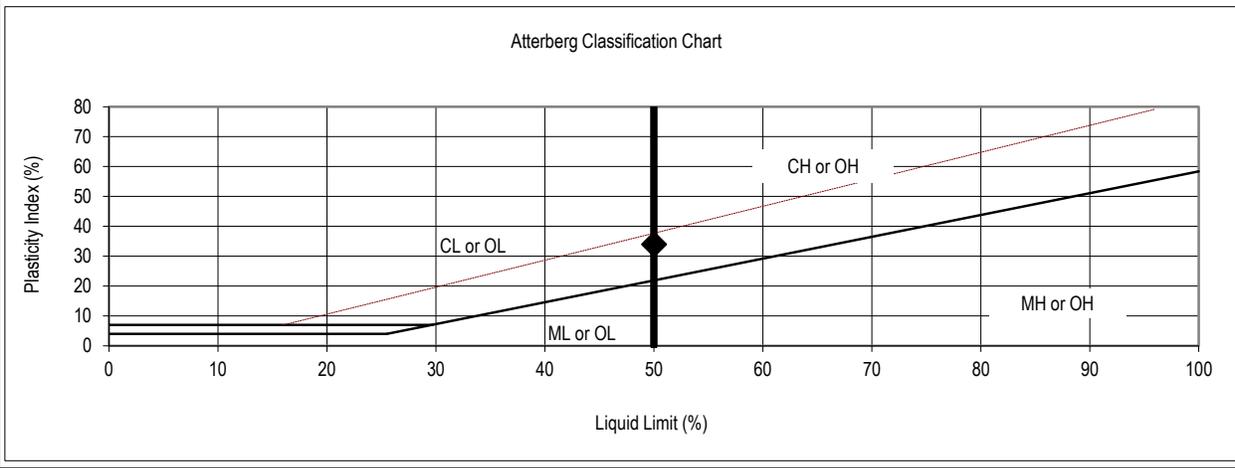
Estimated % of Sample Retained on No. 40 Sieve: _____ Sample Air Dried: yes
 Test Method A or B: A

Sample No.:	LIQUID LIMIT:					PLASTIC LIMIT:		
	1	2	3	4	5	1	2	3
Pan ID:	B	D	W			X	E	
Wt. Pan (gr)	38.97	38.28	37.77			38.20	36.45	
Wt. Wet Soil + Pan	47.73	46.42	46.61			46.08	44.90	
Wt. Dry Soil + Pan (g)	44.85	43.73	43.58			45.02	43.75	
Wt. Water (gr)	2.88	2.69	3.03			1.06	1.15	
Wt. Dry Soil (gr)	5.88	5.45	5.81			6.82	7.30	
Water Content (%)	49.0	49.4	52.2			15.5	15.8	
Number of Blows, N	34	25	17					

LIQUID LIMIT = 50 PLASTIC LIMIT = 16



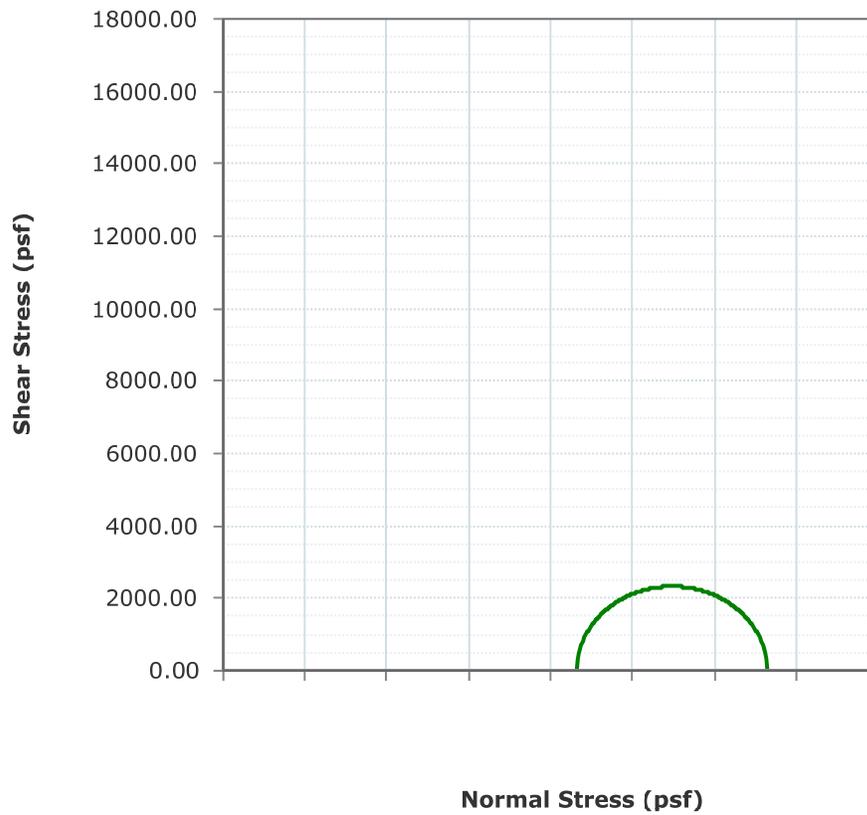
Plasticity Index = 34
 Group Symbol = CH



Unconsolidated Undrained Test

ASTM D2850

Mohr Circles



■ Specimen 1

Strength Intercept = NA

Strength Intercept = NA

Normal Stress (psf)

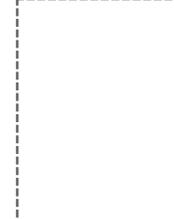
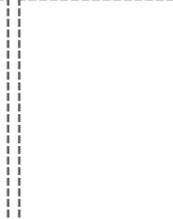
Project:	MOSIAC
Project Number:	70852
Sampling Date:	
Sample Number:	L-1
Sample Depth:	6.5
Location:	T19-5
Client Name:	
Remarks:	

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)	60.0							
Height (in)	5.194							
Diameter (in)	1.900							
Water Content (%)	23.2							
Wet Density (Units)	121.5							
Dry Density (pcf)	98.6							
Degree of Saturation (%)	87.3							
Void Ratio	0.722							
Height To Diameter Ratio	2.734							
Test Data	1	2	3	4	5	6	7	8
Comp. Strength at Failure (psf)	4643.17							
σ_1 at Failure (psf)	13283.16							
σ_3 at Failure (psf)	8640.00							
Rate of Strain (in/min)	0.064925							
Axial Strain at Failure (%)	15.47							
After Test	1	2	3	4	5	6	7	8
Final Water Content (%)	24.9							

Project:	MOSIAC
Project Number:	70852
Sampling Date:	
Sample Number:	L-1
Sample Depth:	6.5
Location:	T19-5
Client Name:	
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
							

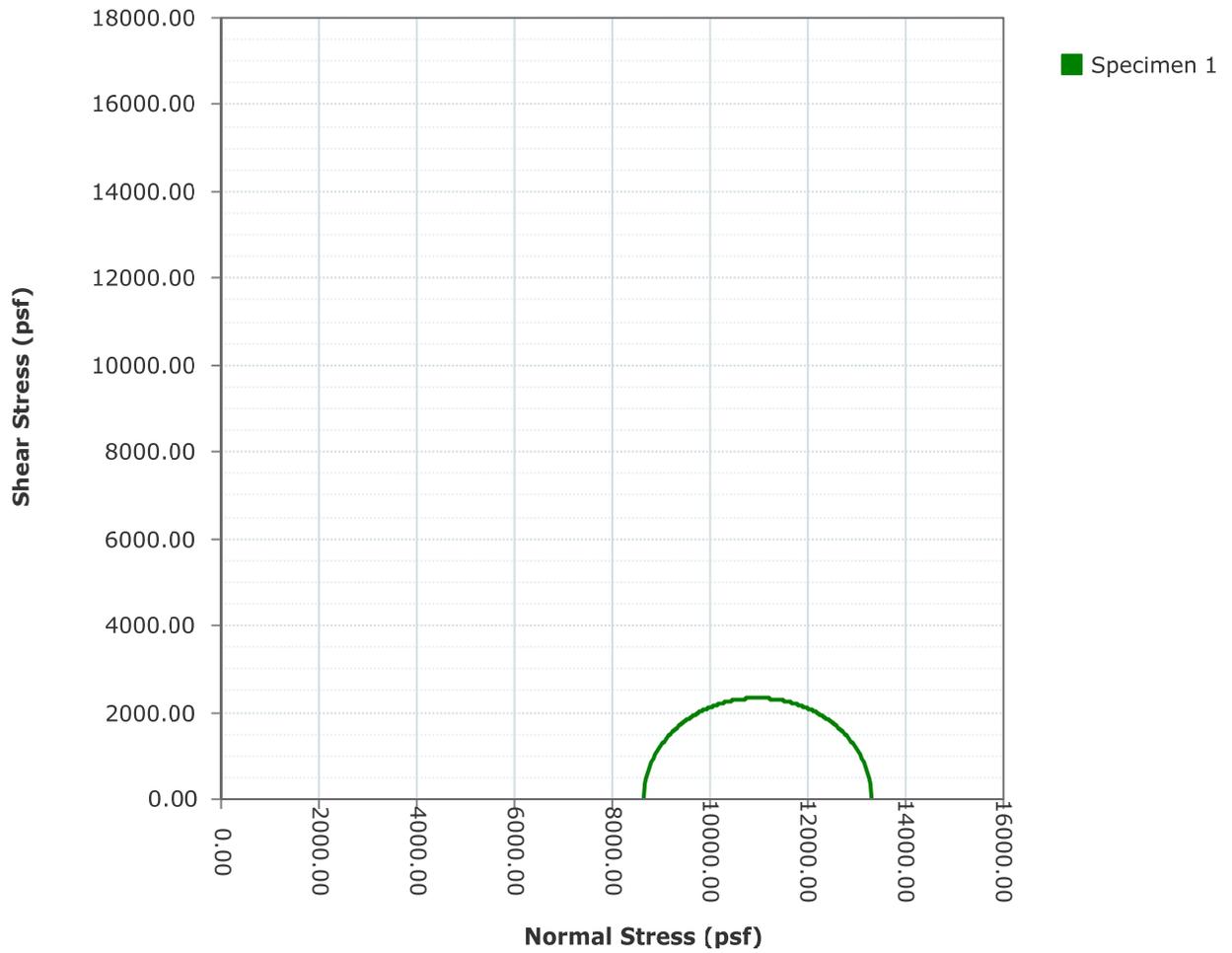
Unconsolidated Undrained Test

ASTM D2850

Specimen 1	
Test Description:	D2850 unconsolidated Undrained
Other Associated Tests:	
Device Details:	Master Loader
Test Specification:	
Test Time:	8/7/2019
Technician:	DJP/LGH
Specimen Code:	
Sampling Method:	Undisturbed
Specimen Lab #:	T19-5
Specimen Description:	
Specific Gravity:	2.720
Plastic Limit:	0
Liquid Limit:	0
Height (in):	5.194
Diameter (in):	1.900
Area (in ²):	2.835
Volume (in ³):	14.73
Large Particle:	
Moisture Material:	Specimen
Moist Weight (g):	469.5
Test Remarks:	

Mohr Circles (Total Stress) Graph

ASTM D2850

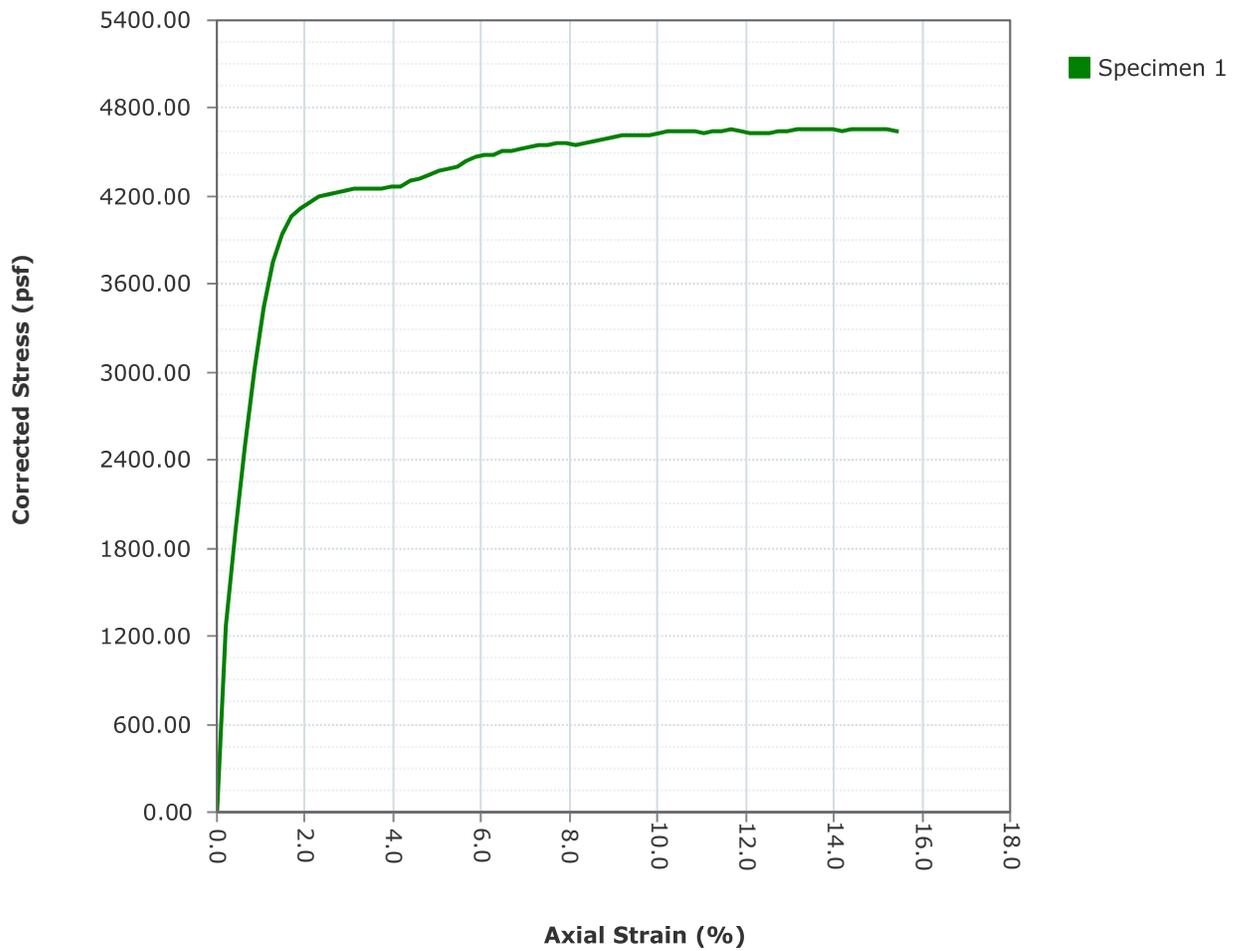


Tangent Results	
Strength Intercept (psi)	NA
Friction Angle (°)	NA

Sample: L-1 from trench T19-5, at 6.5 feet bgs

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 (in ²)	Axial Strain (%)	Stress (psf)	Corrected Compressive		σ_1 (psf)	σ_3 (psf)	$\frac{\sigma_1}{\sigma_3}$	p (psf)	q (psf)
									Stress (psf)	Stress (psf)					
0	00:00:00	1.7	0.0001	0.0	0.000	2.835	0.0	0.00	0.00	8,640.00	8,640.00	1.000	8,640.00	0.00	
1	00:00:10	26.7	0.0105	25.0	0.010	2.841	0.2	1,267.98	1,265.44	9,905.44	8,640.00	1.146	9,272.72	632.72	
2	00:00:20	39.7	0.0218	37.9	0.022	2.847	0.4	1,927.13	1,919.07	10,559.07	8,640.00	1.222	9,599.53	959.53	
3	00:00:30	51.1	0.0330	49.4	0.033	2.853	0.6	2,507.77	2,491.86	11,131.86	8,640.00	1.288	9,885.93	1,245.93	
4	00:00:40	61.6	0.0440	59.9	0.044	2.859	0.8	3,040.14	3,014.42	11,654.42	8,640.00	1.349	10,147.21	1,507.21	
5	00:00:50	70.3	0.0551	68.6	0.055	2.866	1.1	3,484.12	3,447.20	12,087.20	8,640.00	1.399	10,363.60	1,723.60	
6	00:01:00	76.6	0.0660	74.8	0.066	2.872	1.3	3,800.18	3,751.93	12,391.93	8,640.00	1.434	10,515.97	1,875.97	
7	00:01:10	80.6	0.0769	78.8	0.077	2.878	1.5	4,002.72	3,943.50	12,583.50	8,640.00	1.456	10,611.75	1,971.75	
8	00:01:20	83.0	0.0876	81.3	0.088	2.884	1.7	4,127.01	4,057.43	12,697.43	8,640.00	1.470	10,668.71	2,028.71	
9	00:01:30	84.4	0.0985	82.7	0.098	2.890	1.9	4,199.50	4,119.93	12,759.93	8,640.00	1.477	10,699.97	2,059.97	
10	00:01:40	85.4	0.1093	83.7	0.109	2.896	2.1	4,250.41	4,161.01	12,801.01	8,640.00	1.482	10,720.50	2,080.50	
11	00:01:50	86.2	0.1199	84.5	0.120	2.902	2.3	4,290.82	4,191.80	12,831.80	8,640.00	1.485	10,735.90	2,095.90	
12	00:02:00	86.9	0.1304	85.2	0.130	2.908	2.5	4,326.37	4,217.81	12,857.81	8,640.00	1.488	10,748.91	2,108.91	
13	00:02:10	87.4	0.1412	85.7	0.141	2.914	2.7	4,350.23	4,232.05	12,872.05	8,640.00	1.490	10,756.03	2,116.03	
14	00:02:20	87.7	0.1517	86.0	0.152	2.921	2.9	4,366.13	4,238.63	12,878.63	8,640.00	1.491	10,759.31	2,119.31	
15	00:02:30	88.0	0.1622	86.3	0.162	2.927	3.1	4,383.28	4,246.49	12,886.49	8,640.00	1.491	10,763.25	2,123.25	
16	00:02:40	88.4	0.1730	86.7	0.173	2.933	3.3	4,402.01	4,255.47	12,895.47	8,640.00	1.493	10,767.74	2,127.74	
17	00:02:50	88.6	0.1838	86.9	0.184	2.939	3.5	4,411.35	4,255.33	12,895.33	8,640.00	1.493	10,767.67	2,127.67	
18	00:03:00	88.8	0.1945	87.1	0.194	2.946	3.7	4,423.29	4,257.75	12,897.75	8,640.00	1.493	10,768.88	2,128.88	
19	00:03:10	89.2	0.2052	87.4	0.205	2.952	3.9	4,439.53	4,264.21	12,904.21	8,640.00	1.494	10,772.11	2,132.11	
20	00:03:20	89.5	0.2164	87.8	0.216	2.959	4.2	4,458.11	4,272.44	12,912.44	8,640.00	1.494	10,776.22	2,136.22	
21	00:03:30	90.4	0.2278	88.6	0.228	2.965	4.4	4,502.35	4,304.98	12,944.98	8,640.00	1.498	10,792.49	2,152.49	
22	00:03:40	91.0	0.2390	89.2	0.239	2.972	4.6	4,531.58	4,323.15	12,963.15	8,640.00	1.500	10,801.57	2,161.57	

Unconsolidated Undrained Test - Tabulated Data - Specimen 1

ASTM D2850

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Disp. (in)	Corrected			Axial Strain (%)	Stress (psf)	Corrected Compressive Stress (psf)		σ_1 (psf)	σ_3 (psf)	$\frac{\sigma_1}{\sigma_3}$	p (psf)	q (psf)
				Load (Lbf)	Disp. (in)	Area (in ²)			σ_1 (psf)	σ_3 (psf)					
23	00:03:50	91.7	0.2502	89.9	0.250	2.979	4.8	4,567.83	4,347.87	12,987.87	8,640.00	1.503	10,813.94	2,173.94	
24	00:04:00	92.3	0.2614	90.5	0.261	2.986	5.0	4,598.72	4,367.31	13,007.30	8,640.00	1.505	10,823.65	2,183.65	
25	00:04:10	92.9	0.2724	91.1	0.272	2.992	5.2	4,629.04	4,386.37	13,026.37	8,640.00	1.508	10,833.18	2,193.18	
26	00:04:20	93.5	0.2831	91.8	0.283	2.999	5.5	4,661.33	4,407.29	13,047.29	8,640.00	1.510	10,843.64	2,203.64	
27	00:04:30	94.4	0.2940	92.7	0.294	3.005	5.7	4,708.19	4,441.77	13,081.76	8,640.00	1.514	10,860.88	2,220.88	
28	00:04:40	95.1	0.3046	93.3	0.305	3.012	5.9	4,740.46	4,462.51	13,102.51	8,640.00	1.516	10,871.26	2,231.26	
29	00:04:50	95.6	0.3153	93.9	0.315	3.018	6.1	4,767.25	4,477.95	13,117.95	8,640.00	1.518	10,878.98	2,238.98	
30	00:05:00	96.0	0.3258	94.3	0.326	3.025	6.3	4,787.40	4,487.13	13,127.13	8,640.00	1.519	10,883.57	2,243.57	
31	00:05:10	96.6	0.3362	94.8	0.336	3.031	6.5	4,815.47	4,503.84	13,143.84	8,640.00	1.521	10,891.92	2,251.92	
32	00:05:20	96.9	0.3468	95.2	0.347	3.038	6.7	4,834.52	4,511.78	13,151.78	8,640.00	1.522	10,895.89	2,255.89	
33	00:05:30	97.4	0.3574	95.6	0.357	3.045	6.9	4,856.72	4,522.62	13,162.62	8,640.00	1.523	10,901.31	2,261.31	
34	00:05:40	97.8	0.3681	96.0	0.368	3.051	7.1	4,877.32	4,531.77	13,171.77	8,640.00	1.525	10,905.88	2,265.88	
35	00:05:50	98.3	0.3791	96.6	0.379	3.058	7.3	4,903.98	4,546.14	13,186.14	8,640.00	1.526	10,913.07	2,273.07	
36	00:06:00	98.7	0.3897	96.9	0.390	3.065	7.5	4,922.93	4,553.59	13,193.59	8,640.00	1.527	10,916.80	2,276.80	
37	00:06:10	99.1	0.4005	97.3	0.400	3.072	7.7	4,944.03	4,562.90	13,202.90	8,640.00	1.528	10,921.45	2,281.45	
38	00:06:20	99.2	0.4115	97.4	0.411	3.079	7.9	4,949.20	4,557.12	13,197.12	8,640.00	1.527	10,918.56	2,278.56	
39	00:06:30	99.4	0.4228	97.6	0.423	3.086	8.1	4,958.77	4,555.20	13,195.20	8,640.00	1.527	10,917.60	2,277.60	
40	00:06:40	99.7	0.4339	97.9	0.434	3.094	8.4	4,974.68	4,559.18	13,199.18	8,640.00	1.528	10,919.59	2,279.59	
41	00:06:50	100.2	0.4451	98.4	0.445	3.101	8.6	5,000.02	4,571.63	13,211.63	8,640.00	1.529	10,925.82	2,285.82	
42	00:07:00	100.8	0.4565	99.1	0.456	3.108	8.8	5,031.46	4,589.31	13,229.31	8,640.00	1.531	10,934.66	2,294.66	
43	00:07:10	101.5	0.4675	99.7	0.467	3.116	9.0	5,064.26	4,608.54	13,248.54	8,640.00	1.533	10,944.27	2,304.27	
44	00:07:20	101.8	0.4782	100.1	0.478	3.123	9.2	5,084.13	4,616.08	13,256.08	8,640.00	1.534	10,948.04	2,308.04	
45	00:07:30	102.2	0.4892	100.5	0.489	3.130	9.4	5,103.39	4,622.85	13,262.85	8,640.00	1.535	10,951.42	2,311.42	

Unconsolidated Undrained Test - Tabulated Data - Specimen 1

ASTM D2850

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Disp. (in)	Corrected			Axial Strain (%)	Stress (psf)	Corrected Compressive Stress		σ_1 (psf)	σ_3 (psf)	$\frac{\sigma_1}{\sigma_3}$	p (psf)	q (psf)
				Load (Lbf)	Disp. (in)	Area (in ²)			σ_1 (psf)	σ_3 (psf)					
46	00:07:40	102.4	0.5000	100.7	0.500	3.137	9.6	5,114.38	4,622.13	13,262.13	8,640.00	1.535	10,951.06	2,311.06	
47	00:07:50	102.7	0.5104	101.0	0.510	3.144	9.8	5,127.28	4,623.49	13,263.49	8,640.00	1.535	10,951.75	2,311.75	
48	00:08:00	103.0	0.5211	101.3	0.521	3.151	10.0	5,143.91	4,627.91	13,267.91	8,640.00	1.536	10,953.96	2,313.96	
49	00:08:10	103.5	0.5317	101.8	0.532	3.159	10.2	5,168.12	4,639.12	13,279.12	8,640.00	1.537	10,959.56	2,319.56	
50	00:08:20	103.8	0.5423	102.0	0.542	3.166	10.4	5,181.48	4,640.58	13,280.58	8,640.00	1.537	10,960.29	2,320.29	
51	00:08:30	104.0	0.5528	102.3	0.553	3.173	10.6	5,193.62	4,640.92	13,280.92	8,640.00	1.537	10,960.46	2,320.46	
52	00:08:40	104.3	0.5632	102.5	0.563	3.180	10.8	5,206.39	4,641.91	13,281.91	8,640.00	1.537	10,960.96	2,320.96	
53	00:08:50	104.4	0.5739	102.6	0.574	3.187	11.0	5,212.73	4,636.83	13,276.83	8,640.00	1.537	10,958.41	2,318.41	
54	00:09:00	104.9	0.5846	103.2	0.585	3.195	11.3	5,239.91	4,650.19	13,290.18	8,640.00	1.538	10,965.09	2,325.09	
55	00:09:10	105.1	0.5953	103.4	0.595	3.202	11.5	5,249.76	4,648.16	13,288.16	8,640.00	1.538	10,964.08	2,324.08	
56	00:09:20	105.4	0.6061	103.7	0.606	3.210	11.7	5,265.20	4,650.87	13,290.87	8,640.00	1.538	10,965.43	2,325.44	
57	00:09:30	105.4	0.6174	103.7	0.617	3.218	11.9	5,265.62	4,639.81	13,279.81	8,640.00	1.537	10,959.90	2,319.90	
58	00:09:40	105.6	0.6284	103.8	0.628	3.226	12.1	5,272.94	4,635.04	13,275.04	8,640.00	1.536	10,957.52	2,317.52	
59	00:09:50	105.8	0.6396	104.0	0.640	3.233	12.3	5,284.10	4,633.47	13,273.47	8,640.00	1.536	10,956.73	2,316.73	
60	00:10:00	106.0	0.6509	104.3	0.651	3.241	12.5	5,296.90	4,633.20	13,273.20	8,640.00	1.536	10,956.60	2,316.60	
61	00:10:10	106.4	0.6619	104.7	0.662	3.249	12.7	5,316.44	4,638.98	13,278.98	8,640.00	1.537	10,959.49	2,319.49	
62	00:10:20	106.6	0.6728	104.9	0.673	3.257	13.0	5,327.54	4,637.52	13,277.52	8,640.00	1.537	10,958.76	2,318.76	
63	00:10:30	107.3	0.6836	105.5	0.684	3.265	13.2	5,358.67	4,653.45	13,293.45	8,640.00	1.539	10,966.73	2,326.73	
64	00:10:40	107.5	0.6944	105.8	0.694	3.273	13.4	5,372.80	4,654.58	13,294.58	8,640.00	1.539	10,967.29	2,327.29	
65	00:10:50	107.8	0.7051	106.1	0.705	3.281	13.6	5,388.66	4,657.24	13,297.24	8,640.00	1.539	10,968.62	2,328.62	
66	00:11:00	108.2	0.7157	106.4	0.716	3.288	13.8	5,405.89	4,661.09	13,301.09	8,640.00	1.539	10,970.54	2,330.55	
67	00:11:10	108.2	0.7263	106.5	0.726	3.296	14.0	5,408.39	4,652.23	13,292.23	8,640.00	1.538	10,966.11	2,326.11	
68	00:11:20	108.4	0.7370	106.6	0.737	3.304	14.2	5,415.65	4,647.25	13,287.25	8,640.00	1.538	10,963.63	2,323.63	

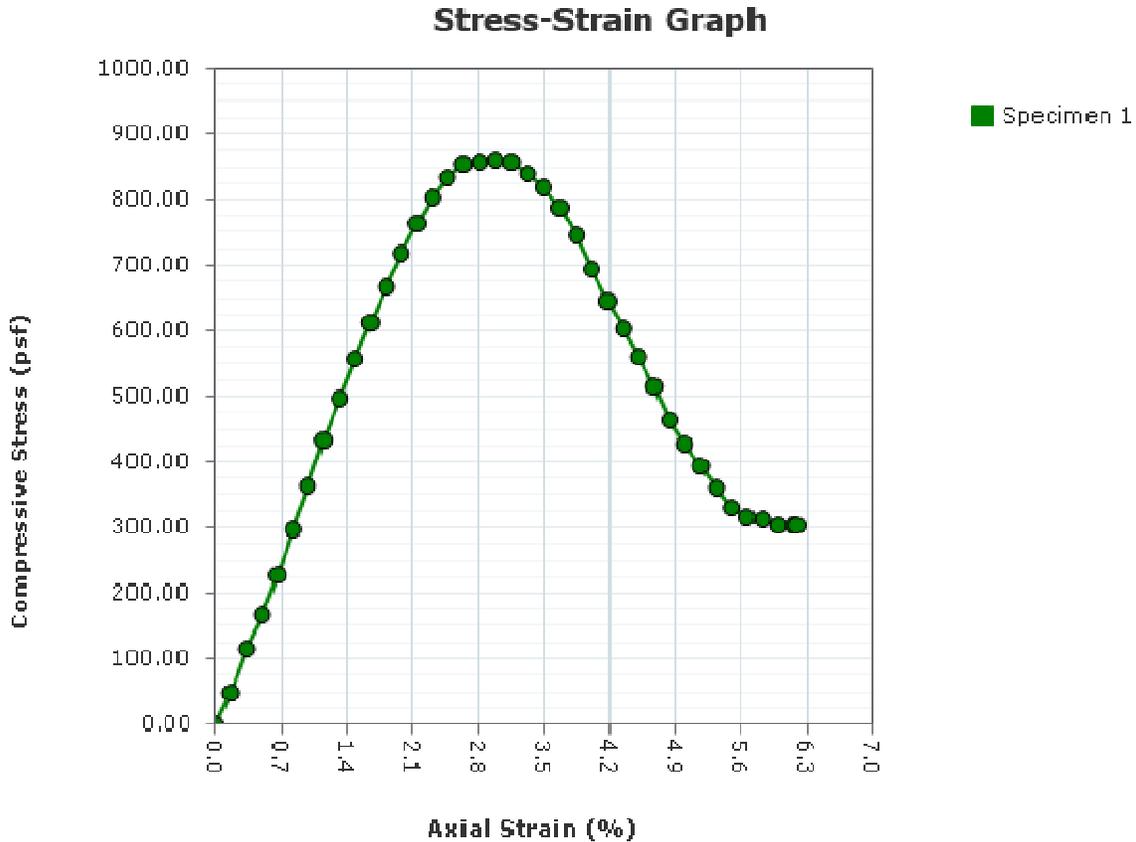
Unconsolidated Undrained Test - Tabulated Data - Specimen 1

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)	σ_1 (psf)	σ_3 (psf)	$\frac{\sigma_1}{\sigma_3}$	p (psf)	q (psf)
69	00:11:30	108.9	0.7475	107.2	0.747	3.312	14.4	5,442.20	4,659.07	13,299.07	8,640.00	1.539	10,969.54	2,329.54
70	00:11:40	109.1	0.7580	107.4	0.758	3.320	14.6	5,454.16	4,658.32	13,298.32	8,640.00	1.539	10,969.16	2,329.16
71	00:11:50	109.4	0.7687	107.7	0.769	3.328	14.8	5,470.02	4,660.59	13,300.59	8,640.00	1.539	10,970.30	2,330.30
72	00:12:00	109.8	0.7794	108.0	0.779	3.336	15.0	5,486.99	4,663.75	13,303.75	8,640.00	1.540	10,971.87	2,331.87
73	00:12:10	109.8	0.7900	108.1	0.790	3.344	15.2	5,488.38	4,653.66	13,293.66	8,640.00	1.539	10,966.83	2,326.83
74	00:12:20	109.9	0.8008	108.1	0.801	3.352	15.4	5,491.29	4,644.76	13,284.76	8,640.00	1.538	10,962.38	2,322.38
75	00:12:22	109.9	0.8037	108.2	0.804	3.354	15.5	5,493.10	4,643.17	13,283.16	8,640.00	1.537	10,961.58	2,321.58

Unconfined Compression Test

D2166



Project: Mosaic
 Project Number: 70852.00
 Received Date: 8/15/2019
 Sampling Date: 6/6/2019
 Sample Number: L1
 Sample Depth: 1.0 ft
 Boring Number: T19-4
 Location:
 Client Name: The Mosaic Project
 Remarks:

Unconfined Compression Test

D2166

	Specimen Number							
Before Test	1	2	3	4	5	6	7	8
Moisture Content (%)	17.7							
Wet Density (pcf)	105.1							
Dry Density (pcf)	89.3							
Saturation (%)	53.3							
Void Ratio	0.902							
Height (in)	5.1270							
Diameter (in)	1.8860							
Strain Limit @ 15% (in)	0.8							
Height To Diameter Ratio	2.72							
Test Data	1	2	3	4	5	6	7	8
Failure Angle (°)	0							
Strain Rate (in/min)	0.05127							
Strain Rate (%/min)	1.00							
Unconfined Compressive Strength (psf)	859.79							
Undrained Shear Strength (psf)	429.89							
Strain at Failure (%)	2.98							

Specific Gravity: 2.72	Plastic Limit: 19	Liquid Limit: 32
Type: Undisturbed	Soil Classification: CL	

Project: Mosaic
Project Number: 70852.00
Sampling Date: 6/6/2019
Sample Number: L1
Sample Depth: 1.0 ft
Boring Number: T19-4
Location:
Client Name: The Mosaic 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



Unconfined Compression Test

D2166

LIMS Code: [TO COME FROM LIMS]

Specimen 1

Other Associated Tests:

Sampling Method: Intact	Material Moisture: Specimen	Source Moisture: After Shear
Molding Date:	Test Date:	
Large Particle: NO	Sensitivity: 0	
Technician: LGH	Test Time: 7/30/2019	
Specimen Description: Unconfined Compression		
Test Remarks:		

Unconfined Compression Test - Specimen 1

D2166

LIMS Specimen Code: [TO COME FROM LIMS]

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Displacement (in)	Corrected Load (Lbf)	Corrected Displacement (in)	Axial Strain (%)	Cross Sectional Area (in ²)	Stress (psf)	Compressive Stress (psf)
0	00:00:00	0.7123572	0.0001	0.0	0.0000	0.0	0.000	0.00	0.00
1	00:00:10	1.667856	0.0082	1.0	0.0082	0.2	2.798	49.25	49.17
2	00:00:20	2.969657	0.0169	2.3	0.0168	0.3	2.803	116.35	115.97
3	00:00:30	3.990159	0.0256	3.3	0.0255	0.5	2.808	168.96	168.11
4	00:00:40	5.179595	0.0338	4.5	0.0338	0.7	2.812	230.27	228.75
5	00:00:50	6.540264	0.0422	5.8	0.0421	0.8	2.817	300.40	297.93
6	00:01:00	7.850502	0.0505	7.1	0.0504	1.0	2.821	367.94	364.32
7	00:01:10	9.196599	0.0589	8.5	0.0589	1.1	2.826	437.32	432.30
8	00:01:20	10.47021	0.0676	9.8	0.0675	1.3	2.831	502.97	496.35
9	00:01:30	11.69838	0.0762	11.0	0.0762	1.5	2.836	566.28	557.86
10	00:01:40	12.78696	0.0846	12.1	0.0845	1.6	2.840	622.39	612.13
11	00:01:50	13.90717	0.0930	13.2	0.0929	1.8	2.845	680.13	667.80
12	00:02:00	14.92575	0.1015	14.2	0.1014	2.0	2.850	732.63	718.15
13	00:02:10	15.87665	0.1100	15.2	0.1099	2.1	2.855	781.65	764.89
14	00:02:20	16.68028	0.1183	16.0	0.1182	2.3	2.860	823.07	804.09
15	00:02:30	17.30002	0.1267	16.6	0.1267	2.5	2.864	855.02	833.89
16	00:02:40	17.7144	0.1353	17.0	0.1352	2.6	2.869	876.38	853.27
17	00:02:50	17.84517	0.1441	17.1	0.1441	2.8	2.874	883.12	858.30
18	00:03:00	17.905	0.1529	17.2	0.1528	3.0	2.879	886.20	859.79
19	00:03:10	17.86511	0.1615	17.2	0.1615	3.1	2.884	884.14	856.30
20	00:03:20	17.56272	0.1703	16.9	0.1703	3.3	2.890	868.56	839.71
21	00:03:30	17.18996	0.1794	16.5	0.1793	3.5	2.895	849.34	819.64
22	00:03:40	16.56024	0.1881	15.8	0.1880	3.7	2.900	816.88	786.92
23	00:03:50	15.74319	0.1970	15.0	0.1970	3.8	2.905	774.77	745.01
24	00:04:00	14.74436	0.2056	14.0	0.2055	4.0	2.910	723.28	694.29
25	00:04:10	13.80727	0.2141	13.1	0.2141	4.2	2.915	674.98	646.80
26	00:04:20	12.93825	0.2227	12.2	0.2226	4.3	2.920	630.19	602.83
27	00:04:30	12.10969	0.2312	11.4	0.2311	4.5	2.926	587.48	561.00
28	00:04:40	11.16838	0.2395	10.5	0.2394	4.7	2.930	538.96	513.79
29	00:04:50	10.19313	0.2479	9.5	0.2478	4.8	2.936	488.69	465.07
30	00:05:00	9.447026	0.2563	8.7	0.2562	5.0	2.941	450.23	427.73
31	00:05:10	8.764199	0.2649	8.1	0.2649	5.2	2.946	415.03	393.59
32	00:05:20	8.093068	0.2735	7.4	0.2735	5.3	2.951	380.44	360.15
33	00:05:30	7.506309	0.2819	6.8	0.2819	5.5	2.956	350.20	330.94

Unconfined Compression Test - Specimen 1

D2166

LIMS Specimen Code: [TO COME FROM LIMS]

Index	Elapsed Time (hh:mm:ss)	Load (Lbf)	Displacement (in)	Corrected Load (Lbf)	Corrected Displacement (in)	Axial Strain (%)	Cross Sectional Area (in ²)	Stress (psf)	Compressive Stress (psf)
34	00:05:40	7.17151	0.2904	6.5	0.2903	5.7	2.961	332.94	314.09
35	00:05:50	7.147541	0.2987	6.4	0.2987	5.8	2.966	331.70	312.38
36	00:06:00	6.973622	0.3074	6.3	0.3073	6.0	2.972	322.74	303.39
37	00:06:10	6.949461	0.3159	6.2	0.3158	6.2	2.977	321.49	301.69
38	00:06:11	6.968253	0.3176	6.3	0.3175	6.2	2.978	322.46	302.49

APPENDIX E:

Seismic Design Parameters



Mosaic

Latitude, Longitude: 37.7418, -122.0551

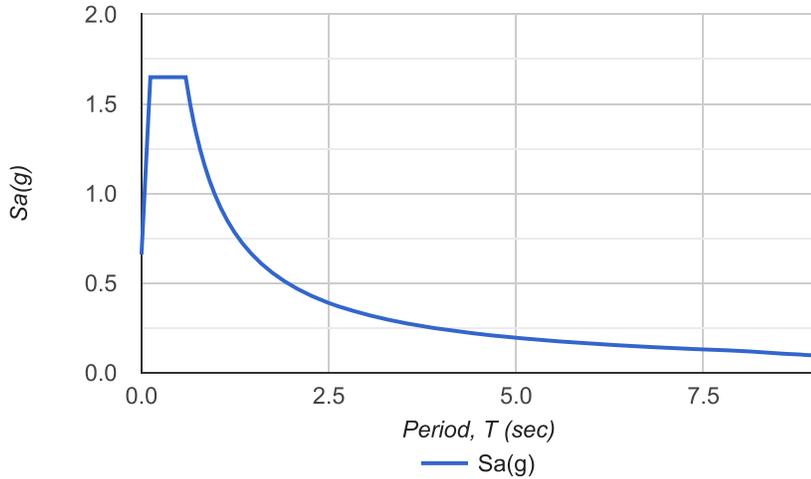


Date	8/13/2019, 2:11:48 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

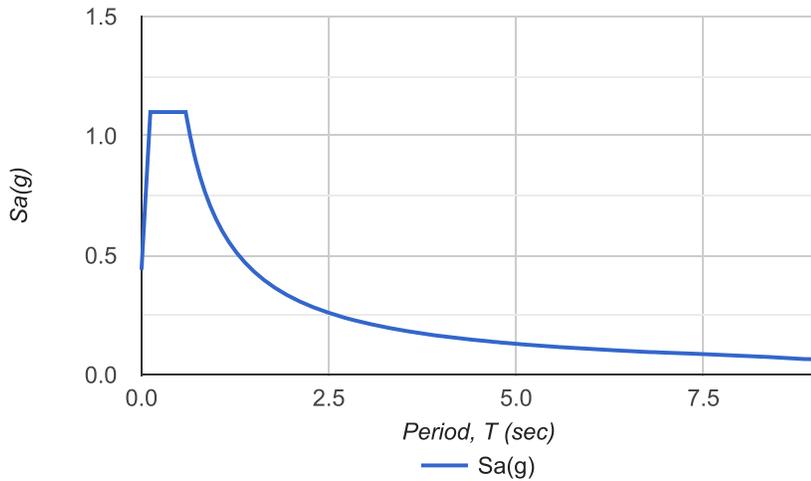
Type	Value	Description
S _S	1.649	MCE _R ground motion. (for 0.2 second period)
S ₁	0.649	MCE _R ground motion. (for 1.0s period)
S _{MS}	1.649	Site-modified spectral acceleration value
S _{M1}	0.974	Site-modified spectral acceleration value
S _{DS}	1.1	Numeric seismic design value at 0.2 second SA
S _{D1}	0.649	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F _a	1	Site amplification factor at 0.2 second
F _v	1.5	Site amplification factor at 1.0 second
PGA	0.638	MCE _G peak ground acceleration
F _{PGA}	1	Site amplification factor at PGA
PGA _M	0.638	Site modified peak ground acceleration
T _L	8	Long-period transition period in seconds
SsRT	2.677	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.556	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.649	Factored deterministic acceleration value. (0.2 second)
S1RT	0.961	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.939	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.649	Factored deterministic acceleration value. (1.0 second)
PGAd	0.638	Factored deterministic acceleration value. (Peak Ground Acceleration)
C _{RS}	1.047	Mapped value of the risk coefficient at short periods
C _{R1}	1.023	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum



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