GEOTECHNICAL DESIGN AND GEOLOGICAL HAZARDS EVALUATION REPORT
HORTICULTURAL CENTER
MERRITT COLLEGE
12500 CAMPUS DRIVE
OAKLAND, CALIFORNIA

Prepared for

Peralta Community College District
333 E 8th Street
Oakland, California 94606

Prepared by

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August 7, 2020

Project Number 0034.011.0001

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August 7, 2020

Atheria Smith
Facilities Planning and Development Manager
Peralta Community College
333 E 8th Street
Oakland, California 94606

Subject: **Geotechnical Design and Geological Hazards Evaluation Report, Proposed Horticultural Center, Merritt College, 12500 Campus Drive, Oakland, California**

Dear Ms. Smith:

Terraphase Engineering Inc. (Terraphase) is pleased to present the attached Geotechnical Design Report for the Merritt College Horticultural Center, to be located at 12500 Campus Drive, in Oakland ("the Site"). Design recommendations for building foundations and site grading are presented, along with other pertinent findings and conclusions. This version of the report was revised to reference the requirements of the 2019 California Building Code.

Terraphase observed and logged nine (9) borings at the Site to assess the subsurface soil conditions. In addition, we reviewed two boring logs for borings installed at the Site by Woodward Clyde during the initial development of the Campus. The results of our assessment indicate that, with proper preparation, the Site will be suitable to support the proposed development, provided that the Site is prepared in accordance with the recommendations contained within the attached report.

We appreciate the opportunity to provide this service for the Peralta Community College District and look forward to being of further assistance as the project proceeds. If you have any questions concerning the contents of the attached report, please feel free to call Jeff Raines at (510) 645-1853 at any time.

Sincerely,

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Attachment

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ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below ground surface
CDF	controlled density fill
CGS	California Geological Survey
DHS	Department of Health Services
DSA	Department of the State Architect
DTSC	Department of Toxic Substances Control
FEMA	Federal Emergency Management Agency
G	the acceleration of gravity at the earth's surface
m/s	meters per second
MRC	maximum rotated component
pcf	pounds per cubic foot
ppm	parts per million
psi	pounds per square inch
the Site	the proposed Merritt College Horticultural Center to be located at 12500 Campus Drive in Oakland, California
Terraphase	Terraphase Engineering Inc.
UBC	Uniform Building Code
USGS	United States Geological Survey
Woodward-Clyde	Woodward-Clyde-Sherard and Associates

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1.0 INTRODUCTION

Terraphase Engineering Inc. (Terraphase) has prepared this report to present the results of our geotechnical engineering and design study for the proposed Merritt College Horticultural Complex to be located at 12500 Campus Drive in Oakland, California ("the Site"; Figure 1). This Geotechnical Design Report is based on the proposal prepared for the Peralta Community College District by Terraphase, dated February 25, 2020.

Woodward-Clyde-Sherard and Associates (Woodward-Clyde) performed extensive investigations (1960, 1962) of the subsurface at the campus in the early 1960s. They installed 16 borings to depths up to 100 feet below ground surface (bgs). Two of these borings were installed in the area of the horticultural complex, including one in the slope northeast of the complex buildings. The locations of the Woodward Clyde and Terraphase subsurface exploration points are presented on Figure 2.

This report was prepared in general accordance with the California Division of the State Architect (DSA) requirements for the design of a public school. DSA consults with the California Geological Survey (CGS) to assess whether the geotechnical work performed for a client site is sufficient. The CGS requirements for the geotechnical reports for client sites are presented in CGS Special Publication 48 (CGS 2019). The project is to be constructed under the 2019 edition of the California Building Code and ASCE 7 (2016).

1.1 Project Description

The Site is located at 37.79° north latitude and 122.167° west longitude. The proposed project consists of the reconstruction of the Merritt College horticultural complex. We estimate that building loads will be approximately 800 pounds per foot for wall loads and 8 kips for internal columns. These estimates were used to recommend appropriate foundation types for the structure and should not be used for structural design.

1.2 Scope of Study

Based on our understanding of the client development, the following scope of services was formulated and completed:

- Terraphase observed and logged nine (9) auger borings.
- Terraphase observed a geophysical survey conducted at the Site.
- Representative soil samples were collected from the borings for analysis in a geotechnical laboratory.
- A soil sample was collected from the slope north of the complex buildings or analysis for asbestos as the Site is located in a regional area known to contain naturally occurring asbestos.

The following engineering analyses were performed to develop geotechnical engineering criteria for the proposed project:

- allowable bearing capacity of shallow foundation systems
- settlement of the proposed shallow foundation systems
- allowable passive resistance and base friction to resist wind and seismic lateral loads
- stability of the Site slopes
- retaining wall loads
- a site-specific seismic hazard study

Recommendations were developed for:

- site preparation and grading
- allowable soil-bearing pressures for shallow foundation systems
- design of slabs-on-grade
- allowable passive soil resistance and base friction
- retaining wall loads
- pavement design and construction

This report summarizes our study results and presents our design and construction recommendations and design criteria, as well as the subsurface data on which they are based.

2.0 BACKGROUND

2.1 Geology

Three different geologic formations are present at the Site: Leona Rhyolite, Knoxville Shale, and Franciscan Serpentine (Graymer 2000, Figure 9). These rocks are arranged in parallel bands, elongated northwest to southeast from one end of the campus to the other. The rhyolite, a bluish-gray, hard, somewhat fractured, fine-grained crystalline volcanic rock, forms the high ridge on the southeast along the axis of the property. The Knoxville Formation consists of interbedded layers of shale, sandstone and limestone. Woodward Clyde Hole 17 (Figure 5) encountered 2 feet of residual soil overlying 25 feet of shale overlying 19 feet of limestone overlying shale to the bottom of the boring[1]. Figure 9 shows the Site to be located near the boundary between the Leona Rhyolite and the Knoxville Formation.

Ages of erosion have cut valleys in the soft shale and left high hills where lies the hard, resistant rhyolite. Serpentine, a blue-green, fine-grained intrusive rock occurs as thin tabular bodies within the shale near the eastern extremity of the campus (Figure 9). Much of this rock is distinctly platy and weak, or highly sheared and greasy to the touch, but, locally, there are large masses of hard crystalline serpentine rock enclosed by sheared serpentine. One serpentine boulder was noted (about 12 inches in diameter) on the hillside above the Horticultural Complex.

2.2 Hydrogeology

Woodward Clyde (1960) encountered measurable groundwater at 877 feet (NAD88) in Hole 17 which is between 28 and 29 feet below the asphalt parking lot surrounding the current Horticultural Complex. Woodward-Clyde (1960, 1962) installed sixteen (16) borings to depths up to 100 feet across the campus. They reported that some of the shear zones in the shale bedrock were wet. Jensen-Van Lienden (2009) installed 11 borings to depths between 2.5 feet and 17.5 feet bgs approximately 1900 feet south of the Site, where each of the borings reached refusal, without encountering groundwater. Department of Water Resources (http://wdl.water.ca.gov/waterdatalibrary/) and United States Geological Survey (USGS) (https://waterdata.usgs.gov/nwis/uv?referred_module=gw&search_criteria=lat_long_bounding_box&search_criteria=site_tp_cd&submitted_form=introduction) databases did not locate a groundwater well in the vicinity of the Site. The Regional Water Quality Control Board database (https://www.waterboards.ca.gov/water_issues/programs/gama/online_tools.html) also failed to locate a water supply well near the Site.

2.3 Site Development

Review of the initial grading plan for the Site indicates the existing structures in the Horticultural Complex were constructed over fill used to level out the valley floor. Figure 4 presents the original grading plan for the Horticultural Complex portion of the Campus while Figure 3

[1] The Woodward Clyde elevations are given in the City of Oakland Datum which is 5.7 feet higher than the NAD88 datum. Woodward Clyde Hole 17 was placed at 917 feet City of Oakland Datum which corresponds to 911.3 feet NAD88.

presents the current topography. There is up to 30 feet of fill on the far southeastern boundary of the Horticultural Complex. The Complex buildings appear to be located on 10 to 20 feet of fill. No signs of distress due to settlement of the fill was noted in the existing structures or pavement indicating that the fill was compacted appropriately during placement.

A retaining wall is located along the southeastern boundary of the fill (Figures 3 and 4). The retaining wall is in poor condition and should be replaced as it supports a fire lane.

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Subsurface Exploration

On June 4 and 5, 2020, Gregg Drilling & Testing, Inc. of Martinez, California used a Mobil B-80 Drill Rig to install 8 borings at the locations shown on Figure 2 and a hand auger to advance a boring at the location of Boring B-7 where there was no access for the drill rig. Bedrock was encountered in all of the borings, except B-7, at depths ranging from 3 to 20 feet below the ground surface (bgs). The fill encountered in the borings was mainly clay and clayey gravels. The two borings in the locations of the deepest fills (B-3 and B-4) had depths to bedrock essentially equal to the fill depths indicating that the soil depths prior to development were shallow.

Boring B-2 was installed in the backfill for a former underground storage tank (UST). While the blow counts for the soil samples in the UST excavation backfill indicated the backfill had been compacted, the backfill beneath future building foot prints should be excavated to a depth of five feet and be recompacted in accordance with the recommendations of Section 7.2 of this report.

Boring logs are attached to this report in Appendix A.

3.2 Laboratory Testing

Soil samples collected from the borings were submitted to the Cooper Testing Laboratory of Palo Alto, California for analysis for soil characteristics: gradation, Atterberg Limits and corrosion characteristics.

A soil sample collected from the northern slope above the existing structures was submitted to Micro Analytical Laboratories of Berkeley, California, for analysis of asbestos by Air Resources Board Method 435 (polarized light microscopy).

The results of the laboratory testing are presented in Appendix B.

3.3 Geophysical Survey

Advanced Geological Services (AGS) of Moraga, California performed a geophysical survey at the Site on May 13, 2020. AGS collected shear and pressure wave velocity profiles along two transits, one at the toe of the northern slope and one up the northern slope parallel to the slope gradient. The purpose of the work was to evaluate the shear wave velocity in the top 30 meters of the Site for the seismic risk assessment and to verify that only a thin veneer of soil was present on the northern slope. The geophysical report is attached to this report in Appendix C.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Site Description

The Site is located at 12500 Campus Drive in Oakland, California. The Site consists of valley fill over bedrock and is level at an elevation of 905 feet above mean sea level based on the NAD88 datum. The fill depth varies in depth from around 30 feet on the far southeastern boundary of the Site to zero feet at the northwestern end of the Site. Bedrock varies between 20 and 10 feet bgs below the existing and proposed buildings.

The site is in a valley between two low hills. The slope to the northeast is a 2.1 horizontal to 1 vertical (2H:1V) graded slope consisting of shale overlying limestone. The limestone does not outcrop and is all situated below the elevation of the complex buildings. The slope was originally, pre-campus development, 3.6H:1V near the complex buildings and 2.3H:1V further to the northwest. It was graded during construction of the Horticultural Complex to provide a level building site resulting in a 2.1H:1V slope to the northeast of the complex.

The hillside to the southwest is ungraded with a 4H:1V slope mapped as rhyolite and is not considered a stability issue locally nor for the proposed project.

The edge of the valley fill along the southeast boundary of the Site is supported by a wood retaining wall in poor condition. We recommend replacing the retaining wall.

4.2 Subsurface Conditions

Bedrock was encountered between 2 feet (Boring B-8) and 20 feet (Borings B-3 and B-4) bgs with the depth to bedrock increasing from northwest to southeast. The site soils consisted of gravelly-clays and clay, probably reworked residual soils and weathered shales excavated from the northeast slope (the southwest slope is rhyolite which would have been very difficult to rip). No serpentine gravel was seen in the auger cuttings from the borings. The site was classified as Site Class C based on the geophysical survey conducted at the Site (Appendix C) where the measured shear wave velocity in the upper 30 meters (VS30) was found to be 681 meters per second (m/s).

Figure 6 presents a cross-section of the Site running southeast to northwest. Figure 7 presents a cross-section of the Site running northeast to southwest.

4.3 Groundwater

Discernable ground water was not encountered in any of the Terraphase borings prior to backfilling. Woodward Clyde encountered groundwater at elevation 889 feet above mean sea level (NAD88) in Hole 17, which would be approximately 16 feet below the current ground surface at the Site.

4.4 Site Seismicity

A review of available earthquake hazard maps (CGS 2003a and Figure 10) indicates that the Site is not located within an Earthquake Special Studies Zone. The nearest such zone is 0.7 miles southeast of the Site and is associated with the mapped Hayward Fault. While ground rupture at the Site is unlikely, strong ground shaking will likely occur at the Site during the useful economic life of the proposed structure. Graymer and Brabb (1995) map several inactive faults crossing the Site.

A site-specific earthquake ground motion study, appended to this report in Appendix D, was conducted for the Site in accordance with the 2019 California Building Code (State of California 2018) and ASCE 7 (ASCE 2016). The 2014 NGA WEST2 ground motion predictors were used in the site-specific seismic hazard assessment. A shear wave velocity of 681 m/s was used in the site-specific seismic hazard assessment. The result of the site-specific seismic hazard assessment was S_{DS} is equal to 1.588g and S_{D1} is equal to 0.711g.

Mapped ASCE 7 (ASCE 2016) seismic design parameters (S_S, S_1, S_{DS} and S_{D1}) are shown in Appendix D based from the ASCE Hazard Tool - https://asce7hazardtool.online/. The Seismic Design Category is E. The mapped expected peak ground acceleration for the Maximum Considered Earthquake at the Site is 1.25g, where "g" is the acceleration of gravity at the earth's surface. The mapped S_{DS} is 1.99g.

4.5 Seismic Environment

Regionally active faults within 100 kilometers of the Site that are capable of producing significant ground shaking at the Site are shown on Figure 11 and presented in Table 1.

Table 1: Known Active Earthquake Faults within 100 Kilometers of the Site
Merritt College Horticultural Center, Oakland, California

Source	Distance (Kilometers)	Magnitude	Mechanism	Angle	To	Lies
Hayward-Rodgers Creek	1.1	7.334	Strike Slip	90	--	E
Calaveras	14	7.025	Strike Slip	90	--	W
Mount Diablo Thrust	16.9	6.7	Reverse	38	NE	SW
Green Valley Connected	19.98	6.8	Strike Slip	90	--	SW
Northern San Andreas	30.9	8.05	Strike Slip	90	--	NE
Greenville Connected	30.92	7	Strike Slip	90	--	W
Greenville Connected U	30.92	7	Strike Slip	90	--	W
San Gregorio Connected	38.12	7.5	Strike Slip	90	--	E
Great Valley 5, Pittsburg Kirby Hills	38.98	6.7	Strike Slip	90	--	SW

Source	Distance (Kilometers)	Magnitude	Mechanism	Angle	To	Lies
Monte Vista-Shannon	39.8	6.501	Reverse	45	SW	N
West Napa	42.3	6.7	Strike Slip	90	--	S
Great Valley 7	52.09	6.9	Reverse	15	SW	W
Great Valley 4b, Gordon Valley	52.1	6.8	Reverse	20	W	S
Point Reyes	61.21	6.9	Reverse	50	NE	E
Hunting Creek-Berryessa	73.84	7.1	Strike Slip	90	--	S
Great Valley 4a, Trout Creek	77.88	6.6	Reverse	20	SW	S
Zayante-Vergeles	79.63	7	Strike Slip	90	--	N
Nonextensional Gridded	86.62	10	SS|R	90	--	S
San Andreas Creeping Section Gridded	86.84	6	Strike Slip	90	--	NW
Great Valley 8	91	6.8	Reverse	15	W	NW
Great Valley 3, Mysterious Ridge	95.8	7.1	Reverse	20	SW	S
Monterey Bay-Tularcitos	96.52	7.3	Strike Slip	90	--	N
Ortigalita	96.86	7.1	Strike Slip	90	--	NW
Maacama-Garberville	98.9	7.4	Strike Slip	90	--	SE

Source: EZ FRISK Version 7.65 Build 004

4.6 Historical Seismicity

The known earthquakes of note to affect the San Francisco Bay Area are shown on Figure 12 and presented in Table 2 below.

**Table 2: Historical Earthquakes in the Bay Area with Magnitudes Greater than 5.0
Merritt College Horticultural Center, Oakland, California**

Location	Date	Depth	Magnitude
South Napa	2014-08-24 10:20:44 (UTC)	11.1 km	6.0
San Francisco Bay area, California	2007-10-31 03:04:54 (UTC)	9.7 km	5.5
Loma Prieta (not shown on figure)	1989-10-18 00:04:15 (UTC)	17.2 km	6.9
San Francisco Bay area, California	1988-06-13 01:45:36 (UTC)	9.1 km	5.3
Northern California	1986-03-31 11:55:39 (UTC)	8.5 km	5.7
Northern California	1984-04-24 21:15:18 (UTC)	8.2 km	6.2

San Francisco Bay area, California	1980-01-27 02:33:35 (UTC)	14.2 km	5.4
San Francisco Bay area, California	1980-01-24 19:01:01 (UTC)	6.5 km	5.1
San Francisco Bay area, California	1980-01-24 19:00:09 (UTC)	11.0 km	5.8
San Francisco Bay area, California	1957-03-22 19:44:21 (UTC)	–	5.3
San Francisco Bay area, California	1955-10-24 04:10:44 (UTC)	–	5.4
San Francisco Bay area, California	1955-09-05 02:01:18 (UTC)	–	5.5
San Francisco Bay area, California	1911-07-01 22:00:03 (UTC)	–	6.6
The 1906 San Francisco Earthquake	1906-04-18 13:12:26 (UTC)	11.7 km	7.9
San Francisco Bay area, California	1903-08-03 06:49:00 (UTC)	–	5.8
San Francisco Bay area, California	1903-06-11 13:12:00 (UTC)	–	5.8
Northern California	1902-05-19 18:31:00 (UTC)	–	5.4
San Francisco Bay area, California	1889-05-19 11:10:00 (UTC)	–	6.0
San Francisco Bay area, California	1868-10-21 15:53:00 (UTC)	–	6.8
Alameda County	1864-03-05 16:49:00 (UTC)	–	6.1
San Francisco Bay area, California	1858-11-26 08:35:00 (UTC)	–	6.1
San Francisco Bay area, California	1836-06-10 15:30:00 (UTC)	–	6.8

5.0 GEOLOGICAL HAZARDS

5.1 Landslides

CGS maps the slopes northeast of the Site as being subject to seismically induced landsliding (Figure 10). Please see Appendix E for a discussion of slope stability.

5.2 Liquefaction

The Site is underlain by compacted clayey fill and bedrock which are not subject to liquefaction or seismic shakedown settlement.

5.3 Ground Rupture Potential

As shown on Figure 10, the Site is not within an earthquake fault zone. Hence, the likelihood of a ground-crossing fault at the Site is low.

5.4 Flooding

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map indicates that the Site is not located within a 100-year flood zone (Figure 8) (FEMA 2009). The Site has been mapped in “Zone X,” which represents “Areas of minimal flooding.”

There is one large East Bay Municipal Utility District water tank located uphill from the Site near Skyline High School. However a release from this tank would pass to the north of the Site, so dam or flood inundation is not an applicable hazard for the Site.

5.5 Expansive Clay and Collapse Potential

Two samples of Site soils were analyzed for Atterberg limits resulting in Plasticity Indices of 13 and 15 which do not indicate significant expansive properties. The existing one-story buildings on the Site, which are at least 50 years old, show no signs of differential movements due to expansive clays. Hence, we judge the likelihood of expansive clays at the Site to be low.

5.6 Naturally Occurring Asbestos

The regional geological map (Figure 9) indicates that there are large areas of serpentinite bedrock in the Site’s vicinity and a serpentinite boulder was seen on the northeast slope above the facility. A composite soil sample collected from the northeast slope was analyzed by the Micro Analytical Laboratory of Emeryville, California by California Air Resources Board Method 435 (polarized light microscopy). No asbestos fibers were seen in the sample. The laboratory report is appended to this report in Appendix B.

Regardless, we recommend that any dust-generating activities at the Site be controlled with water sprays.

5.7 Other Hazards

Certain other potential geological hazards, including tsunamis, seiches, naturally occurring radon, and oil and gas fields, do not appear to pose significant risks at the Site, for the reasons discussed briefly below.

- **Tsunamis and Seiches.** Tsunamis do not pose an appreciable risk at this inland location. Seiches do not pose an appreciable risk given the absence of nearby surface water bodies.

- **Naturally Occurring Radon.** The California Department of Health Services (DHS) maintains a database of radon measurements in California, based on zip code. No elevated radon results (greater than or equal to 4.0 picoCuries per liter) have been reported in 47 measurements from the 94619 (Oakland) zip code, which includes the Site.

- **Oil and Gas Fields.** The Site is not located within an oil or gas field, as recognized by the California Department of Oil, Gas, and Geothermal Resources (DOGGR 2019).

- **Volcanos.** While the Site contains large amounts of Pleistocene-age (11,700 to 1,800,000 years before the present) igneous rock (Leona Rhyolite; USGS 1968), it is unlikely that there will be a new eruption within the useful economic lifetime of the proposed building.

5.8 Conditional Geotechnical Topics

The proposed structure will not have a basement or deep foundations. There are no nearby structures that might be affected by the new structure.

6.0 FOUNDATIONS

Conventional spread footing foundations or slab-on-grade foundations are suitable for support of the proposed building loads. Foundation design recommendations are presented in Section 7.5.

6.1 Settlement Estimates (Including Seismic Shakedown)

Settlement was estimated for a foundation supported on the native soils (engineered fill), as will be less than ¼ inch. As indicated above, the existing one-story buildings at the Site do not display any indications of differential settlements.

7.0 DESIGN RECOMMENDATIONS

7.1 Site Preparation and Grading

The existing buildings should be demolished and removed. Existing foundation elements should be removed Soils disturbed by the demolition of the existing structures should be over-excavated and be recompacted in accordance with the recommendations in Section 7.3 of this report.

7.2 Fill Recommendations

It does not appear that any fill will need to be imported to the Site. For completeness, if any fill is brought to the Site it must meet the following requirements.

Imported fill materials should be approved by the engineer before being brought to the Site. Imported fill shall be certified as clean from the source (not from former industrial sites or similar locations; not chemically affected). Any imported fill should be characterized in accordance with Department of Toxic Substances Control guidance (DTSC 2001).

Imported fill should be nonexpansive, with between 5% and 25% finer than a No. 200 sieve and meet the following requirements: minimum R-Value of 35 (California Department of Transportation [Caltrans] 301), maximum expansion index of 25 (Uniform Building Code [UBC] 18-2), and maximum plasticity index of the fine fraction of 12 (ASTM International [ASTM] D4318). The soil should be compacted in lifts no greater than 8 inches loose to a minimum of 90% of the soil's maximum dry density as determined using the methodology of ASTM D1557 (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort). A representative of the geotechnical engineer should observe site grading, including stripping, scarifying, and placing and compacting of fill and backfill.

Imported fill should not have chloride concentrations in excess of 400 parts per million (ppm), sulfate concentrations in excess of 1,500 ppm, and the pH should not be less than 6.

Controlled density fill (CDF), if used, shall be composed of cementitious materials, aggregate, water, and an air-entraining admixture, as follows:

1. Cementitious materials shall be Portland cement in combination with fly ash.
2. Admixture shall be an air-entraining agent.
3. Aggregate Content: CDF mixture shall contain no aggregate larger than 3/8 inch. Amount passing a No. 200 sieve shall not exceed 12 percent. No plastic fines shall be present.
4. Air Content: Total calculated air content of the sample, prepared in accordance with ASTM C231, shall not exceed 30 percent.

5. Strength: At 28 days, CDF shall have an unconfined compressive strength of from 50 pounds per square inch (psi) to a maximum of 150 psi.

7.3 Excavation and Backfilling

Trenches should be excavated as required by the plans and specifications, using appropriate equipment. Where necessary, trenches should be sloped or shored by the contractor, in accordance with the governing safety standards to provide a safe work site. The contractor shall be responsible for any temporary slopes and trenches excavated at the Site and for design of shoring, should it be required.

Excavations should be backfilled with compacted fill, in accordance with the stricter of the recommendations contained in this section or in accordance with local requirements. Fill material, including over-excavated Site soils, should be placed in lifts no greater than 8 inches in loose thickness and compacted by mechanical means. Backfill should be compacted to at least 90% of the soil's maximum dry density (ASTM D1557) except where located within a pavement section where the upper 18 inches of the backfill below subgrade level will require compaction to at least 95% (ASTM D1557) of the soil's maximum dry density.

7.4 Excavations Adjacent to Buildings

Trenches and other excavations located adjacent to existing foundations should be located such that an imaginary line drawn at a 45-degree angle from the bottom of the outer edge of the spread footing does not intersect the trench.

Trenches and other excavations that will pass close to a future spread footing or slab-on-grade foundation should be backfilled with clean fill compacted to at least 95% relative compaction or with flowable fill prior to construction of the foundation or slab.

Trenches to be excavated parallel to an existing slab-on-grade foundation should be located such that an imaginary line drawn at a 45-degree angle from the bottom of the outer edge of the slab does not intersect the trench. If this is not possible, the trench can be installed in 5-foot-long sections with each section backfilled with clean fill compacted to at least 95% relative compaction or with flowable fill prior to excavation of the next segment of the trench.

For other trench/foundation layouts, please consult with the engineer.

7.5 Spread or Continuous Footings

Spread or continuous footings should bear on the native soils (fill). Continuous and isolated spread footings should have minimum widths of 18 inches and 24 inches, respectively, and should extend at least 18 inches below the lowest exterior grade or to the top of bedrock. The following are recommended allowable bearing pressures for foundation elements:

**Table 3: Allowable Bearing Pressures for Spread or Continuous Footings
Merritt College Horticultural Center, Oakland, California**

Loading Condition	Allowable Bearing Pressure
Dead Loads	2,500 psf
Dead plus Live Loads	3,000 psf
All Loads, including Wind or Seismic	4,000 psf

Note: psf = pounds per square foot

The minimum size footings listed above will likely govern rather than the allowable bearing pressures as the minimum footing size will likely have more than enough capacity to support the building loads.

Footing concrete should be poured neat against engineered fill or bedrock. Any disturbed or softened material encountered at the bottom of the footing excavations should be removed to expose firm bearing material. Footing excavations should be kept moist before concrete placement.

Continuous footings should be reinforced with a minimum of at least two (2) #4 bars top and bottom in the longitudinal direction unless otherwise determined by the structural engineer. Isolated spread footings should be reinforced with a minimum of two (2) #4 bars in each direction. Reinforcement should be spaced 12 inches on center in each direction unless otherwise determined by the structural engineer.

Before issuing the construction bids, the geotechnical engineer should review the foundation plans and prepare a review letter. In addition, the geotechnical engineer should observe foundation operations.

7.6 Soil Spring Constants

Soil spring constants can be modeled using a spring constant (modulus of subgrade reaction) of 60 pounds per cubic inch. This value does not need to be scaled by the width of grade beams. It is not applicable to mat foundations.

7.7 Concrete Slabs-on-Grade

Slab-on-grade floors should be supported on a minimum of 4 inches of clean gravel or crushed rock. We recommend that moisture-sensitive foundations in direct contact with the subsurface (mechanical rooms, office and classrooms, et cetera) be underlain by a moisture barrier. A typical moisture barrier should include a capillary moisture break consisting of at least four (4) inches of clean, free-draining gravel or crushed rock (1/2 to 3/4 inch gradation) overlain by a moisture-proof membrane of at least 10 mils thick (15-mil Stego, Grace FlorPrufe, or equivalent). The vapor retarder should be covered with two (2) inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction unless the structural engineer recommends placing the concrete directly on the vapor barrier. Water should not be allowed to accumulate in the capillary break or sand prior to casting of the slab.

The vapor retarder should meet the requirements for Class C vapor retarders as given in ASTM Standard E1745-97. The vapor retarder should be installed in general accordance with the methodology documented in ASTM Standard E1643-98. These requirements include overlapping seams by at least six (6) inches, taping seams, and sealing penetration through the vapor retarder. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in the following table.

Material for support of slabs should conform to the following gradation specification:

Table 4: Subslab Foundation Materials
Merritt College Horticultural Center, Oakland, California

Material	Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	1 inch	90 – 100
	¾ inch	30 – 100
	½ inch	5 – 25
	$^{3}/_{8}$ inch	0 – 6
Sand	No. 4	100
	No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed. There should be no free liquid in the sand. If the sand has been placed and there is a possibility for precipitation, the sand should be covered with Visqueen and measures be made available to collect the precipitation and remove it from the Visqueen.

The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement. All slabs should be poured at a maximum slump of less than 5 inches. Excessive water content is the major cause of concrete cracking.

The project structural engineer should design the reinforcement and joints of any slabs proposed for the Site. The following recommendations are minimums. Slabs-on-grade should be a minimum of 4 inches thick and should be reinforced with at least No. 4 reinforcing bars placed at 18 inches on-center both ways at or slightly above the center of the structural section. Reinforcing bars should have a minimum clear cover of 1.5 inches, and hot bars should be cooled prior to placement of concrete. The aforementioned reinforcement may be used for anticipated uniform floor loads not exceeding 100 psf. If floor loads greater than 100 psf are anticipated, the slab should be evaluated by a structural engineer.

We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet, though the structural engineer should make the final decision on construction joints. Construction joints that abut the foundations should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

7.8 Lateral Loads

Resistance to lateral loads from wind or seismic forces would be obtained from passive resistance on the vertical faces of footings. We recommend an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used for a passive resistance value acting on faces of embedded foundation members. The top foot of soil resistance, but not the weight of the top foot of soil, should be neglected in these calculations unless the soil around the footings is topped with asphalt. The friction on the bottoms of footings and nonstructural slabs-on-grade also may be included in the design. A friction coefficient of 0.35 can be used for calculating base friction for footings. Where a vapor barrier is used between slab-on-grade and soil, a friction coefficient of 0.20 is recommended. These friction coefficient values do not include a factor of safety.

Backfill against structures should be compacted to a minimum of 90% relative compaction (ASTM D1557).

7.9 Pavement Design

Pavements for this project are expected to consist of parking and travel areas. We have assumed that traffic loading will consist of light-duty pavement for light auto traffic, parking. We have assumed a Traffic Index of 4 for pavement design calculations.

The table below presents recommended pavement sections for the assumed Traffic Index based on the Caltrans Flexible Pavement Design Method. If imported fill is required in pavement areas, it should have an R-Value that is equal to or greater than that of the existing soils.

We recommend the following pavement design sections based on our experience with similar projects.

Table 5: Flexible Pavement Section
Merritt College Horticultural Center, Oakland, California

Recommended Pavement Designs (R-Value =35)		
Traffic Index	**Pavement Component**	**Minimum Thickness (inches)**
4	asphalt concrete	3
	aggregate base	4

Note: While the Traffic Index is 4, the calculation, per Caltrans requirements, is based on a Traffic Index of 5.

Aggregate Base is to be Caltrans Type 2. Asphalt concrete shall meet the current requirements of the Caltrans District Engineer for the Oakland area.

To prepare for pavement construction, the exposed subgrade, if it is native soil, should be scarified to a depth of 6 inches and be compacted to at least 95% relative compaction (ASTM D1557).

Aggregate base should be compacted in one lift to a minimum of 95% relative compaction (ASTM D1557).

Concrete slabs-on-grade should be used for trash-collection areas and other locations that may experience heavy wheel or impact loads. The slab thickness should be designed to accommodate the anticipated vehicle loading and the subgrade modulus of 100 pounds per cubic inch divided by the width of the slab in feet. The concrete pavement should be supported on a minimum of 6 inches of Caltrans Class 2 aggregate base rock compacted to at least 95% relative compaction (ASTM D1557) over 6 inches of recompacted subgrade, also compacted to at least 95% relative compaction (ASTM D1557).

7.10 Site Drainage

All exterior surface areas should be sloped a minimum of 2% away from the buildings to facilitate drainage. In hardscape areas, drainage gradients should be maintained to carry surface water to area drains or off the Site. Surface-water ponding should not be allowed anywhere on the Site during or after construction. If planter areas will be created between buildings and walkways, drainage inlets should be placed and the ground surface sloped to collect and drain surface water. A representative of the geotechnical engineer should review the site-drainage plans and conduct a final drainage review.

7.11 Soil Corrosivity

The 2018 Caltrans Corrosion Guidelines (Caltrans 2018) considers sites to be corrosive if one or more of the following conditions exist for the representative soil/water sample collected from the site:

- Soil with less than 1,100 ohm-centimeters resistivity
- Chloride concentration is 500 parts per million (ppm) or greater
- Sulfate concentration is 1,500 ppm or greater
- pH is 5.5 or less

As shown in Appendix B, the chloride concentration in the soil sample collected from the subsurface soils was 6 milligrams per kilogram (mg/kg, ppm); the sulfate concentration was 81 mg/kg; the pH was 6.5 and the minimum resistivity was 3,988 Ohm-centimeters.

We are not corrosion engineers. Caltrans would not consider the sample collected at the Site to be representative of a corrosive soil.

7.12 Exterior Flatwork

It is recommended that exterior concrete flatwork be a minimum of 4 inches thick and reinforced with reinforcing bars. Exterior flatwork should be underlain by at least 4 inches of aggregate base rock conforming to Caltrans Class 2 standards that is compacted to a minimum of 92% relative compaction (ASTM D1557). The exterior flatwork should be poured separately from building foundations so that they act independently of the walls and foundations. Soils below exterior flatwork should be scarified to a depth of 6 inches and be compacted to a minimum of 90% relative compaction (ASTM D1557).

7.13 Retaining Wall Loads

The retaining wall supporting the fire lane at the southeast end of the Site is in poor condition and should be replaced. The replacement wall can be designed to support a soil with an equivalent fluid weight of 50 pounds per cubic foot.

Walls taller than 6 feet in height should be designed to incorporate an equivalent fluid load equal to 12 pounds per cubic foot to account for seismic earth pressures. This assumes that the wall can move up to 6 inches outward during an earthquake.

Place rock drains or geosynthetic drainage composites behind the retaining wall to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A 12-inch-thick layer of gravel (Caltrans Specification 68-2.02F) placed directly behind the wall, or
2. A 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve and less than 1% passing a No. 200 sieve. Envelop rock in a Mirafi 140N (or equivalent) geotextile.

Composite drainage layers should be permeable both horizontally and vertically if used with a wall that is not watertight (e.g., lagging walls).

As the wall will drain to the adjacent property, the wall drainage should be connected to a spreader pipe 10 feet from the property line.

Lateral loads acting on retaining walls due to vehicular surcharges (e.g., firetrucks) should be superimposed on the earth pressures. Lateral loads due to traffic loads should be computed for any portion of the retaining wall face within a 45-degree plane of the adjacent fire lane. A lateral load, equal to 40 percent of the vertical load from fire truck wheels should be applied to the retaining wall at a point 45 degrees down from the edge of pavement.

The design loads presented above do not include a factor of safety.

8.0 DESIGN REVIEW AND CONSTRUCTION MONITORING

Terraphase recommends that the geotechnical aspects of the project be reviewed by Terraphase during the design process. The scope of services may include:

- assisting the design team in providing specific recommendations for special cases
- reviewing the foundation design and evaluating the overall applicability of our recommendations
- reviewing the geotechnical portions of the project for possible cost savings through alternative approaches
- reviewing the proposed construction techniques to evaluate whether they satisfy the intent of our recommendations
- reviewing and stamping drawings

Terraphase recommends that foundation construction and earthwork performed during construction be monitored by a qualified representative from our office, including:

- site preparation (stripping and grading)
- placement of compacted fill and backfill
- all foundation excavations
- construction of slab, roadway, and/or parking-area subgrade

Terraphase's representative should be present to observe the soil conditions encountered during construction to evaluate the applicability of the recommendations presented in this report to the soil conditions encountered and to recommend appropriate changes in design or construction procedures, if conditions differ from those described herein.

9.0 LIMITATIONS

The opinions and recommendations presented in this report are based upon the scope of services, information obtained through the performance of the services, and the schedule as agreed upon by Terraphase and the party for whom this report was originally prepared. This report is an instrument of professional service and was prepared in accordance with the generally accepted standards and level of skill and care under similar conditions and circumstances established by the geotechnical consulting industry. No representation, warranty, or guarantee, express or implied, is intended or given. To the extent that Terraphase relied upon any information prepared by other parties not under contract to Terraphase, Terraphase makes no representation as to the accuracy or completeness of such information. This report is expressly for the sole and exclusive use of the party for whom this report was originally prepared for a particular purpose and only in its entirely. Only the party for whom this report was originally prepared and/or other specifically named parties have the right to make use of and rely upon this report. Reuse of this report or any portion thereof for other than its intended purpose, or if modified, or if used by third parties, shall be at the user’s sole risk.

Furthermore, nothing contained in this report shall relieve any other party of its responsibility to abide by contract documents and applicable laws, codes, regulations, or standards.

Review

In the event that any change in the nature, design, or location of the proposed structure(s) is planned, the conclusions and recommendations in this report shall not be considered valid nor relied upon unless the changes are reviewed and the conclusions and recommendations of this report are modified or verified in writing.

Terraphase should be provided the opportunity for a general review of final design plans and specifications to assess that our recommendations have been properly interpreted and included in the design and construction documents.

Construction

To verify conditions presented in this report and modify recommendations based on field conditions encountered in the field, Terraphase should be retained to provide geotechnical engineering services during the construction phase of the project. This is to observe compliance with design concepts, specifications, and recommendations contained in this report, and to verify and refine our recommendations as necessary in the event that subsurface conditions differ from those anticipated prior to the start of construction.

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FIGURES

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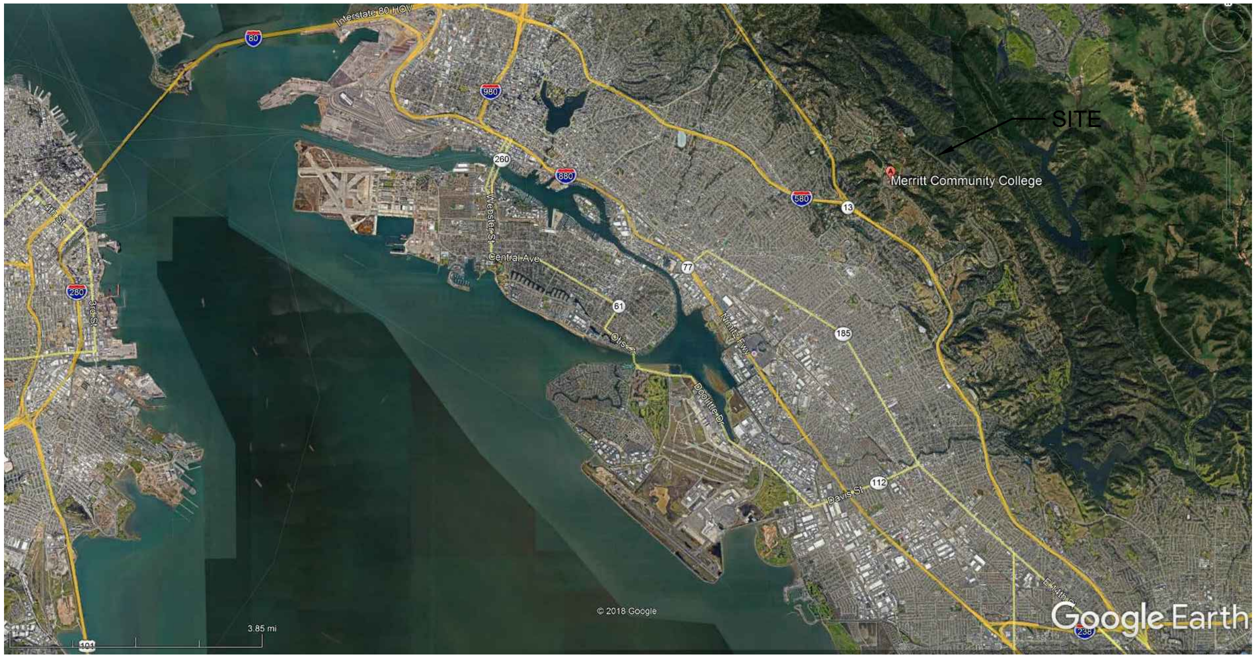

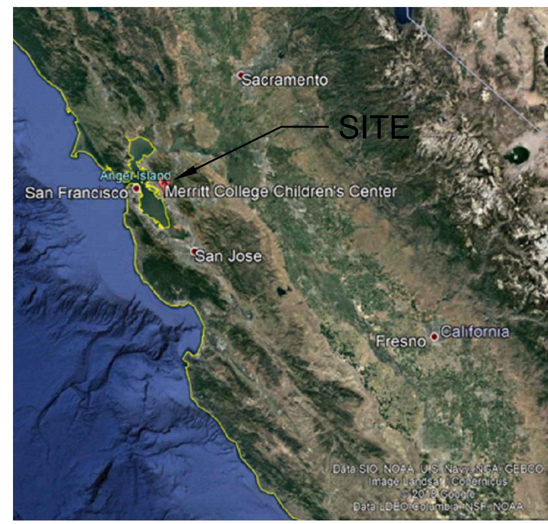

N

0 0.75 1.5 3
Approximate Scale in Miles

Source: Google Earth

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DISTR	**SITE LOCATION**
terraphase engineering	PROJECT: HORTICULTURAL CENTER	
	PROJECT NUMBER: 0034.005.0003	**Figure 1**

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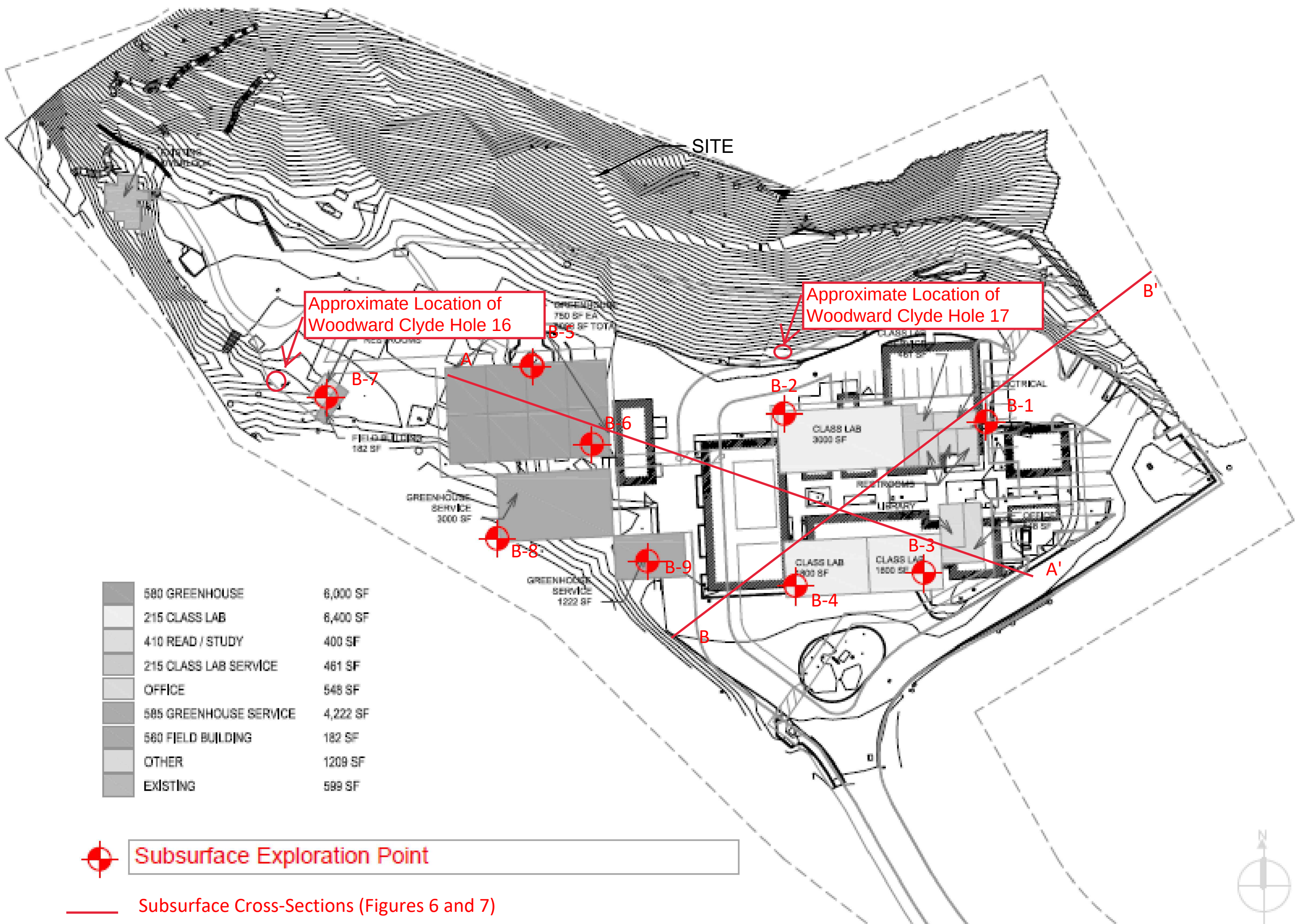

0 25 50 100
Approximate Scale in Feet

Source: IDA Structural Engineers

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DISTR	**BORING LOCATIONS**
terraphase engineering	PROJECT: HORTICULTURAL CENTER	
	PROJECT NUMBER: 0034.005.0003	**Figure 2**

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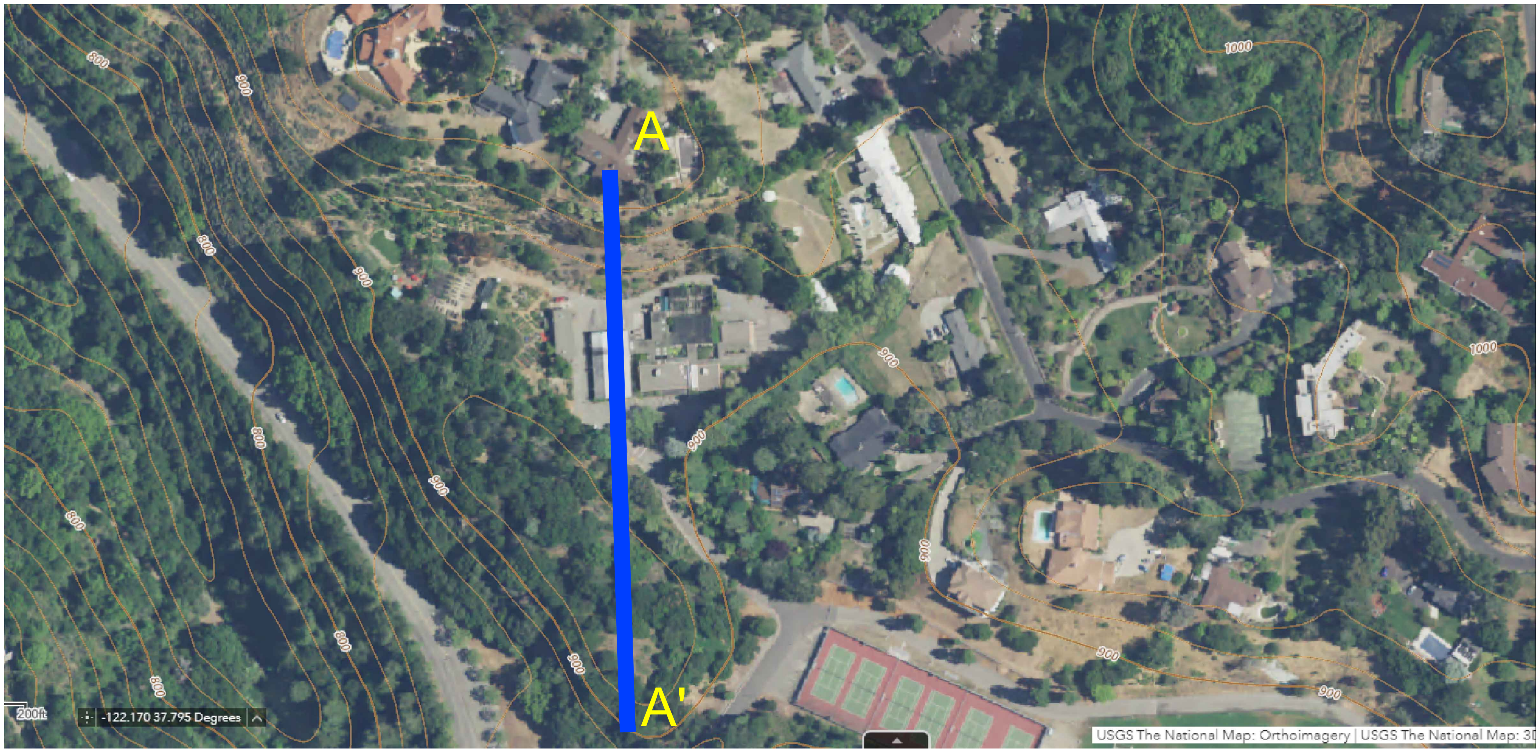

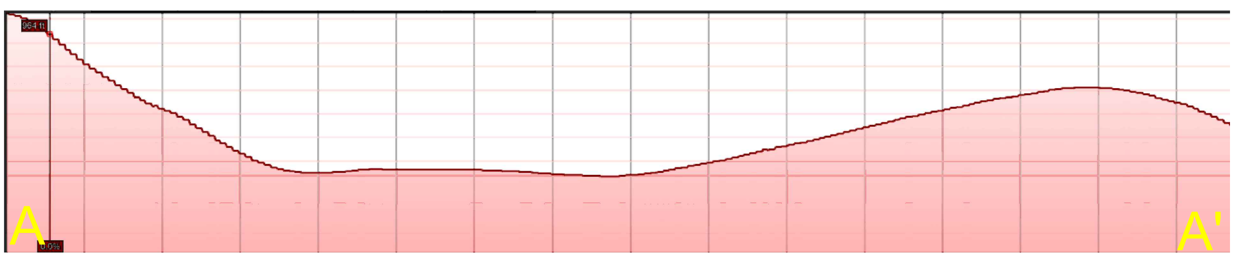

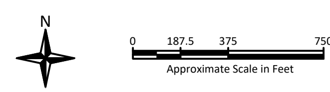

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DISTR	**REGIONAL TOPOGRAPHY**
terraphase engineering	PROJECT: HORTICULTURAL CENTER	
	PROJECT NUMBER: 0034.005.0003	**Figure 3**

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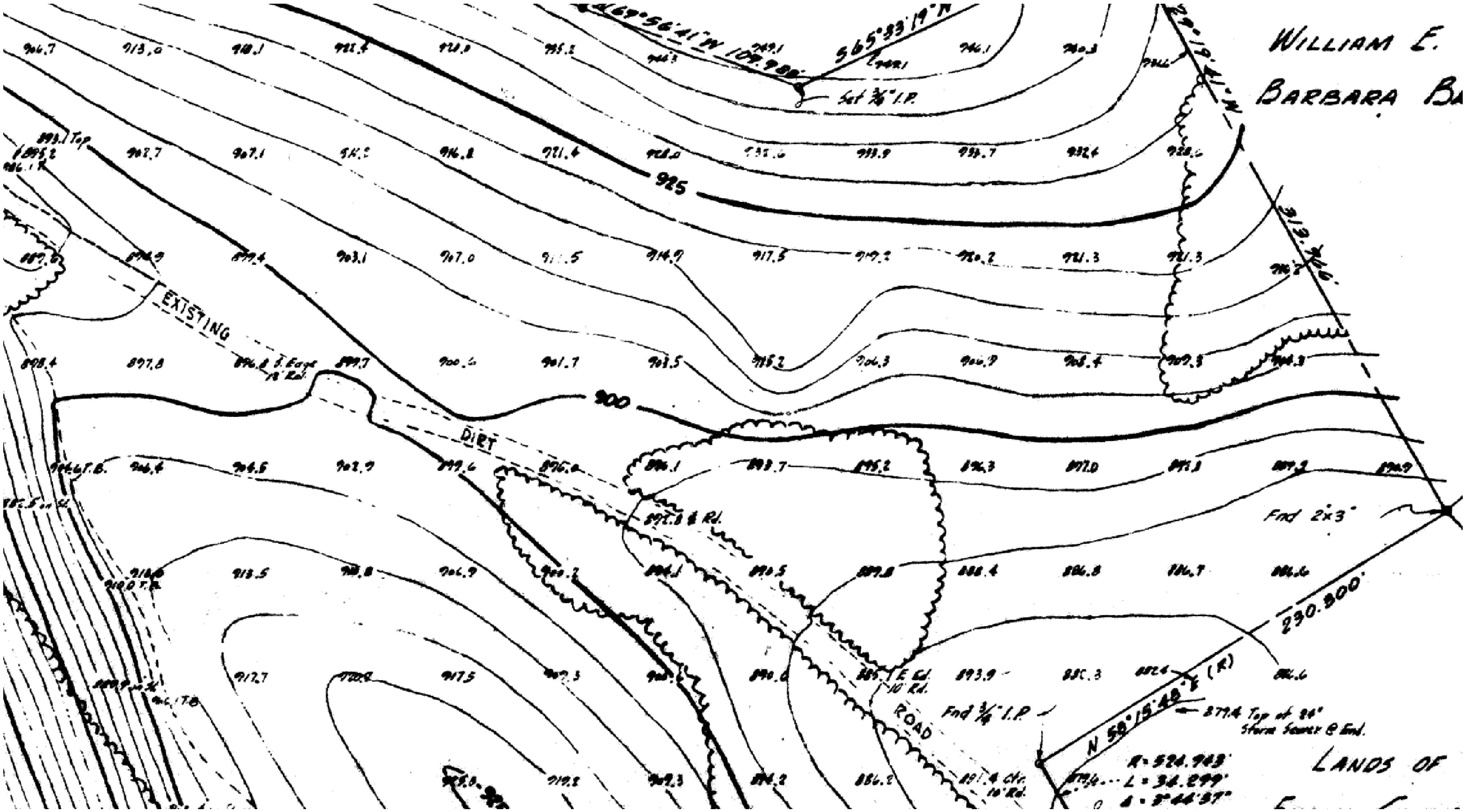
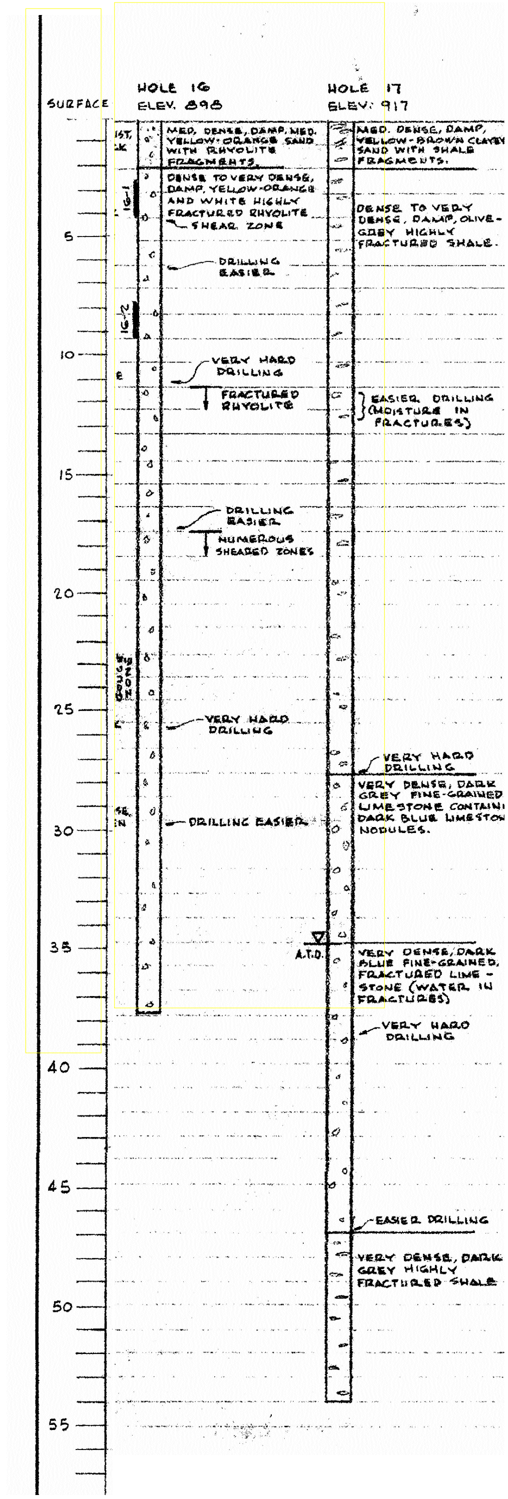


Source: Frahm, Mayke & Elder, September 1961

N

0 18.8 37.5 75.0
Approximate Scale in Feet

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DISTR	PRE-DEVELOPMENT TOPOGRAPHY
terraphase engineering	PROJECT: HORTICULTURAL CENTER	
	PROJECT NUMBER: 0034.005.0003	Figure 4



SAMPLE 16-1

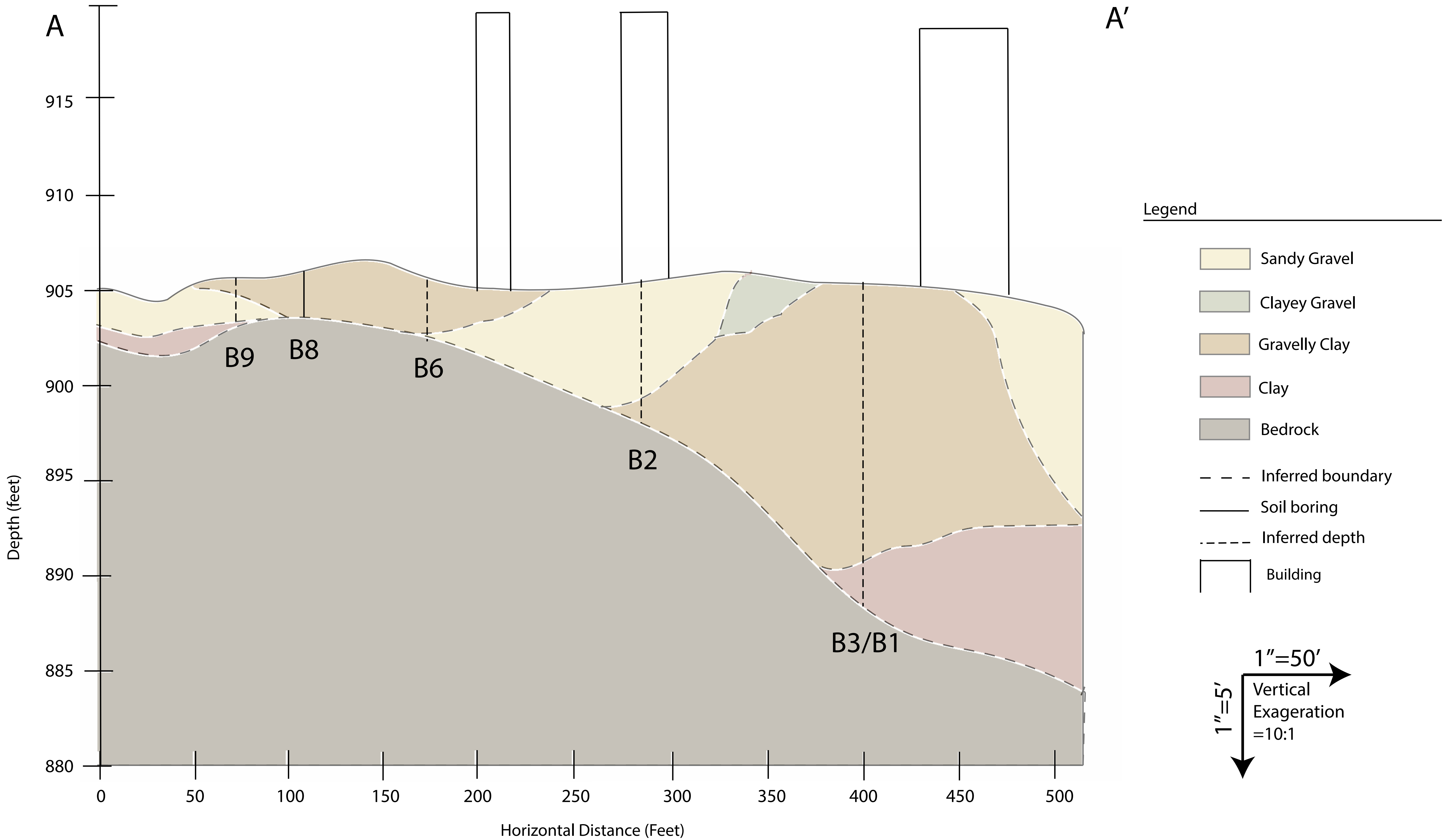
50 BLOWS/FT.
WC - 14
DD - 95
UN - 1310

SAMPLE 16-2

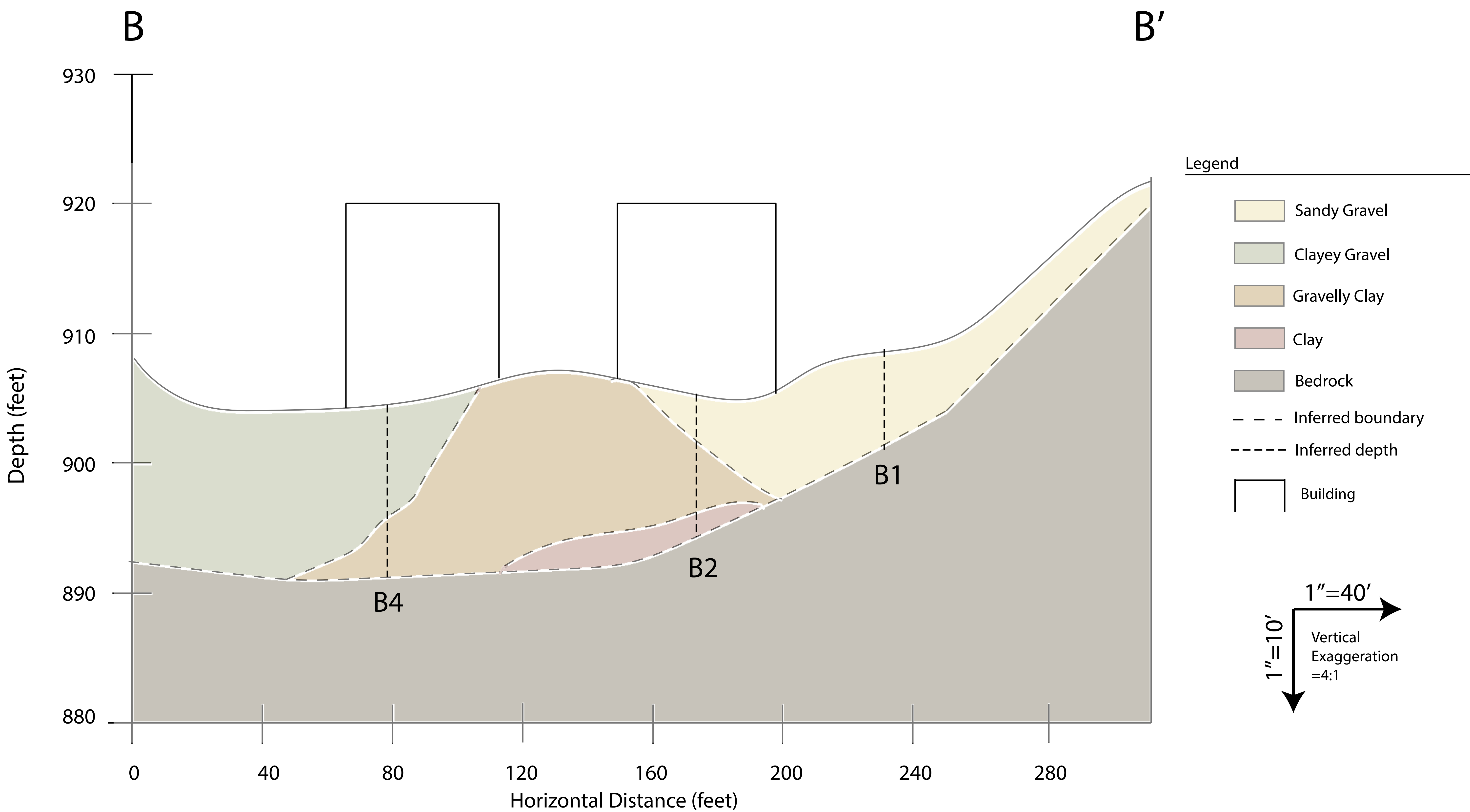
55 BLOWS/FT.
WC - 29
DD - 95
UN - 8800

Source: Woodward Clyde 1962

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DISTR	WOODWARD CLYDE BORING LOGS
terraphase engineering	PROJECT: HORTICULTURAL CENTER	
	PROJECT NUMBER: 0035.005.0003 / DATE: 4-20-20	Figure 5



SAFETY FIRST	CLIENT:	CROSS-SECTION A A′
terraphase engineering	PROJECT: Merritt College Oakland, California	
	PROJECT NUMBER: 0034.011.001	Figure 6



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SAFETY FIRST	CLIENT:	CROSS-SECTION B B'
terraphase engineering	PROJECT: Merritt College Oakland, California	
	PROJECT NUMBER: 0034.011.001	Figure 7

National Flood Hazard Layer FIRMette

FEMA

37°47'39.75"N

122°10'20.26"W

122°9'42.80"W

37°47'11.31"N

Feet 1:6,000

0 250 500 1,000 1,500 2,000

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- Coastal Transect
- Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

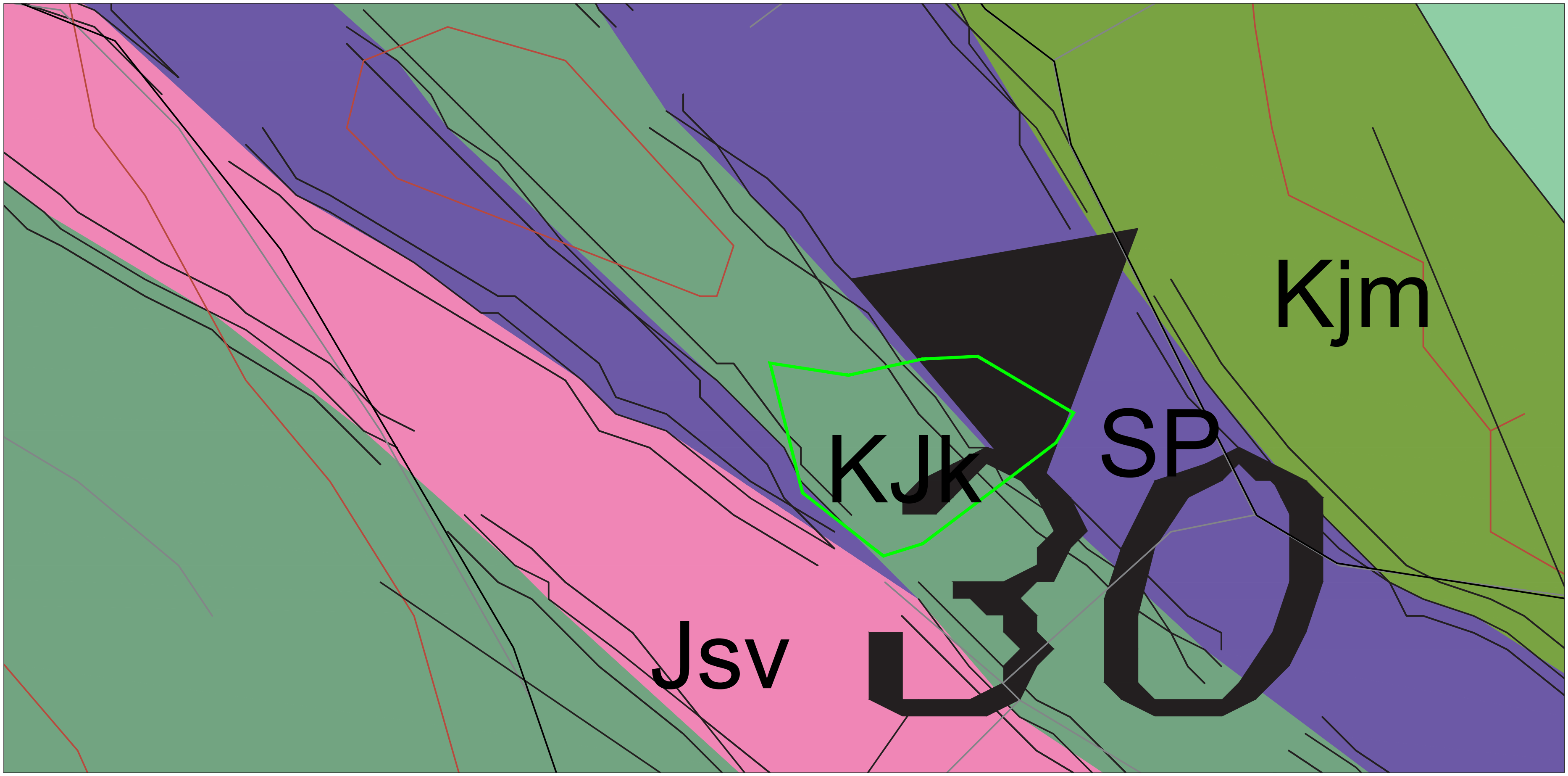
The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 8/12/2019 at 12:20:01 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DIST	**FEMA 100-Year Flood Zones**
terraphase engineering	PROJECT: MERRITT COLLEGE HORTICULTURE	
	PROJECT NUMBER: 0034.011.0001	**Figure 8**



Jsv - Keratophyre and quartz keratophyre

SP - Serpentinite

Kjk - Knoxville Formation - sandstone and shale

Site Extent of Paved Area

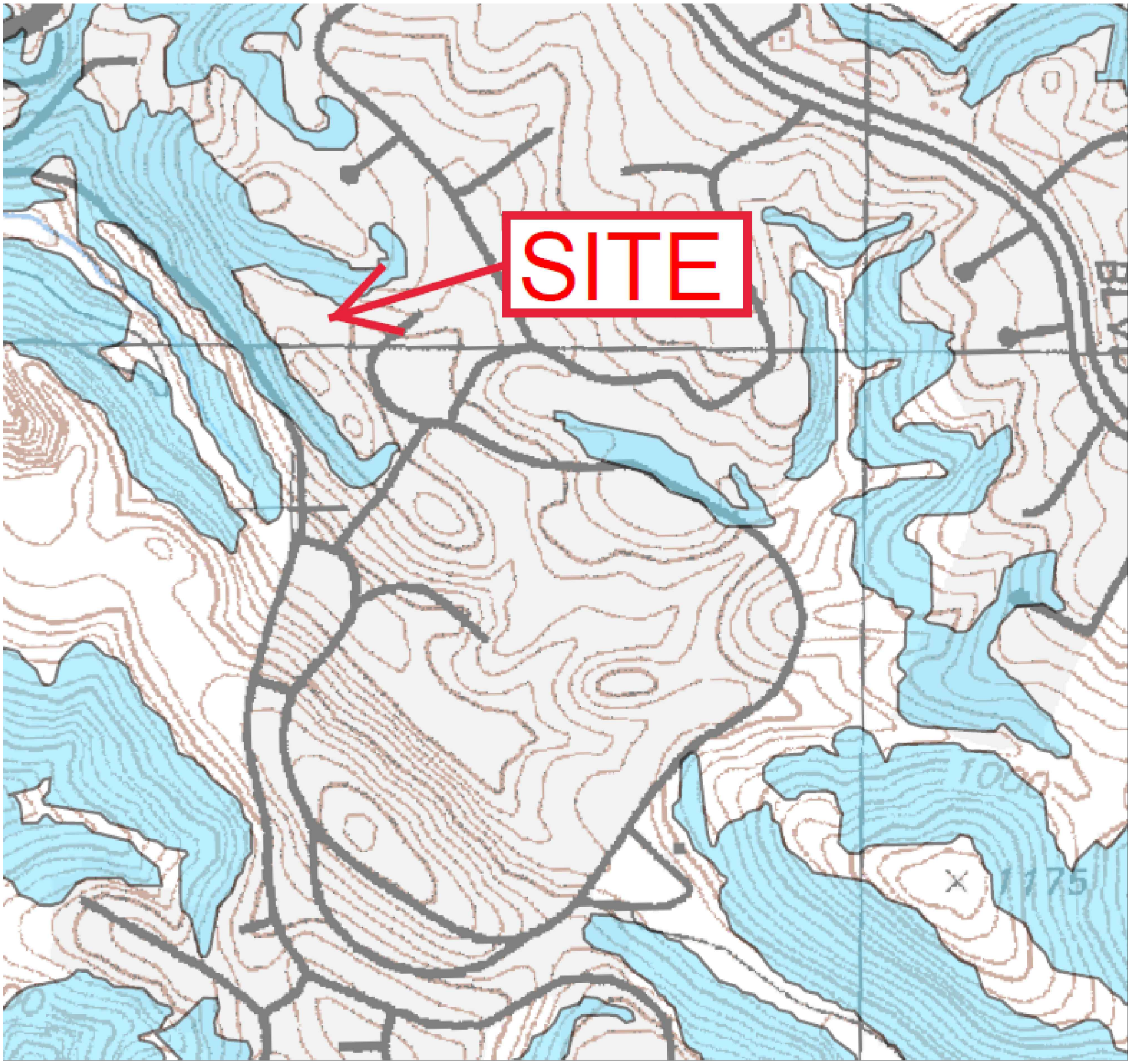
Kjm - Joaquin Miller Formation

N

0 50 100 200
Approximate Scale in Feet

Source: Graymer 2000

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DIST	GEOLOGICAL MAP
terraphase engineering	PROJECT: MERRITT COLLEGE HORTICULTURE	
	PROJECT NUMBER: 0034.011.0001	Figure 9



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

1906 C ? ?

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

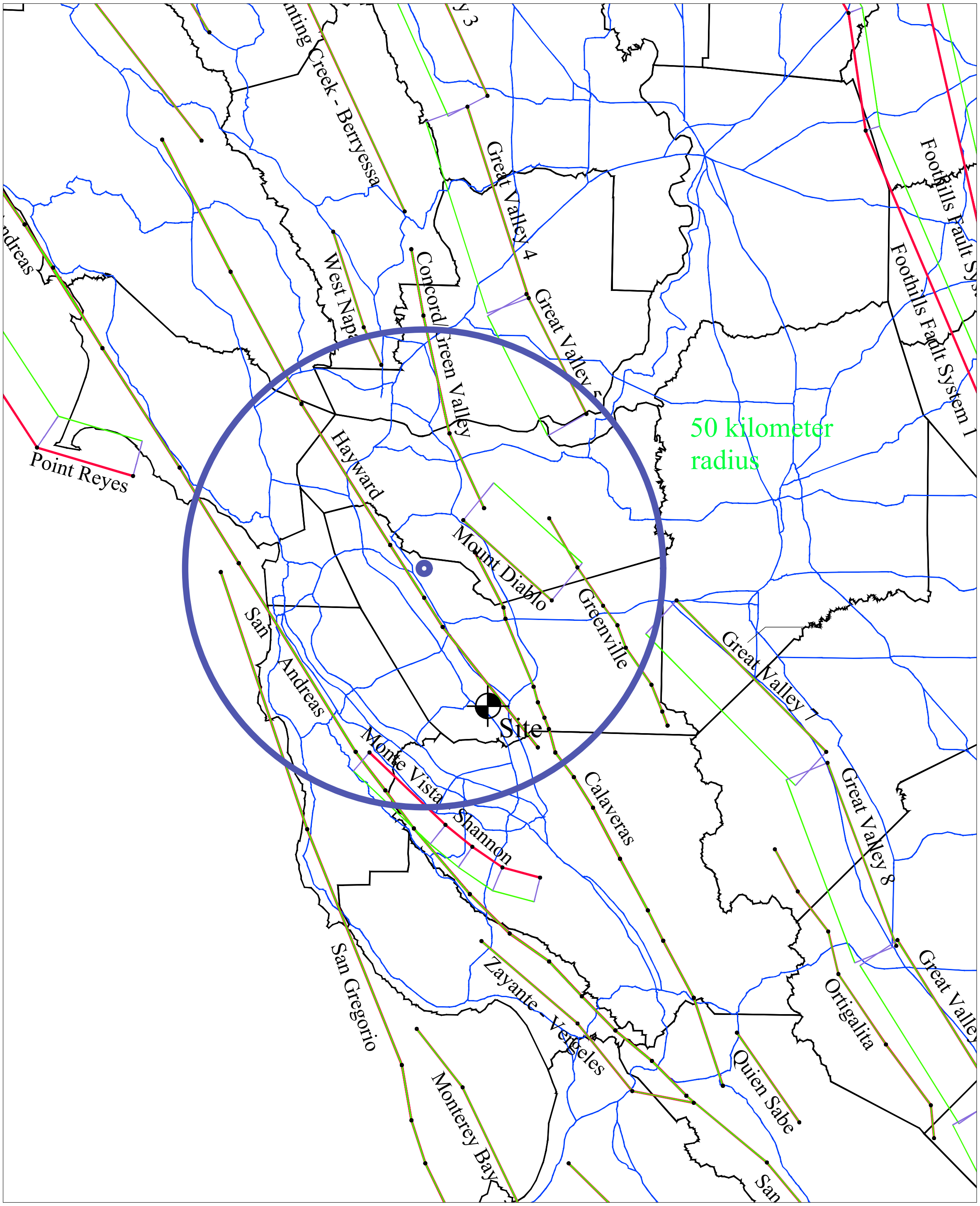
Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

Source: California Geological Survey

SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DISTR		**SEISMIC HAZARD ZONE MAP**
terraphase engineering	PROJECT: HORTICULTURE COMPLEX		
	PROJECT NUMBER: 0035.005.0003	DATE: 9-4-19	**Figure 10**

J:\CADD FILES\0034 Peralta\Merritt College\fIGURE 8 SEISMIC HAZARD HORT CENTER.dwg DRAWN BY: ---; CHECKED BY: ---



County Line	Major Road	Source: Cao et al, 2003
Fault Bend	Fault Trace	Fault Plane Bottom Projected to Surface

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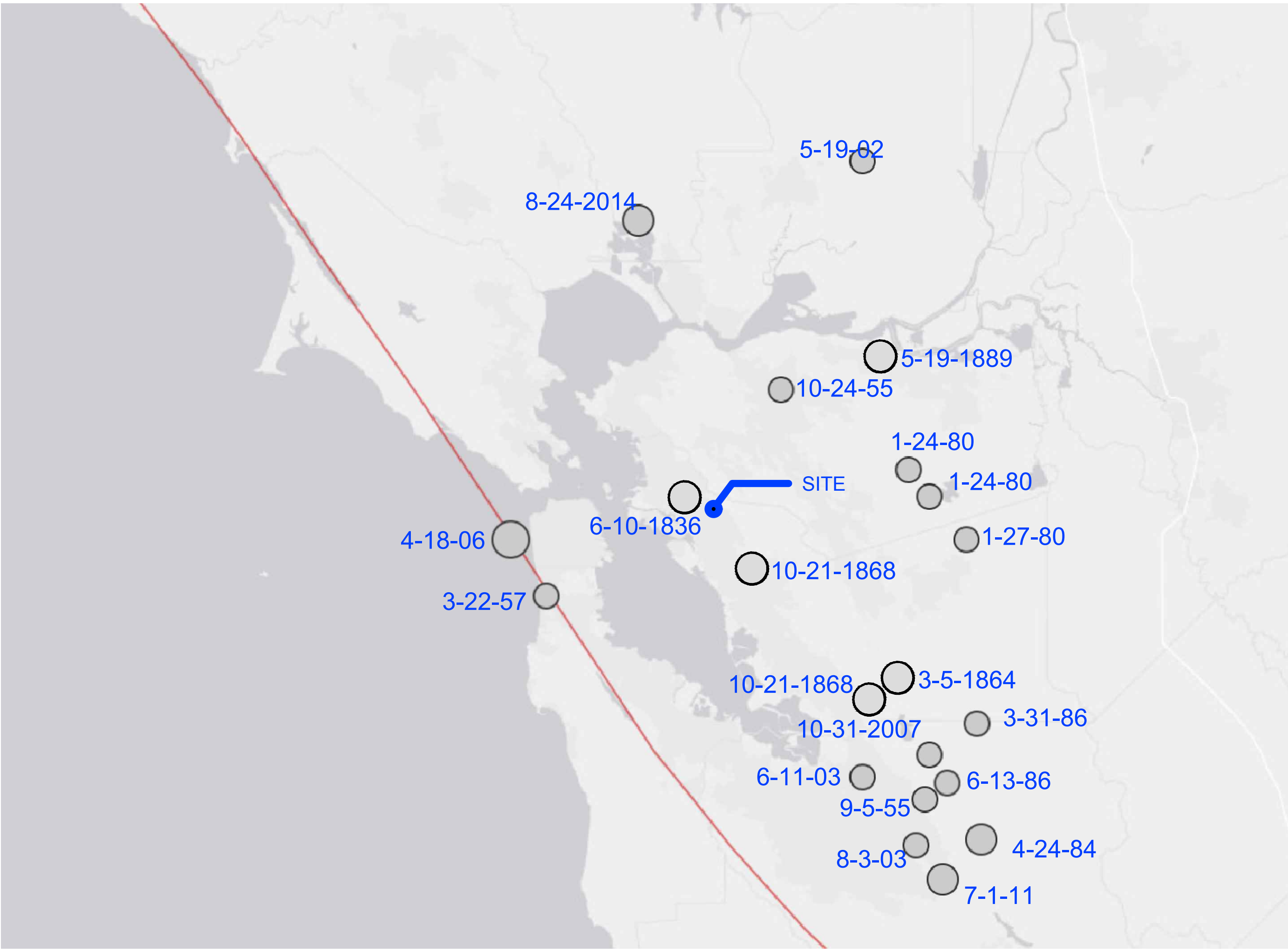
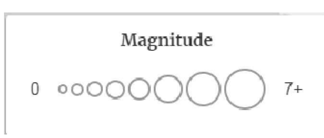
SAFETY FIRST	CLIENT: Peralta Community College District	**EARTHQUAKE FAULT MAP**
terraphase engineering	PROJECT: Child Development Center	
	PROJECT NUMBER: 0034.005.0001	**Figure 11**



Source:
https://earthquake.usgs.gov/earthquakes/search/



SAFETY FIRST	CLIENT: Peralta Community College District	**HISTORICAL EARTHQUAKES**
terraphase engineering	PROJECT: Merritt Horticultural Complex	
	PROJECT NUMBER: 0034.011.0001	**Figure 12**

APPENDIX A
BORING LOGS

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Project: **Merritt College Horticultural Center**	# Log of Boring B-1
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 1 of 1**
Project Number: **0034.004.004**	

Date(s) Drilled **06/04/2020**	Logged By **Jeff Raines**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **10 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **Hand Auger, SPT**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0							10YR-7/2	SANDY GRAVEL, dry, no dry strength, ~60% gravel, ~30% sand, ~10% fines	
		34 52 to 4"							
5					Shale		GLEY 1-3/10GY	SHALE/MUDSTONE, highly weathered, crumbles easily	
10								Bottom of boring = 10 feet bgs	
15									
20									

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Project: **Merritt College Horticultural Center**	Log of Boring B-2
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 1 of 1**
Project Number: **0034.004.004**	

Date(s) Drilled **06/04/2020**	Logged By **Jeff Raines**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **12.5 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **California, Hand Auger**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					GM			Cuttings become clayier between 5 and 10 feet bgs SILTY GRAVEL, damp	
5		5 5 5							
10		7 5 4		0/18				PEA GRAVEL with SILT, no recovery	
12.5								Bottom of Boring = 12.5 feet bgs. BEDROCK, auger scraping, no recovery	
15									
20									

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Project: **Merritt College Horticultural Center**	# Log of Boring B-3
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 1 of 2**
Project Number: **0034.004.004**	

Date(s) Drilled **06/04/2020**	Logged By **Jeff Raines / Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **21.5 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **California, SPT**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					Asphalt			Asphalt, 2-inches	
					CL			GRAVELLY CLAY, damp, grey with orange mottles	
	California	7 5 5					10YR-3/1	Sticky damp dark reddish gray clay with gravel that is fine and angular up to 0.75-inch, ~40% gravel, 60% fines, at 3 feet bgs 2-inch gravel at 4 feet bgs	
5	SPT	3 4 6			CL			CLAY with GRAVEL, olive, stiff, trace organics (very fine roots), 30% gravel, 70% fines, bedrock?	
10	SPT	6 10 9						Wet, olive brown, with fine gravel, ~30% gravel, 70% fines, at 10 feet bgs. Auger making scraping noise - rouned 1-inch gravel in auger cuttings from 10 to 15 feet bgs.	
15	SPT	3 7 10			CL		GLEY 1-2.5/10Y	CLAY, wet, greenish black, soft to stiff, low to medium plasticity, no sand	
20									

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Project: **Merritt College Horticultural Center**	Log of Boring B-3
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 2 of 2**
Project Number: **0034.004.004**	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
20		67 for 5"			Bedrock			BEDROCK - chert and medium fine sand	
								Bottom of Boring = 21.5 feet bgs	
25									
30									
35									
40									
45									
50									

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Project: **Merritt College Horticultural Center**	**Log of Boring B-4**
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 1 of 1**
Project Number: **0034.004.004**	

Date(s) Drilled **06/04/2020**	Logged By **Jeff Raines / Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **19.5 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **California, Hand Auger, SPT**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0				GC	7YR-3/2	CLAYEY GRAVEL, damp, dark brown, angular to subrounded, fine, poorly graded	
3	7 7 6					Some 1.5-inch gravels	
5				CL		GRAVELLY CLAY, damp, some water in gravel fractures	
8	4 9 6						
10							
13	5 11 14	3.0			10YR-3/4	Stiff, dark yellowish brown, mostly olive brown mottles with gray inclusions at 13 feet bgs	
15							
17	5 9 20	2.5			10YR-3/4	Clay, moist, dark yellowish brown, weathered with angular pebbles, driller reports very hard drilling at 17 feet bgs	
19	56 for 6"			Bedrock		BEDROCK - Driller reports really hard at 19 feet bgs	
19.5						Bottom of boring = 19.5 feet bgs	
20							

terraphase engineering

Project: **Merritt College Horticultural Center**	# Log of Boring B-5
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 1 of 1**
Project Number: **0034.004.004**	

Date(s) Drilled **06/04/2020**	Logged By **Jeff Raines / Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **11.5 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **California, Hand Auger, SPT**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					GC			CLAYEY GRAVEL, brown, 1-inch rounded gravel	
		5 8 8	2.2		CL		10YR-4/4 & GLEY 1-2.5/10Y	CLAY, few gravel, dark yellowish brown and greenish black, low plasticity	
5									
		7 29 40			Bedrock			BEDROCK, weathered	
10		11 25 23							
								Bottom of Boring = 11.5 feet bgs	
15									
20									

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Project: **Merritt College Horticultural Center** Project Location: **12500 Campus Dr, Oakland, CA 94619** Project Number: **0034.004.004**	Log of Boring B-6 **Sheet 1 of 1**

Date(s) Drilled **06/04/2020**	Logged By **Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **6 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **California, Hand Auger, SPT**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					CL		7.5YR-4/3	GRAVELLY CLAY with SAND, damp, brown, low plasticity, angular to subangular gravel, fine grained poorly graded sand, organic material, ~20% gravel, 10% sand, 70% fines	
								Light brown clay at 2.7 feet bgs	
		15 21 29							
5		11 25 23			Bedrock			BEDROCK, chert/mudstone with fine grained sandy clay, weathered shale?	
								Bottom of Boring = 6 feet bgs	
10									
15									
20									

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Project: **Merritt College Horticultural Center** Project Location: **12500 Campus Dr, Oakland, CA 94619** Project Number: **0034.004.004**	Log of Boring B-7 **Sheet 1 of 1**

Date(s) Drilled **06/05/2020**	Logged By **Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **3.75 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **Hand Auger**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					SM		5YR-4/3	SILTY SAND, reddish brown, loose, organic materials	
3.75								Bottom of Boring = 3.75 feet bgs	
5									
10									
15									
20									

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terraphase engineering

Project: **Merritt College Horticultural Center**	Log of Boring B-8
Project Location: **12500 Campus Dr, Oakland, CA 94619**	**Sheet 1 of 1**
Project Number: **0034.004.004**	

Date(s) Drilled **06/05/2020**	Logged By **Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **3 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **Hand Auger**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					Fill			SANDY CLAY, damp, with angular to subangular loose well graded gravel	
		20 28 39			Bedrock			BEDROCK, not as weathered as borings B-1 through B-6	
								Bottom of Boring = 3 feet bgs	
5									
10									
15									
20									

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terraphase engineering

Project: **Merritt College Horticultural Center** Project Location: **12500 Campus Dr, Oakland, CA 94619** Project Number: **0034.004.004**	Log of Boring B-9 **Sheet 1 of 1**

Date(s) Drilled **06/05/2020**	Logged By **Cass Wolf**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch Hollow Stem Auger**	Total Depth of Borehole **13.5 feet bgs**
Drill Rig Type **Mobile B80**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation
Groundwater Level and Date Measured	Sampling Method(s) **Hand Auger, SPT**	Hammer Data **140 pound hammer, 30-inch fall**
Borehole Backfill **Cement Grout**	Location	

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
0					Fill			GRAVELLY SILT	
		8 10 16			CL		5YR-4/3	SILTY CLAY, damp, reddish brown, stiff, organic material	
5		8 8 12	>4.5						
10		8 11 19			Bedrock			BEDROCK, weathered	
		13 26 37							
								Bottom of Boring = 13.5 feet bgs	
15									
20									

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Project: **Merritt College Horticultural Center**
Project Location: **12500 Campus Dr, Oakland, CA 94619**
Project Number: **0034.004.004**

Key to Log of Boring

Sheet 1 of 1

Depth (feet)	Sample Type	Sampling Resistance (blows/foot)	Pocket Penetrometer (tsf)	Recovered (in) / Total (in)	USCS Symbol	Graphic Log	Munsell Soil-Color	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10

COLUMN DESCRIPTIONS

1 Depth (feet): Depth in feet below the ground surface.

2 Sample Type: Type of soil sample collected at the depth interval shown.

3 Sampling Resistance (blows/foot): Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.

4 Pocket Penetrometer (tsf): Hand-held pocket penetrometer used to determine the unconfined compressive strength of cohesive soils. Displayed in units of tons per square foot (tsf).

5 Recovered (in) / Total (in): Inches of recovery / total inches of boring tubing.

6 USCS Symbol: USCS symbol of the subsurface material.

7 Graphic Log: Graphic depiction of the subsurface material encountered.

8 Munsell Soil-Color: Color of subsurface material according to Munsell soil-color charts.

9 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.

10 REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)

AF

TYPICAL SAMPLER GRAPHIC SYMBOLS

Hand auger sampler

2-inch-OD unlined split spoon (SPT)

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)

Water level (after waiting)

Minor change in material properties within a stratum

Inferred/gradational contact between strata

? Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

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APPENDIX B
LABORATORY RESULTS

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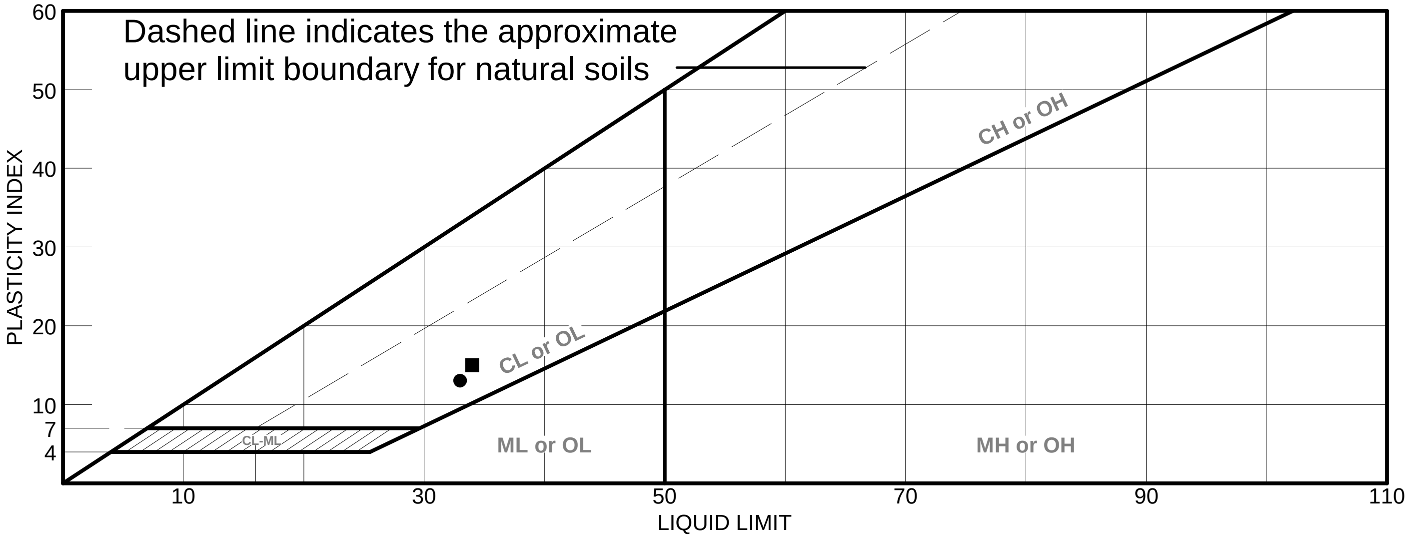
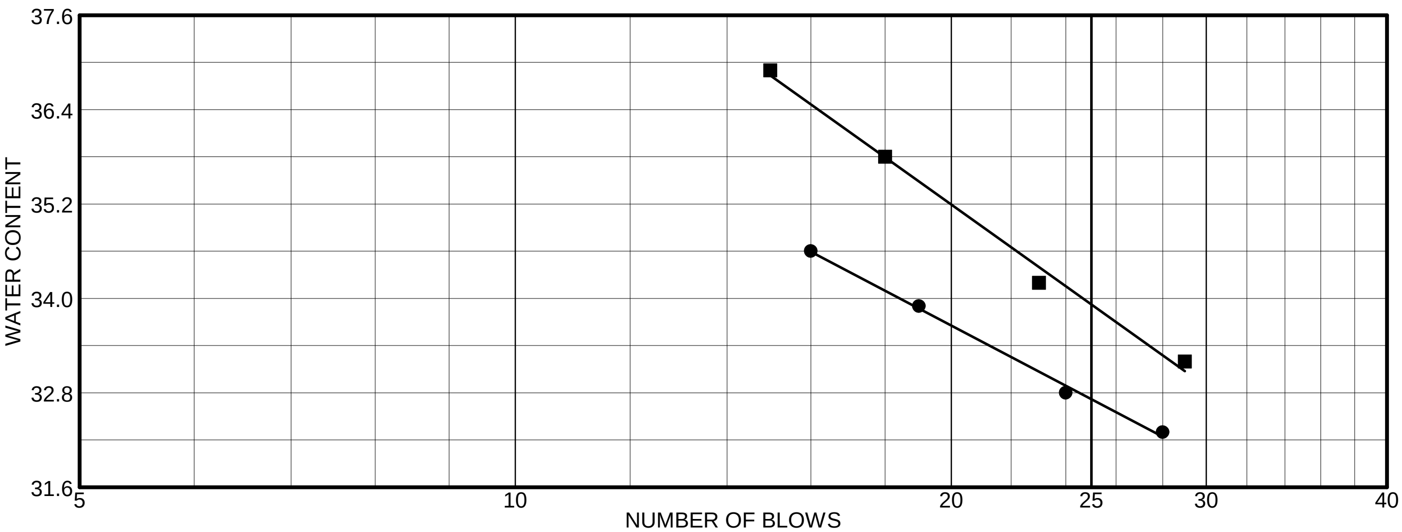
LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Yellowish Brown Sandy Lean CLAY	33	20	13			
■	Brown GRAVEL w/ Lean Clay & Sand	34	19	15			

Project No. 734-077 **Client:** Terraphase Engineering

Project: Merritt College - 0034.011.001

● **Source:** B-9 **Elev./Depth:** 4-6'

■ **Source:** B-8 **Elev./Depth:** 2'

Remarks:

●

■

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

#200 Sieve Wash Analysis
ASTM D 1140

COOPER TESTING LABORATORY

Job No.:	734-077	**Project No.:**	0034.011.001	**Run By:**	MD
Client:	Terraphase Engineering	**Date:**	6/29/2020	**Checked By:**	DC
Project:	Merritt College				

Boring:	B-5							
Sample:								
Depth, ft.:	3-4							
Soil Type:	Dark Brown CLAY w/ Sand							
Wt of Dish & Dry Soil, gm	542.2							
Weight of Dish, gm	172.3							
Weight of Dry Soil, gm	370.0							
Wt. Ret. on #4 Sieve, gm	9.8							
Wt. Ret. on #200 Sieve, gm	93.2							
% Gravel	**2.6**							
% Sand	**22.6**							
% Silt & Clay	**74.8**							

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL #	734-077	**Date:**	6/26/2020	**Tested By:**	PJ	**Checked:**	PJ
Client:	Terraphase Engineering	**Project:**	Merritt College			**Proj. No:**	0034.011.001
Remarks:							

Sample Location or ID			Resistivity @ 15.5 ^{o}C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-9	-	4-6	-	3988	-	6	81	0.0081	6.5	-	16.7	Yellowish Brown Sandy Lean CLAY

MICRO ANALYTICAL LABORATORIES, INC.

BULK ASBESTOS ANALYSIS - PLM ARB 435

1225
Jeff Raines
Terraphase Engineering, Inc.
1404 Franklin St, Ste 600
Oakland, CA 94612

PROJECT:
PROJECT NO. 0034.011.0001

Micro Log In **273178**
Total Samples 1
Date Sampled 07/17/2020
Date Received 07/21/2020
Date Analyzed 07/22/2020

SAMPLE INFORMATION	ASBESTOS INFORMATION QUANTITY (AREA %) / TYPES / LAYERS / DISTINCT SAMPLES	DOMINANT OTHER MATERIALS
Client #: 1 Micro #: 273178-01 Analyst: SL BULK Asb. / Total Pts. 0 / 400 Matrix Removed 0% Sensitivity 0.250%	**ND**	2 % CELLULOSE Matrix Type: ROCK FRAGMENTS

Technical Supervisor: _______________ 7/22/2020
Baojia Ke, Ph.D. Date Reported

Analyses use Polarized Light Microscopy (PLM), Micro Analytical SOP PLM-101,Rev.1/4/2013 for building materials (based on EPA-600/R93-116 (1993)), and California ARB 435 (1991) for applicable soil, rock, or aggregate samples. NOTES: Weight % cannot be determined by PLM estimation or point counts. Asbestos fibers with diameter below ~1 μm may not be detected by PLM. The absence of asbestos in dust or debris (including wipe or microvacuum), and in some compact materials, including floor tiles, cannot be conclusively established by PLM, and should be confirmed by Transmission Electron Microscopy (TEM). Only dominant non-asbestos materials are indicated. This report must not be interpreted as a conclusive identification of non-asbestos (fibrous or not). Quantities of non-asbestos fibers are estimated, not point counted. Preparation (all samples): grinding, milling; teasing bundles apart; drying, if needed, by hotplate. Acid dissolution, ashing, or other matrix reduction techniques may be applied to some samples; residue asbestos % is corrected for amount of matrix removed. Various sample interferences may prevent detection of small asbestos fibers, and hinder determination of some optical properties. Notes are made if point counting is used; otherwise, asbestos is quantified by calibrated visual estimation. Detection limit is material dependent. Detection of asbestos traces (<<1%) may not be reliable or reproducible by PLM. Lower quantitation limit (reporting limit) of PLM estimation is 1%. The Cal-OSHA definition of asbestos-containing construction material is 0.1% asbestos by weight; however, reliable determination of asbestos weight percent at this level cannot be done by PLM, and TEM is recommended. Sample heterogeneity is indicated by listing more than one distinct layer or material on the report. Composite asbestos percentages on multilayered samples are applicable only to layered wall systems (wallboard, joint compound, and related materials); compositing is based on clients' descriptions of a material as "joint compound". Clients are solely responsible for identification and description of bulk materials listed on field forms. Laboratory sample descriptions may differ from descriptions given by the client. Quality Control (QC): all results have been determined to be within acceptance limits prior to reporting. Samples that were reanalyzed are denoted by two sets of analyst initials. Unless otherwise stated in this report, all samples were received in acceptable condition for analysis. This report must not be used to claim product endorsement by NIST or any U.S. Government agency. This report shall not be reproduced except in full, without the approval of Micro Analytical Laboratories, Inc., and pertains only to the samples analyzed as received. ND = NO ASBESTOS DETECTED.

APPENDIX C
GEOPHYSICAL SURVEY REPORT

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ADVANCED GEOLOGICAL SERVICES

1605 School Street, #4
Moraga CA 94556
925 (808-8965)

May 21, 2020

Jeff Raines
Terraphase Engineering Inc.
1404 Franklin Street, Suite 600
Oakland, California 94612

Subject: Seismic Survey Report
Merritt College
Oakland, California

Dear Mr. Raines

1.0 INTRODUCTION

This letter presents the results of Advanced Geological Services, Inc. (AGS) seismic refraction and seismic surface-wave (MASW) surveys at Merritt College in Oakland, California (Figure 1). The survey objective was to assess bedrock depth and rippability, along with the average shear-wave velocity of the upper 100 feet of sediment (Vs30) to establish the IBC Seismic Site Class to aid future construction activities in the area.

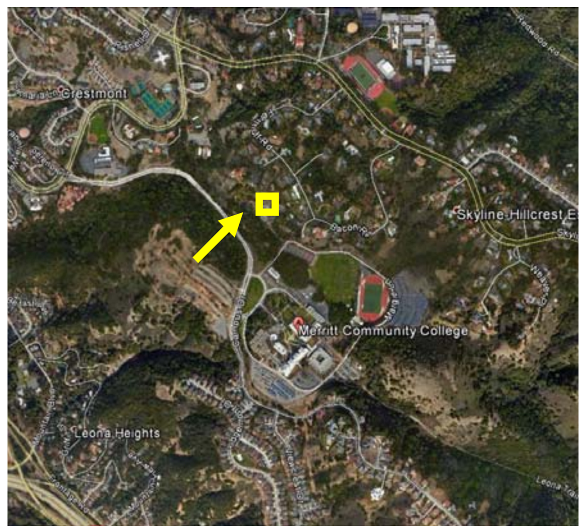


Figure 1 - Seismic Survey Area, Merritt College

The survey was performed on May 13, 2020 by AGS senior geophysicist Roark W. Smith. In general, the survey entailed the collection of seismic data along two perpendicular lines located in the survey area. The refraction data were processed using the SeisImager software program to develop models showing P-wave velocity layering along each seismic line. The surface-wave data were processed with SeisImager/SW, using the multi-channel analysis of surface waves (MASW) processing technique to delineate subsurface velocity layering and assess the average shear-wave velocity of the upper 30 meters/100 feet (V_s30), which determines the site's IBC site classification.

2.0 SUMMARY OF FINDINGS

- Three P-wave velocity layers were detected by the refraction survey. Layer V_1 is the thin uppermost layer representing surficial soil. Layer V_2 may represent fill material and/or weathered bedrock along SL-1 and weathered bedrock along SL-2. Layer V_3 represents little-weathered bedrock; it occurs at depth ranging from 1 to 15 feet bgs along SL-1 and about 4 feet bgs along SL-2 and exhibits a P-wave velocity of about 3,750 fps along SL-1 and 3,680 fps along SL-2.

- On the basis of the Caterpillar Performance Handbook "rip chart", the subsurface material the

Merritt College site should be rippable down to at least 20 feet bgs.

- The refraction data also indicate that one or more large substructures (e.g., sewer pipe or concrete foundation remnant) may be present in the survey area.

- On the basis of the MASW survey, Vs30 along seismic line SL-1 is 2,234 feet per second, which equates to Seismic Site Class C ("very dense soil and soft rock"). The MASW results also indicate a subsurface layer boundary at about 22 feet bgs.

3.0 SITE DESCRIPTION

The investigation was performed in the Landscape Horticulture area in the northern portion of the Merritt College campus. One seismic line was placed on an asphalt-paved roadway and a second line, oriented roughly perpendicular to the first line, was run northward up the adjacent grassy hillside (Figure 2).

4.0 SEISMIC REFRACTION OVERVIEW

The seismic refraction method uses compressional (P-) wave energy to delineate seismic velocity layers within the subsurface. Interpretation entails correlating the velocity layers to geologic features such as soil and various types of bedrock. To perform a refraction survey, an elastic wave (compressional, or P-wave) is generated at certain locations ("shotpoints") along a survey line. The P-wave energy is usually produced by striking the ground with a sledgehammer. As the P-wave propagates through the ground it is refracted along boundaries between geologic layers exhibiting different seismic velocities.

Part of the refracted P-wave energy returns to the ground surface where it is detected by vibration-sensitive devices called geophones, which are placed in a co-linear array along the seismic survey line. The geophone data are fed to a seismograph, where they are recorded, and then to a computer, where they are analyzed to determine the depth and velocities of subsurface seismic layers. Key data for refraction analysis are the positions of the geophones and shotpoints along a seismic line, and the amount of time it takes for the refracted wave to travel from the shotpoint to each geophone location. Because the P-wave is the fastest traveling of all types of seismic waves, it can be readily identified as the first deflection ("first break") on a seismic trace.

Additional discussion of the refraction method, its limitations, and the relationship between seismic velocity and geologic materials is presented in Appendix A.

5.0 SEISMIC SURFACE-WAVE (MASW) OVERVIEW

The Seismic Surface-Wave method entails the use of data processing techniques known as Spectral Analysis of Surface Waves (SASW), Multi-channel Analysis of Surface Waves (MASW), and/or Refraction Micro-tremor (REMI). Surface-wave surveys use essentially the same field set-up as a conventional seismic refraction survey (i.e., a geophone array), but a different part of the recorded seismic signal— the Rayleigh (surface) wave— is analyzed instead of the P-wave. Briefly, a surface-wave survey entails measuring the velocity of surface waves using an array of motion detectors (geophones) placed on the ground surface. Because surface-wave velocity closely follows shear-wave velocity (90 to 95% of V_S), surface-wave velocity data can be used to estimate shear wave velocity (V_S).

Surface-Waves are seismic waves that travel along or near the surface of the earth; they can be "active-source" waves generated specifically for the seismic survey (e.g., with explosives or a hammer blow to the ground surface) or "passive-source" waves generated by ambient natural and cultural sources such as ocean waves and vibrations from vehicle traffic and factories. In general, active-source waves are of higher frequency and provide information about the shallower subsurface, while passive-source waves are of lower frequency and can provide deeper subsurface information, albeit with lower resolution. MASW or SASW surveys use active-source surface waves, usually generated with a sledgehammer. Refraction Micro-tremor, or REMI surveys use ambient surface waves.

Surface-Waves travel in assemblages of frequencies, with each frequency having a corresponding wavelength. Because surface-waves are influenced by subsurface material to a depth approximately equal to the surface-wave's wavelength, a velocity vs. depth profile can be generated by measuring the velocity of surface-waves of varying wavelengths. Surface waves with shorter wavelengths (higher frequencies) respond to the material properties (e.g., stiffness) of shallower materials while waves with longer wavelengths (lower frequency) respond to deeper materials.

Specialized computer software is used to identify surface-waves in the recorded data and prepare a 'velocity spectrum' image, which the geophysical analyst interprets to produce a 'dispersion curve' that depicts how velocity varies with frequency (hence, depth). The dispersion curve is then used to prepare a model depicting subsurface velocity layering at a point that is taken to be at the center of the geophone array. Surface-wave surveys produce a 1-dimensional (1-D) profile showing S-wave velocity variations with depth at a point that is taken to be at the center of the geophone array.

6.0 FIELD PROCEDURES

AGS installed two seismic lines for this investigation as directed by Terraphase— one line (SL-1) was on the asphalt paved roadway and the second line (SL-2) was placed on the adjacent grassy hillside, roughly perpendicular to the first line (Figure 2). As dictated by available space and the site conditions, each seismic line comprised an array of geophones spaced five (5) feet apart for total lengths of 115 and 90 feet, for SL-1 and SL-2, respectively. For line SL-1 (on pavement) AGS replaced the spikes on the bases of the geophones, which are normally used to couple the geophones to soil, with metal base plates.

Both refraction and MASW data were obtained along SL-1, while only refraction work was done along SL-2. Three shotpoints were used for the refraction survey, while one shotpoint was used for the MASW survey. For the refraction survey, shotpoints were located five feet off each end of the geophone array and a third shotpoint was placed in the middle of the array. For the MASW survey, one shotpoint located off the west end of the geophone array, 10 feet from the nearest geophone, was used.

AGS generated seismic energy through multiple impacts with a 16-lb sledge hammer against a metal plate placed on the ground surface at each shotpoint location. In general, 5 hammer blows were struck at the end shots and 3 blows were struck at the center shotpoint, a technique called "stacking," which is used to increase the signal-to-noise ratio and thus improve data quality.

The seismic waves were detected using 4.5-Hz geophones from GeoSpace Corp and recorded by a DAQLink II seismic system connected to a laptop computer. The data were recorded for 2 seconds using a 0.125 millisecond (ms) sample rate. After the seismic data were acquired AGS mapped the seismic line locations by referencing them to distinctive site features such as the retaining wall, building corner, light standard and storm drain lid.

7.0 DATA PROCESSING AND ANALYSIS

7.1 Seismic Refraction

Seismic data were transferred from the seismograph to a desktop computer where they were processed using the *SeisImager* software package by Geometrics, Inc. Briefly, *SeisImager* is a computer inversion program that generates an initial velocity layer model, produces synthetic data from the model, and then adjusts the model so that the synthetic data better matches the observed field data. The agreement between the synthetic and observed data provides an indication of how well the model represents the true subsurface conditions.

First, AGS used the *SeisImager* module *PickWin* to interpret ("pick") the P-wave arrivals ("first breaks") for each of the shotpoint data sets ("shot gathers") per line. *PickWin* was also used to check (against the geophysicist's field log) that the proper locations were assigned to the geophones and shotpoints. Next, the first break files were fed to the SeisImager module *PlotRefra*, which was used review time-distance (TD) plots for the seismic lines and assign a seismic layer to each arrival time. For the refraction analysis, each P-wave arrival is considered to have refracted from a distinct seismic layer.

The number of layers resolved by the seismic survey, and their thickness and average velocity, is indicated by straight line segments on the TD plot; because these straight-line segments represent a constant velocity condition within the subsurface, the tend to represent a distinct geologic layer. The topographic elevation files were incorporated into the analysis at this point. Next, a time-term inversion was performed to produce layered velocity models presented on Figure 3. Time-term inversion is a linear least-squares technique that uses the layer assignments and the distances and travel times between the shotpoints and the geophones to develop a velocity layer model that best fits the observed data.

7.2 Seismic Surface-Wave (MASW)

MASW data processing was performed using the *SeisImager/SW* software package by Geometrics, Inc. In general, surface wave data processing entails first producing a velocity spectrum image, which shows the phase velocity for the various frequencies of surface waves detected. This image is used as the basis for interpreting ("picking") a dispersion curve, which is a graph that depicts how surface-wave velocity varies with frequency (hence, with depth). The dispersion curve is then used to prepare an initial 1-D model of surface-wave velocity versus depth using a one-third wavelength approximation (i.e., a given phase velocity is assigned to a depth that is one-third of the wavelength of the corresponding surface-wave). The initial velocity layer model is then adjusted using an inversion process until the corresponding synthetic dispersion curve achieves a "best-fit" match to the original dispersion curve (the one that was interpreted from the observed data— i.e., the velocity spectrum image). The degree or closeness of the fit between the interpreted and synthetic curves (expressed as a RMS percentage error) provides an indication of how well the model represents actual subsurface conditions.

Separate, independent processing was performed for each of the surface-wave data sets to produce the velocity spectrum images from which dispersion curves were picked. The curves were then inverted to produce a velocity layer model that depicts S-wave velocity variations with depth at a single point, which is taken to be at the center of the geophone array (Figure 3).

8.0 RESULTS

The investigation results are presented on Figures 2 and 3 and are summarized on Table 1, below. Figure 2 shows the seismic line locations. Figure 3 presents the seismic refraction investigation results in the form of P-wave velocity layer models depicting compressional- (P-) wave velocity layering along seismic lines SL-1 and SL-2; Figure 3 also presents the S-wave velocity layer model and associated velocity spectrum image generated from the surface-wave data obtained along line SL-1. Table 1 summarizes the P- and S-wave velocity layering, rippability, and IBC Seismic Site Class for the investigation area.

Overall, three P-wave velocity layers were detected by the refraction survey; they are designated, from shallowest to deepest, as Layer V_1, Layer V_2, and Layer V_3. Layer V_1 is the thin uppermost layer representing surficial soil; it exhibits velocities of about 1,650 fps along SL-1 and 880 fps along SL-2. Although Layer V_1 is shown only in the eastern portion of the layer model for SL-1 (Figure 3), it is likely that V_1 extends along the entire length of the seismic line but is only a foot or two thick, which is too thin to be detected with the 5-foot geophone spacing used.

Layer V_2 may represent fill material or weathered bedrock along SL-1 and weathered bedrock along SL-2. The thick lens of V_2 (green area) near the center of SL-1 may represent fill material or a localized zone of weathered bedrock. Layer V_3 represents little-weathered bedrock; it occurs at depths ranging from 1 to 15 feet bgs along SL-1 and about 4 feet bgs along SL-2 and exhibits a P-wave velocity of about 3,750 fps along SL-1 and 3,680 fps along SL-2.

On the basis of the Caterpillar Performance Handbook “rip chart”, “metamorphic rock” exhibiting P-wave velocities less than 7,200 fps can be considered “rippable”; accordingly, the subsurface material the Merritt College site should be rippable down to at least 20 feet bgs. On the basis of the MASW survey, Vs30 along seismic line SL-1 is 2,234 feet per second, which equates to Seismic Site Class C (“very dense soil and soft rock”). The MASW results also indicate an S-wave velocity layer boundary at about 22 feet bgs.

With respect to the refraction data, it is worth noting that high-velocity “early arrivals” (i.e., sets of P-wave arrivals within the first 10 milliseconds that exhibited unrealistic high-velocity trends of 20,000 fps or greater) were observed on some of the shot gathers (Figure 2, right). AGS believes these early arrivals are associated with one or more nearby substructures (e.g., buried foundation or large concrete sewer pipe) and are not associated with soil and bedrock. As such, AGS did not “pick” these P-wave arrivals and instead picked P-wave arrival sets that

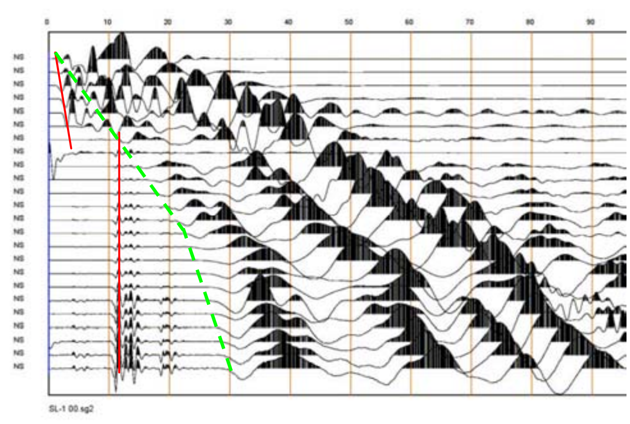


Figure 2 Example "Early Arrival" noise (red lines) on SL-1. (Interpreted refracted arrivals marked by dashed green line)

were later in time and exhibited more realistic P-wave velocities; however, it is recognized that these early "substructure arrivals" may have obscured useable P-wave arrivals associated with shallow bedrock and thus may have reduced the accuracy of the resulting subsurface layer models.

Table 1 Summary of Merritt College Seismic Survey Results

Seismic Line No.	Layer Vp2* (fps)	Layer Vp3 (fps)	Depth to Vp3 (ft)	Vs30, Site Class	MASW Results-Remarks	Site Conditions
SL-1	3,080	3,750	1 to 15	2,234, **C**	Good, coherent Velocity Spectrum Image (VSI)	Asphalt-paved road
SL-2	2,055	3,680	4	NA	NA	grassy hill

***Table 2 Rip Chart for "Metamorphic" Rock** (from The Caterpillar Performance Handbook, 12th Edition)*

Ripper	Rippable	Marginally Rippable	Non-Rippable
D9R	less than 7,200	7,200 to 9,200	greater than 9,200

Table 3 Site Class Definitions (from ASCE 7-02 and ASCE 7-05))

Site Class	Soil Profile Name	Soil Shear Wave Velocity (fps)
A	Hard Rock	>5,000
B	Rock	2,500 < 5,000
C	Very Dense Soil and Soft Rock	1,200 < 2,500
D	Stiff Soil	600 < 1,200
E	Soft Clay Soil	<600

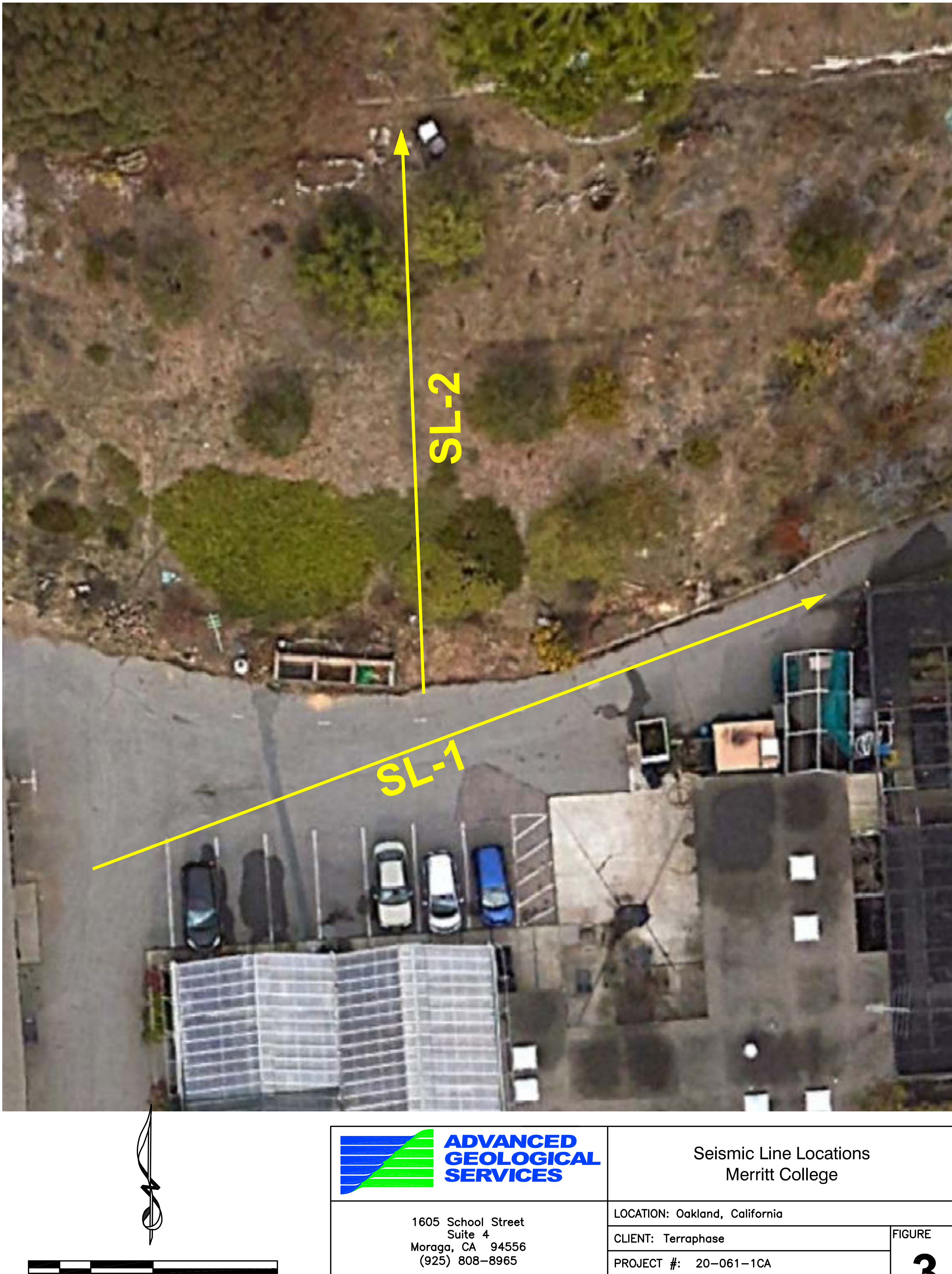
9.0 CLOSING

All geophysical data and field notes collected as a part of this investigation will be archived at the AGS office. The data collection and interpretation methods used in this investigation are consistent with standard practices applied to similar geophysical investigations. The correlation of geophysical responses with probable subsurface features is based on the past results of similar surveys although it is possible that some variation could exist at this site. Due to the nature of geophysical data, no guarantees can be made or implied regarding the targets identified or the presence or absence of additional objects or targets.

AGS appreciates working for you. We enjoyed this project and we look forward to working with you again.

Sincerely,
Roark W. Smith, GP 987
Senior Geophysicist
Advanced Geological Services, Inc.

Tables:	Table 1	Seismic Investigation Results Summary
	Table 2	“Rip Chart” for Metamorphic Rock
	Table 3	Seismic Site Class Definitions
Figures:	Figure 1	Survey Area Location (imbedded in Report text, above)
	Figure 2	Example “Early Arrival“ Noise (imbedded in Report text, above)
	Figure 3	Seismic Survey Line Locations
	Figure 4	Seismic Refraction Survey Results
Attachments:	Appendix A:	Seismic Velocity and Limitations of the Refraction Method



ADVANCED GEOLOGICAL SERVICES

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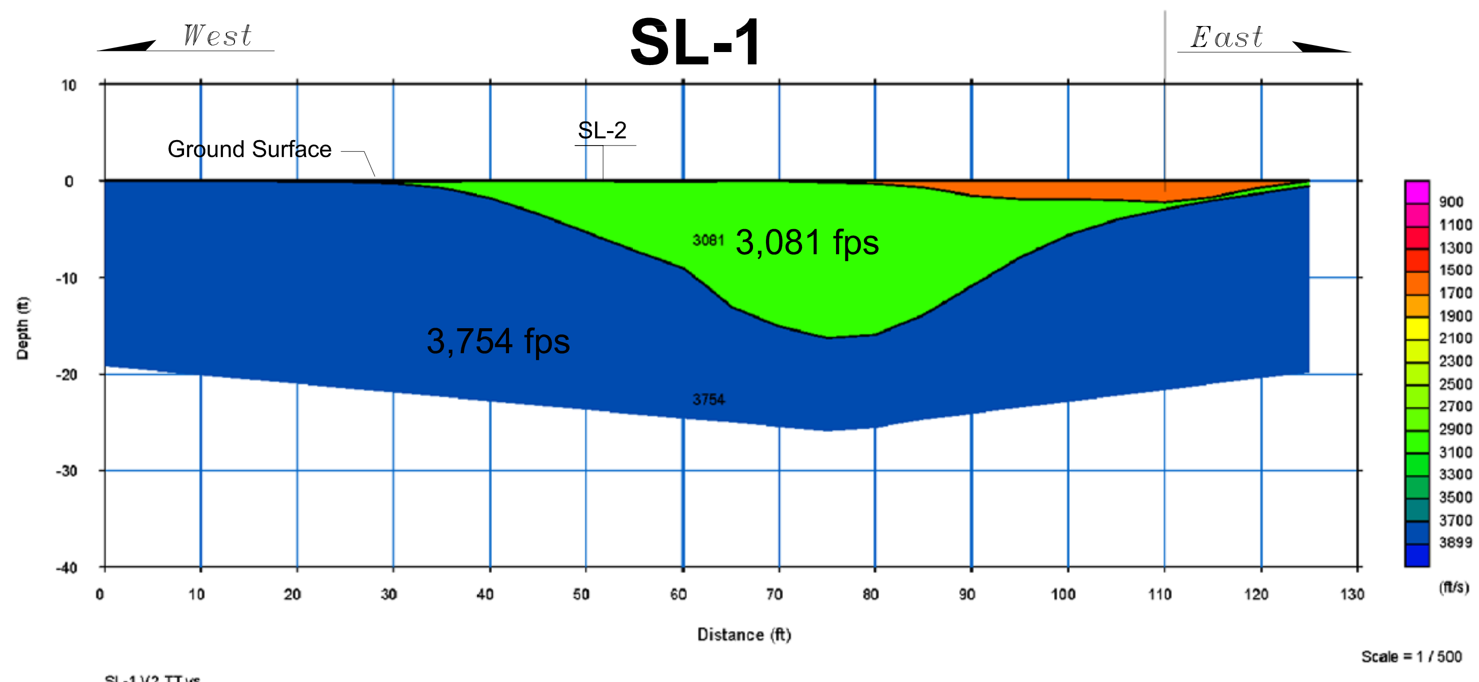
Moraga, CA 94556
(925) 808-8965

Seismic Line Locations
Merritt College

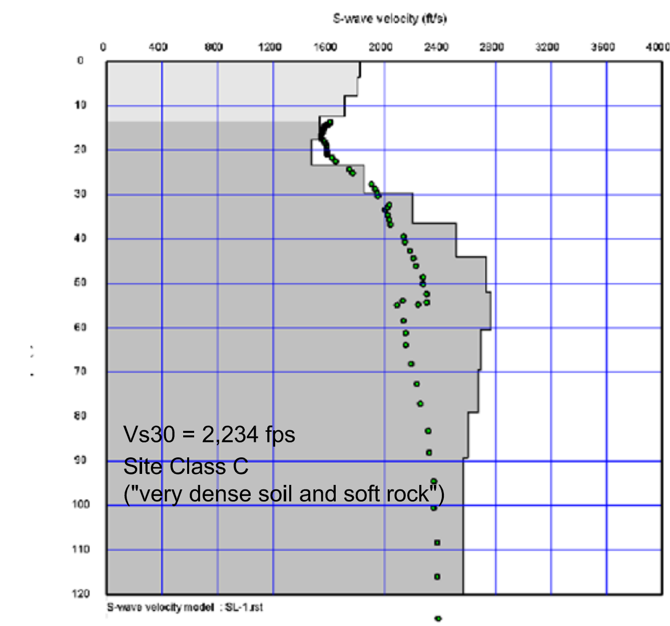
LOCATION: Oakland, California	
CLIENT: Terraphase	FIGURE
PROJECT #: 20-061-1CA	3
DATE: Mar 19, 2020 DRAWN BY: R. SMITH	

REFRACTION SURVEY RESULTS (SL-1 & SL-2)

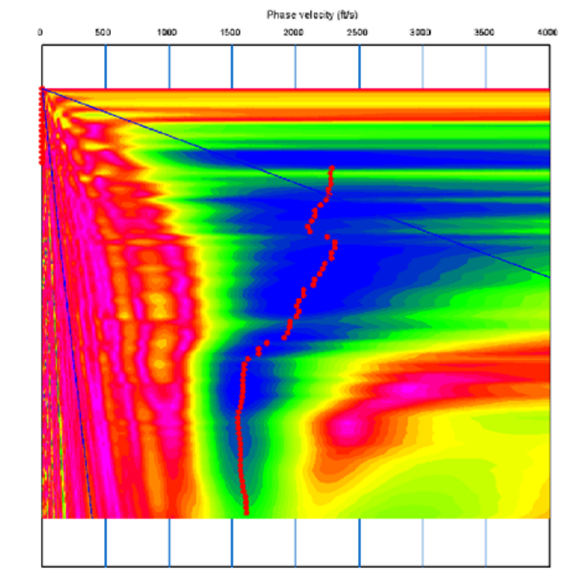


P-wave Velocity Layer Model

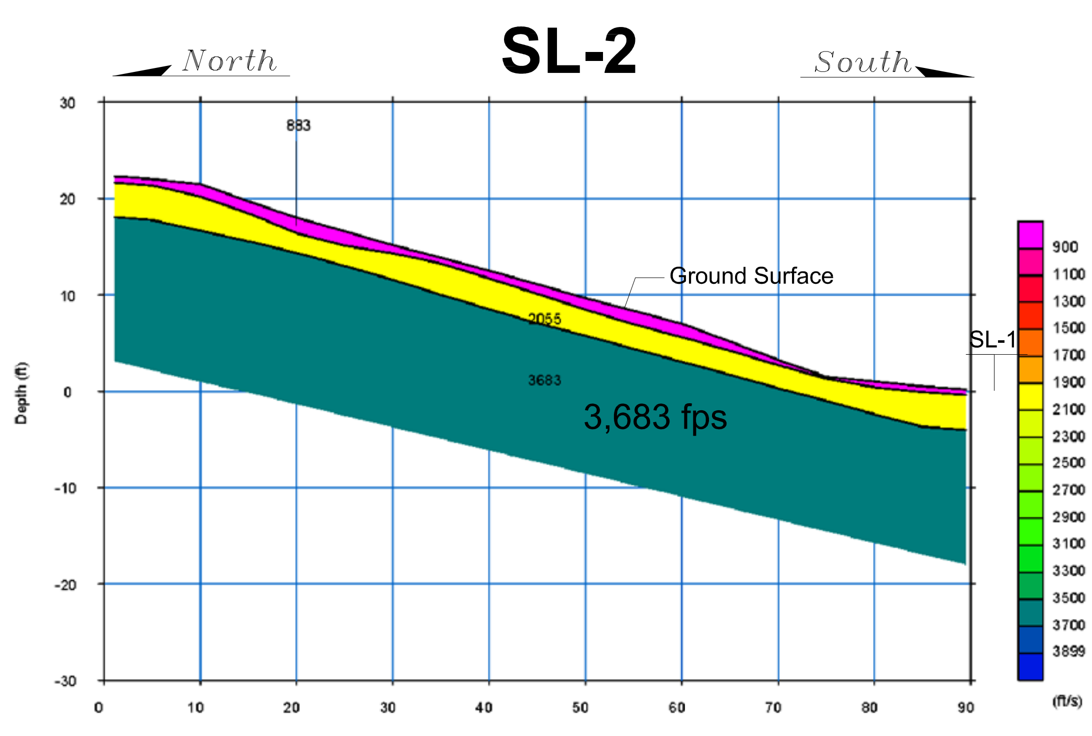
MASW SURVEY RESULTS (SL-1)



S-wave Velocity Layer Model



Velocity Spectrum Image



P-wave Velocity Layer Model

SEISMIC LINE LOCATIONS

ADVANCED GEOLOGICAL SERVICES

1605 School Street
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Seismic Refraction and
MASW Survey Results
Merritt College

LOCATION: Oakland, California

CLIENT: Terraphase

PROJECT #: 20-061-1CA

DATE: Mar 19, 2020

DRAWN BY: R. SMITH

FIGURE 4

APPENDIX A

SEISMIC VELOCITY AND LIMITATIONS OF THE REFRACTION METHOD

The physical properties of earth materials (fill, sediment, rock) such as compaction, density, hardness, and induration dictate the corresponding seismic velocity of the material. Additionally, other factors such as bedding, fracturing, weathering, and saturation can also affect seismic velocity. In general, low velocities indicate loose soil, poorly compacted fill material, poorly to semi-consolidated sediments, deeply weathered, and highly fractured rock. Conversely, high velocities are indicative of competent rock or dense and highly compacted sediments and fill. The highest velocities are measured in unweathered and little fractured rock.

There are certain limitations associated with the seismic refraction method as applied for this investigation. These limitations are primarily based on assumptions that are made by the data analysis routine. The data analysis routine assumes that the velocities along the length of each spread are uniform. If there are localized zones within each layer where the velocities are higher or lower than indicated, the analysis routine will interpret these zones as changes in the surface topography of the underlying layer. A zone of higher velocity material would be interpreted as a low in the surface of the underlying layer. Zones of lower velocity material would be interpreted as a high in the underlying layer. The data analysis routine also assumes that the velocity of subsurface materials increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer will not be resolved. Also, a velocity layer may simply be too thin to be detected.

The quality of the field data is critical to the construction of an accurate depth and velocity profile. Strong, clear "first-break" information from refracted interfaces will make the data processing, analysis, and interpretation much more accurate and meaningful. Vibrational noise or poor subsurface conditions can decrease the ability to accurately locate and pick seismic waves from the interfaces.

Due to these and other limitations inherent to the seismic refraction method, resultant velocity cross-sections should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally.

APPENDIX D
SITE-SPECIFIC SEISMIC HAZARD ASSESSMENT

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D-1 Site-Specific Seismic Hazard

D-1.1 Introduction

A site-specific seismic hazard assessment (SSSHA) was prepared for the Site using the shear wave velocity of 681 meters per second [m/s] – please see Appendix C for the report of the geophysical survey. The Seismic Design Category is E.

D-1.2 Methodology

The SSSHA consists of a probabilistic and a deterministic seismic risk assessment. The probabilistic risk assessment was based on a 2% in 50-year probability level (approximately 1 time in 2,474 years). It used the third Uniform California Earthquake Rupture Forecast (UCERF3) fault model and the 2014 Next Generation of Attenuation Ground Motion Predictors (Abrahamson et al. 2014, Boore et al. 2014, Campbell et al. 2014, Chiou andYoungs 2014 and Idriss 2014) – equally weighted, except Idriss weighted 0.12 in the Probabilistic model. Based on the results of our subsurface work and subsurface work in the adjacent sites, the shear wave velocity in the top 30 meters (VS30) was taken to be 681 meters per second (Site Class C). The software used to implement the analysis was the HazardSpectrumGUI- 1.5.0.jar from the USGS's Open Seismic Hazard Program (https://github.com/opensha). The results of the program were adjusted using the uniform hazard parameters CRS and CR1 from the ASCE 7 (2016) and the directivity factors from ASCE 7 (2016) Section 21.2.

The deterministic spectra was calculated using the PEER NGA Spreadsheet (https://apps.peer.berkeley.edu/ngawest2/databases/).The deterministic risk assessment was run at the 84% probability level (84% of all earthquake accelerations at the Site should be less than the result). A deterministic seismic risk assessment assesses the impacts on a Site assuming that all of the active faults in the Site vicinity can affect the site, but not all at the same time. The two faults that were assessed for the Site were the San Andreas Fault (Santa Cruz Mountains, Peninsula, and North Coast segments) and the combined Hayward North and Rodgers Creek Fault (HN+RC). The San Andreas Fault is located 30.9 kilometers west of the Site and the HN+RC is located 1.1 kilometers east of the Site. Based on the Building Seismic Safety Council 2014 Event Set (Petersen et al. 2015) the San Andreas Fault is believed capable of producing a magnitude 8.0 event while the HN+RC Fault is believed capable of producing a magnitude 7.33 event. The Hayward Fault was found to have higher spectral accelerations at every spectral period than the San Andreas Fault.

Per ASCE7 Addendum 1, if the largest acceleration at any period of the deterministic spectra is less than 1.5xFa, the deterministic spectra should be scaled by 1.5xFa. That was not necessary here because the largest deterministic spectral acceleration was 2.39g (where g is the acceleration of gravity at the earth's surface) and 1.5xFa is 1.8g (Site Class C).

The lower of the deterministic and probabilistic hazard curves at each spectral period was then taken to be the Maximum Considered Earthquake (MCE) Spectra. The design response spectra is then 2/3rds of the MCE Spectra. The resulting design spectrum was then compared to 80% of the design spectrum based on ASCE7 Section 21.3 (Site Class C).

The resulting ASCE 7 structural design parameters are:

S_{DS} = 1.59g S_{D1} = 0.71g

Where S_{DS} is equal to the maximum of 90% of the spectral acceleration at any period between 0.2 and 5 seconds and S_{D1} is the maximum of the products of spectral period times spectral acceleration between 1 and 2 seconds. Per ASCE 7, the S_{DS} and S_{D1} based on the site specific analysis cannot be less than 80% of the map based parameters (1.985g and 0.889g) governed for S_{DS}.

The resulting Spectra are presented in Table D-1 and shown on attached Figure D-1.

Figure D-1

Table D-1
Results of the Site Specific Seismic Risk Assessment
Merritt College Horticultural Facility
Oakland, California

	Uniform Hazard (2% in 50 years Probabilistic Spectrum)	C_R (Risk Coefficient)			Deterministic Risk Hayward Fault 7.3 magnitude and 1.12 km distance		S_{aM} (Site-specific MCE_R spectral response acceleration)	Map Based General Response Spectrum	80% of Map Based General Response Spectrum	Design Response Spectrum
Period (secs)	Probabilistic Risk 2% in 50 years OpenSHA HazardSpectrum.jar (RotD50)	USGS Mapping tool; CRS=0.911, CR1=0.899	Directivity Factors - Section 21.2 ASCE7	Adjusted OpenSHA Spectra for directivity and risk	Deterministic Peer Spreadsheet RotD50	Peer Spreadsheet Max Rotated (Spreadsheet times ASCE7 Section 21.2 Directivity Factors)	Per ASCE 7 21.2.3 the lower of the probabilistic ground motions and deterministic ground motions	USGS Mapping Tool design response spectrum; S_{D1}=0.886, S_{DS}=1.985 (Fa=1.2, Fv=1.4)	=Map based general response * 0.8	ASCE 7 21.3; S_a=2/3S_{aM}
0.010	1.284	0.911	1.100	1.287	1.05	1.16	1.16	0.927	0.74	0.77
0.020	1.323	0.911	1.100	1.325	1.08	1.19	1.19	1.060	0.85	0.85
0.030	1.456	0.911	1.100	1.459	1.19	1.31	1.31	1.193	0.95	0.95
0.050	1.835	0.911	1.100	1.839	1.43	1.57	1.57	1.459	1.17	1.17
0.075	2.358	0.911	1.100	2.363	1.82	2.00	2.00	1.791	1.43	1.43
0.090	2.550	0.911	1.100	2.555	1.973	2.17	2.17	1.985	1.59	1.59
0.100	2.687	0.911	1.100	2.692	2.09	2.29	2.29	1.985	1.59	1.59
0.150	3.050	0.911	1.100	3.057	2.38	2.62	2.62	1.985	1.59	1.75
0.200	3.037	0.911	1.100	3.043	2.37	2.61	2.61	1.985	1.59	1.74
0.250	2.783	0.910	1.113	2.818	2.22	2.47	2.47	1.985	1.59	1.65
0.300	2.517	0.910	1.125	2.575	2.02	2.27	2.27	1.985	1.59	1.59
0.400	2.109	0.908	1.150	2.203	1.71	1.97	1.97	1.985	1.59	1.59
0.448	1.970	0.907	1.162	2.077	1.605	1.86	1.86	1.985	1.59	1.59
0.500	1.819	0.907	1.175	1.937	1.49	1.75	1.75	1.778	1.42	1.42
0.750	1.306	0.903	1.238	1.459	1.05	1.30	1.30	1.185	0.95	0.95
1.000	0.979	0.899	1.300	1.144	0.80	1.05	1.05	0.889	0.71	0.71
1.500	0.602	0.899	1.325	0.717	0.51	0.67	0.67	0.593	0.47	0.47
2.000	0.421	0.899	1.350	0.511	0.36	0.49	0.49	0.445	0.36	0.36
3.000	0.266	0.899	1.400	0.335	0.25	0.34	0.33	0.296	0.24	0.24
4.000	0.186	0.899	1.450	0.243	0.17	0.25	0.24	0.222	0.18	0.18
5.000	0.148	0.899	1.500	0.199	0.13	0.20	0.20	0.178	0.14	0.14

OPEN SHA Report

IMR Param List:

IMR = NGAWest2 2014 Averaged Attenuation Relationship; IMR Weights = ['Abrahamson, Silva & Kamai (2014)': 0.22, 'Boore, Stewart, Seyhan & Atkinson (2014)': 0.22, 'Campbell & Bozorgnia (2014)': 0.22, 'Chiou & Youngs (2014)': 0.22, 'Idriss (2014)': 0.12]; Std Dev Type = Total; Tectonic Region = Active Shallow Crust; Additional Epistemic Uncertainty = null; Component = RotD50; Gaussian Truncation = None

Site Param List:

Longitude = -122.166942; Latitude = 37.793341; Vs30 = 681.0; Vs30 Type = Measured; Depth 2.5 km/sec = 0.5; Depth 1.0 km/sec = 30.0

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = Mean UCERF3; Mean UCERF3 Presets = FM3.1 Branch Averaged; Apply Aftershock Filter = false; Aleatory Mag-Area StdDev = 0.0; Background Seismicity = Include; Treat Background Seismicity As = Point Sources; Use Quad Surfaces (otherwise gridded) = false; Fault Grid Spacing = 1.0; Probability Model = Poisson; Sect Upper Depth Averaging Tolerance = 100.0; Use Mean Upper Depth = true; Rup Mag Averaging Tolerance = 1.0; Rupture Rake To Use = Def. Model Mean; Fault Model(s) = FM3_1; Ignore Cache = false

TimeSpan Param List:

Duration = 50.0

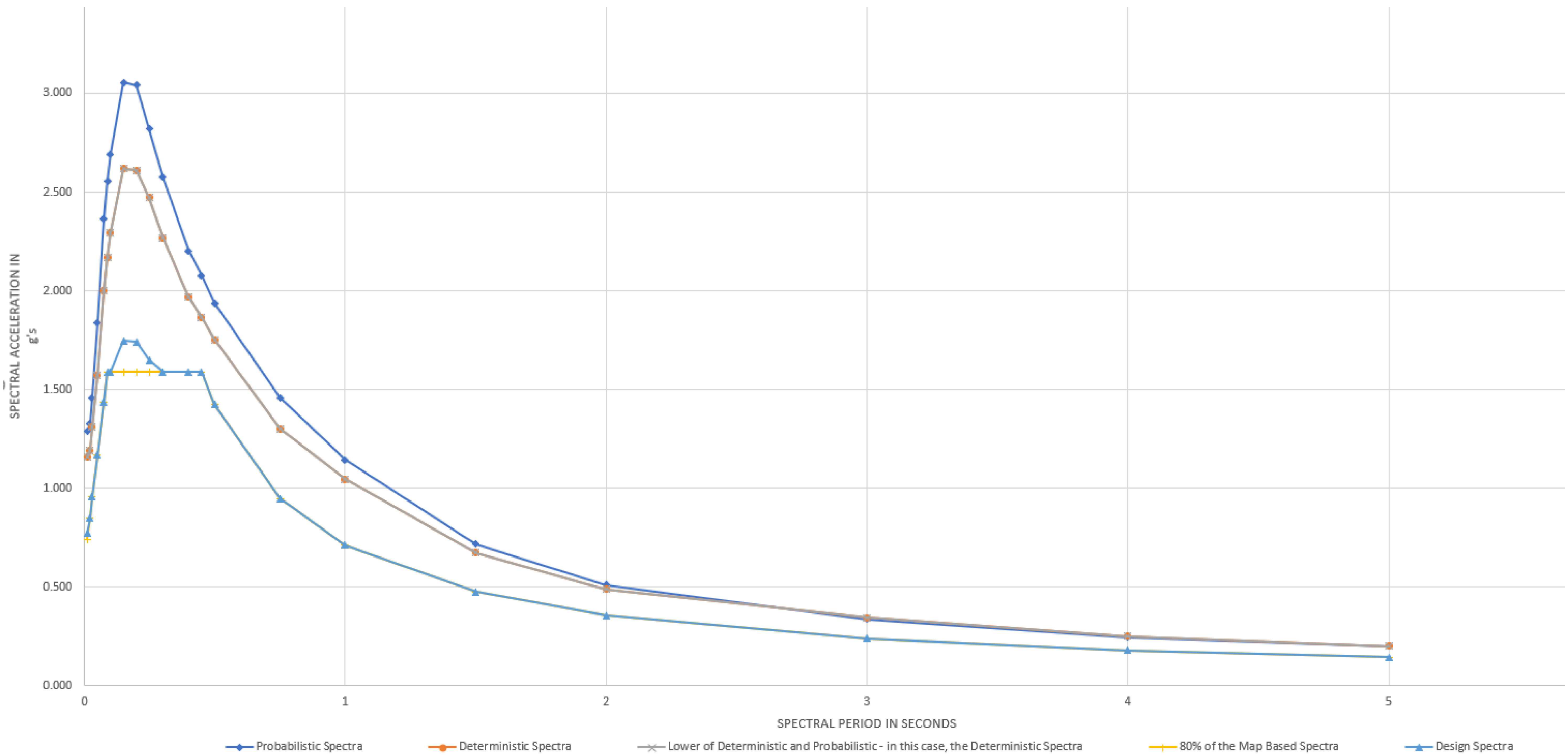
Maximum Distance = 200.0; Pt Src Dist Corr = None

Deterministic Spreadsheet

T (s)	PSa Median for 5% damping	PSa Median + 1.σ for 5% damping
0.01	0.57961	1.05061
0.02	0.59611	1.08234
0.03	0.65149	1.18969
0.05	0.77216	1.42879
0.075	0.96534	1.81625
0.1	1.10172	2.08576
0.15	1.25848	2.38081
0.2	1.25487	2.37216
0.25	1.17497	2.22207
0.3	1.05940	2.01520
0.4	0.89144	1.71052
0.5	0.76682	1.48969
0.75	0.52749	1.05100
1	0.39904	0.80441
1.5	0.24967	0.50740
2	0.17709	0.36088
3	0.12000	0.24501
4	0.08485	0.17181
5	0.06565	0.13314

Input variables

M_w	***Dip (deg)***
7.32	90
R_{RUP} (km)	***Z_{TOR} (km)***
1.12	0
R_{JB} (km)	***Z_{HYP} (km)***
1.12	8
R_X (km)	***$Z_{1.0}$ (km)***
1.12	0.048
Ry0 (km)	***$Z_{2.5}$ (km)***
999	999
V_{S30} (m/sec)	***W (km)***
681	999
U (BSSA13)	***Vs30Flag***
0	measured
F_{RV}	***F_{AS}***
0	No
F_{NM}	***Region***
0	California
F_{HW}	
0	



SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DIST	Seismic Spectra
terraphase engineering	PROJECT: MERRITT COLLEGE HORTICULTURE	
	PROJECT NUMBER: 0034.011.0001	FIGURE D-1

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APPENDIX E
SLOPE STABILITY ANALYSIS

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E1 Introduction

E.1.1 General

Analysis of the factor of safety of the northeast slopes at the Merritt Horticultural Complex (the Site) was conducted in general accordance with the methodology presented in Southern California Earthquake Center (2002) - Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California.

E1.2 Geometry and Material Types

The gradient of the northeast slope above the proposed new structures for the Site is generally uniform at 2.1H:1V (Figure E.1). The proposed structures are at least 28 feet away from the toe of slope and the new greenhouses at least 21 feet away from the toe of slope. The existing structures are within 12 feet of the toe of slope.

Woodward Clyde Hole 17 encountered Knoxville Formation shale, sandstone and limestone to the final depth of the boring (Figure 5 in the main text). Geological mapping (Graymer 2000) (indicates the upper portions of the Site are Serpentinite. The geophysical survey did not indicate a change in material type up to approximately elevation 936 feet above mean sea level (msl, NAD88) (the parking lot at the Site is at elevation 907 feet above msl, so the material type was assumed to be Knoxville Formation to that elevation and serpentinite above elevation 936 feet msl.

The geologic maps (Graymer 2000) indicate that the bedrock bedding is into the slope, i.e., favorable bedding.

The slope is at the top of a regional hill and hence the groundwater table is probably not influenced significantly by infiltration. Woodward Clyde encountered groundwater in Hole 17 far below the ground surface. There are no springs or other indications (wet spots with heavy vegetation) on the slope indicative of a water table in the slope. So the slope stability analyses were run assuming the slope is dry. We did assess the effect of water on stability (please see below).

E1.3 Material Strengths

Material strengths were obtained from CGS (2003a). The material strengths are shown below in Table E-1.

Table E-1
Material Strengths
Merritt College Horticultural Complex
Oakland, California

Material	Φ (°)	C (psf)	ϒ (pcf)
Knoxville Formation	32.5	628	140
Serpentinite	24.5	656	140

Notes pcf – pounds per cubic foot
psf – pounds per square foot

To account for potential strength loss during shaking, though rock shouldn't lose strength, these values were reduced by 10% in the slope stability analysis.

E1.4 Seismic Parameter

In accordance with CGS Note 48, a pseudo-static acceleration parameter equal to the site adjusted Peak Ground Acceleration (PGA_M) of 1.246g/1.5 = 0.83g.

E1.5 Slope Stability Analysis

The above parameters were implemented in Slope/W (version 9.0.0.15234) using the method of Morgenstern-Price to calculate the factor of safety against slope failure. The slope stability results are presented below and in Figure E-2. With a pseudostatic acceleration of 0.83g, the factor of safety for the slope was less than 1.0.

The slope stability analysis was then rerun using progressively smaller pseudostatic acceleration parameters until the acceleration causing the factor of safety to be 1.0 was found. This acceleration was 0.42g.

A Newmark "sliding block" analysis was then used to evaluate how much a failed block on the slope would move during the Maximum Considered Earthquake. The seismogram for the Rinaldi Receiving Station (Los Angeles Reservoir) was scaled to a PGA of 1.246g (Figure E-3). Positive accelerations above 0.42g and negative accelerations below -0.42g were then integrated twice to develop movements. The results of this analysis are presented below in Table E-2.

Table E-2
Newmark Method Movements
Merritt College Horticultural Complex
Oakland, California

Start	Stop	Peak Velocity (ft/sec)	Movement (inches)
1.79	2.21	2.78	6.58
2.66	2.85	0.52	0.67
3.03	3.15	0.26	0.11
3.82	3.92	0.59	0.20
5.58	5.82	1.13	0.07
Total			7.64
2.16	2.83	-6.61	-28.80
3.94	4.12	-0.67	-0.69
6.27	6.38	-0.32	-0.23
Total			-29.72

Displacement of a failed block on the slope of 30 inches will not impinge on the parking lot assuming no secondary mobilization or flow, and hence, other than damaging irrigation lines, will not be a significant hazard for the new development (which is further from the slope than the existing buildings).

The Rinaldi Receiving station is an alluvium site. The analysis was also run on the seismogram from the Griffith Park Observatory seismogram, a bedrock site, recorded during the Northridge Earthquake and the resulting slope movement was smaller.

We also assessed the yield acceleration through a failure at the toe of the assumed base of the serpentinite at elevation 936 feet above mean sea level (NAD88). The yield acceleration was determined to be 0.55g. The resulting slope movement was 19 inches which does not pose a threat to the proposed buildings.

Putting a groundwater table at 17 feet bgs produced a yield acceleration of 0.34g in the slope. Using this in the Newmark analysis increased the slope movement by less than 12 inches.

This analysis is extremely conservative. The SCEC guidance for implementing DGM Special Publication 117 (SCEC 2002) recommends using the PGA from the 10% in 50 years earthquake adjusted by a factor less than 1.0 that depends on the earthquake magnitude and acceleration. For the Site using the SP-117 recommendations, the pseudostatic acceleration would be 0.28g and the factor of safety would be greater than 1.0.

Slope/W Output

File Information

File Version: 9.00
Title: Merritt College Horticulture Slope
Created By: jrr
Revision Number: 34
Date: 08/03/2020
Time: 04:15:11 PM
Tool Version: 9.0.0.15234
File Name: merritt horticulture.gsz
Directory: C:\Users\Jeff\Desktop\
Last Solved Date: 08/03/2020
Last Solved Time: 04:15:13 PM

Project Settings

Unit System: U.S. Customary Units

Analysis Settings

Slope Stability

Kind: SLOPE/W
Method: Morgenstern-Price
Settings
- Side Function
- Interslice force function option: Half-Sine
- PWP Conditions from: (none)
- Unit Weight of Water: 62.430189 pcf

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack Option: (none)

Distribution
- F of S Calculation Option: Constant

Advanced
- Geometry Settings
  - 11.0 Minimum Slip Surface Depth: 0.1 ft

12.0 Number of Slices: 30

Factor of Safety Convergence Settings

13.0 Maximum Number of Iterations: 100

14.0 Tolerable difference in F of S: 0.001

Solution Settings

15.0 Search Method: Root Finder

16.0 Tolerable difference between starting and converged F of S: 3

17.0 Maximum iterations to calculate converged lambda: 20

18.0 Max Absolute Lambda: 2

Materials

Shale

Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion': 566 psf
Phi': 29.8 °
Phi-B: 0 °

Serpentinite

Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion': 590 psf
Phi': 22.3 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Type: Range
Left-Zone Left Coordinate: (0, 964) ft
Left-Zone Right Coordinate: (100, 964) ft
Left-Zone Increment: 4
Right Type: Point
Right Coordinate: (221.2, 907) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 964) ft
Right Coordinate: (300, 905.5) ft

Seismic Coefficients

Horz Seismic Coef.: 0.42
Vert Seismic Coef.: 0

Points

	X	Y
Point 1	100 ft	964 ft
Point 2	130 ft	948 ft
Point 3	137.4 ft	945 ft
Point 4	153.4 ft	937 ft
Point 5	154.2 ft	936 ft
Point 6	160.6 ft	936 ft
Point 7	221.2 ft	907 ft
Point 8	229.5 ft	907 ft
Point 9	229.6 ft	905.5 ft
Point 10	300 ft	905.5 ft
Point 11	300 ft	805.5 ft
Point 12	0 ft	805.5 ft
Point 13	0 ft	964 ft
Point 14	0 ft	936 ft

Regions

	Material	Points	Area
Region 1	Serpentinite	1,2,3,4,5,14,13	3,558.1 ft²
Region 2	Shale	14,5,6,7,8,9,10,11,12	35,880 ft²

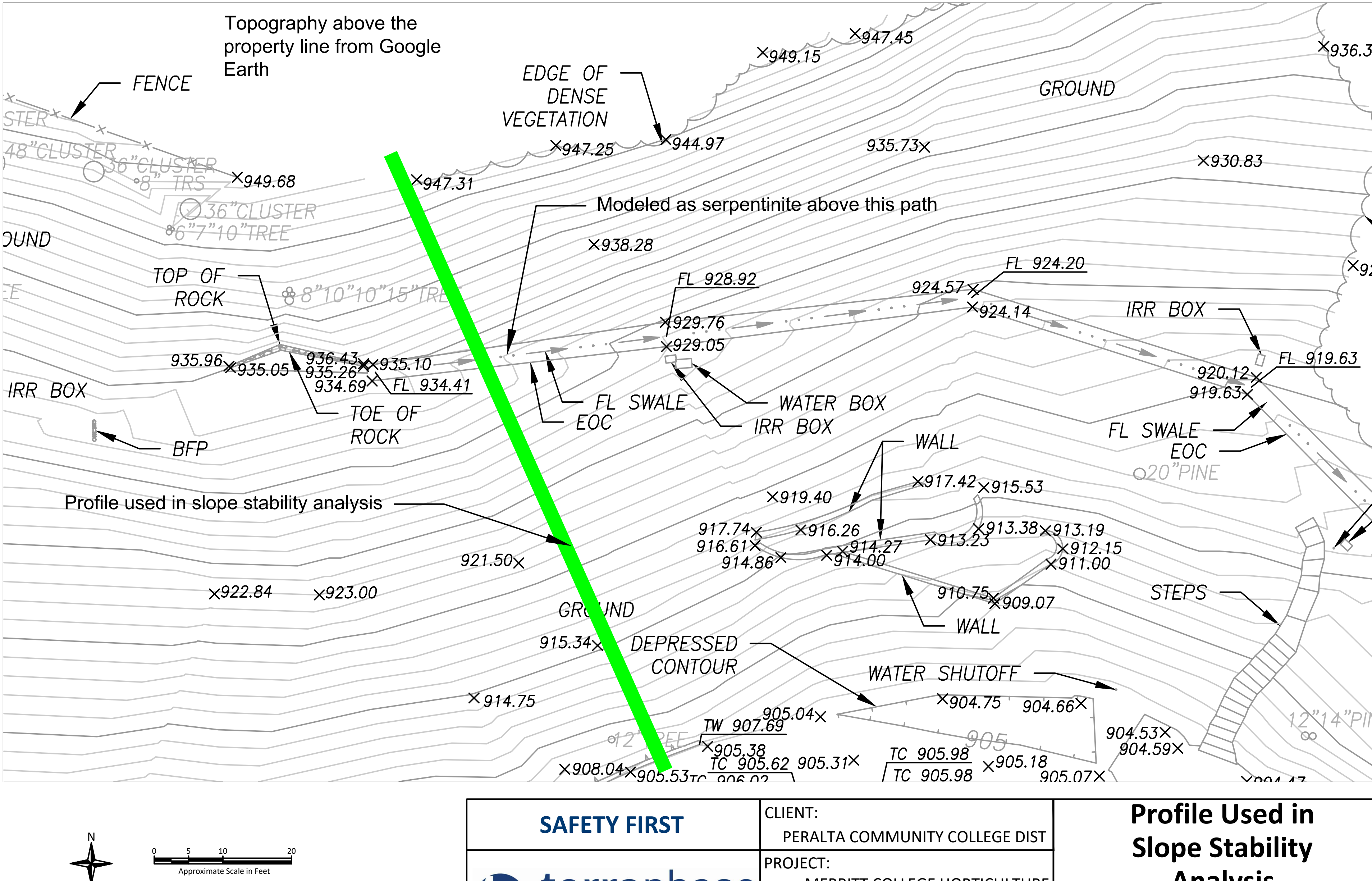
Current Slip Surface

Slip Surface: 12
Factor of Safety: 1.001
Volume: 3,531.1921 ft³
Weight: 494,366.89 lbf
Resisting Moment: 80,401,099 lbf·ft
Activating Moment: 80,268,618 lbf·ft
Resisting Force: 320,692.2 lbf
Activating Force: 320,480.86 lbf
Slip Rank: 1 of 25 slip surfaces

Exit: (221.2, 907) ft
Entry: (50, 964) ft
Radius: 236.44818 ft
Center: (204.64181, 1,142.8677) ft

Slip Slices

	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength
Slice 1	52.866259 ft	961.60008 ft	0 psf	-84.17547 psf	-34.522876 psf	590 psf
Slice 2	58.598776 ft	956.94865 ft	0 psf	431.64558 psf	177.03075 psf	590 psf
Slice 3	64.331294 ft	952.58365 ft	0 psf	853.01349 psf	349.84633 psf	590 psf
Slice 4	70.063811 ft	948.48572 ft	0 psf	1,200.8621 psf	492.50944 psf	590 psf
Slice 5	75.796329 ft	944.63829 ft	0 psf	1,492.724 psf	612.21073 psf	590 psf
Slice 6	81.528847 ft	941.02699 ft	0 psf	1,743.7918 psf	715.18112 psf	590 psf
Slice 7	87.261364 ft	937.63936 ft	0 psf	1,967.6859 psf	807.00679 psf	590 psf
Slice 8	92.595717 ft	934.67173 ft	0 psf	2,188.6681 psf	1,253.462 psf	566 psf
Slice 9	97.531906 ft	932.08927 ft	0 psf	2,405.5503 psf	1,377.6717 psf	566 psf
Slice 10	103 ft	929.40659 ft	0 psf	2,526.1655 psf	1,446.7486 psf	566 psf
Slice 11	109 ft	926.65108 ft	0 psf	2,562.2406 psf	1,467.4091 psf	566 psf
Slice 12	115 ft	924.0948 ft	0 psf	2,623.4208 psf	1,502.4473 psf	566 psf
Slice 13	121 ft	921.73085 ft	0 psf	2,709.9915 psf	1,552.0268 psf	566 psf
Slice 14	127 ft	919.55311 ft	0 psf	2,819.5451 psf	1,614.7687 psf	566 psf
Slice 15	133.7 ft	917.34617 ft	0 psf	3,004.8822 psf	1,720.9122 psf	566 psf
Slice 16	140.06667 ft	915.42515 ft	0 psf	3,205.2784 psf	1,835.6802 psf	566 psf
Slice 17	145.4 ft	913.97785 ft	0 psf	3,339.4473 psf	1,912.5195 psf	566 psf
Slice 18	150.73333 ft	912.66318 ft	0 psf	3,454.7277 psf	1,978.5412 psf	566 psf
Slice 19	153.8 ft	911.95064 ft	0 psf	3,476.9072 psf	1,991.2435 psf	566 psf
Slice 20	157.4 ft	911.21 ft	0 psf	3,661.0565 psf	2,096.7069 psf	566 psf
Slice 21	163.35455 ft	910.0689 ft	0 psf	3,881.6068 psf	2,223.0172 psf	566 psf
Slice 22	168.86364 ft	909.15869 ft	0 psf	3,849.8615 psf	2,204.8365 psf	566 psf
Slice 23	174.37273 ft	908.38142 ft	0 psf	3,733.662 psf	2,138.2884 psf	566 psf
Slice 24	179.88182 ft	907.73579 ft	0 psf	3,527.0007 psf	2,019.9324 psf	566 psf
Slice 25	185.39091 ft	907.2207 ft	0 psf	3,231.0331 psf	1,850.4301 psf	566 psf
Slice 26	190.9 ft	906.8353 ft	0 psf	2,854.0175 psf	1,634.5112 psf	566 psf
Slice 27	196.40909 ft	906.57895 ft	0 psf	2,409.9794 psf	1,380.2082 psf	566 psf
Slice 28	201.91818 ft	906.45125 ft	0 psf	1,916.3806 psf	1,097.5215 psf	566 psf
Slice 29	207.42727 ft	906.45197 ft	0 psf	1,391.3741 psf	796.84746 psf	566 psf
Slice 30	212.93636 ft	906.58112 ft	0 psf	851.35857 psf	487.57765 psf	566 psf
Slice 31	218.44545 ft	906.83891 ft	0 psf	309.43718 psf	177.21635 psf	566 psf



SAFETY FIRST

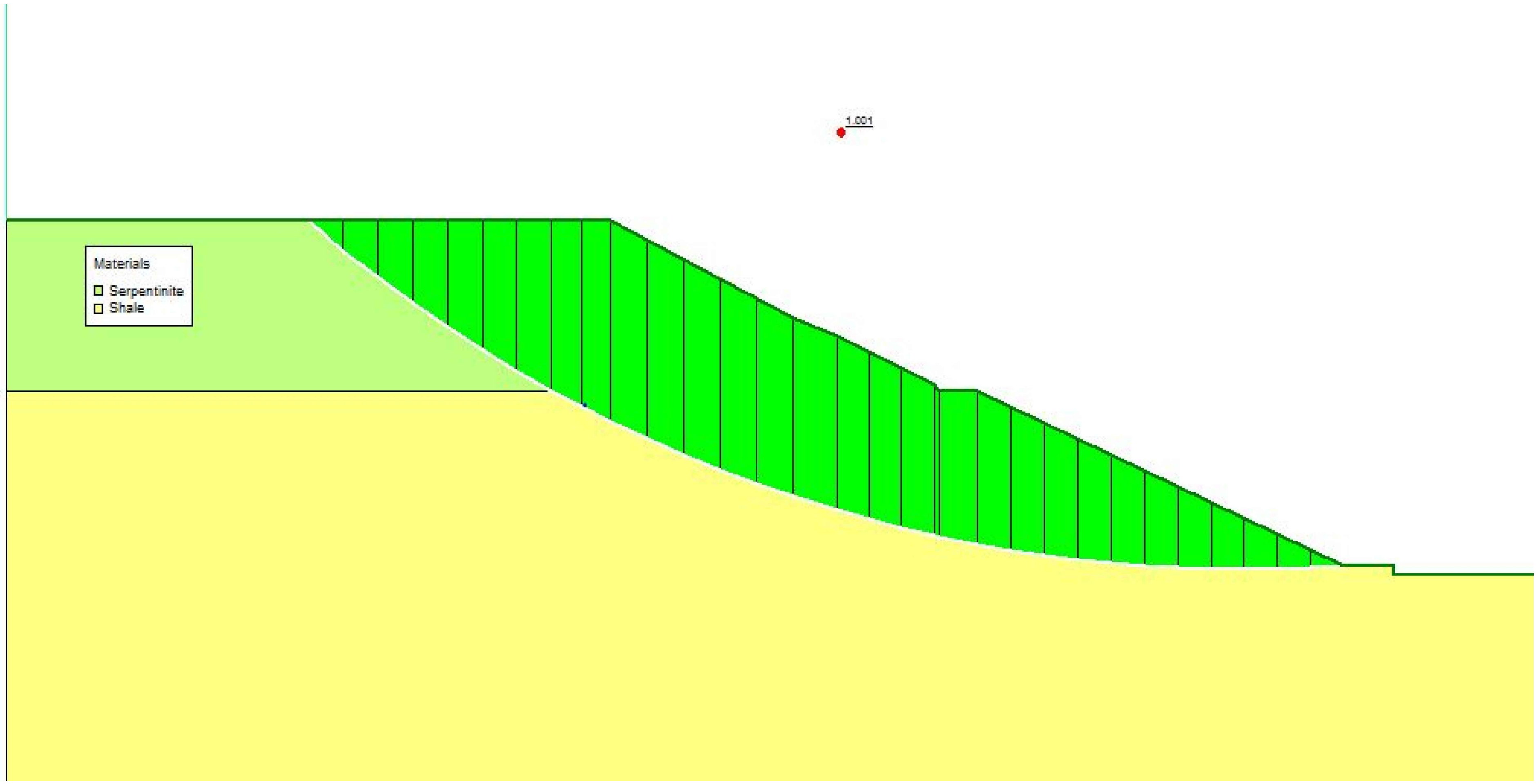
terraphase engineering

CLIENT:	PERALTA COMMUNITY COLLEGE DIST
PROJECT:	MERRITT COLLEGE HORTICULTURE
PROJECT NUMBER:	0034.011.0001

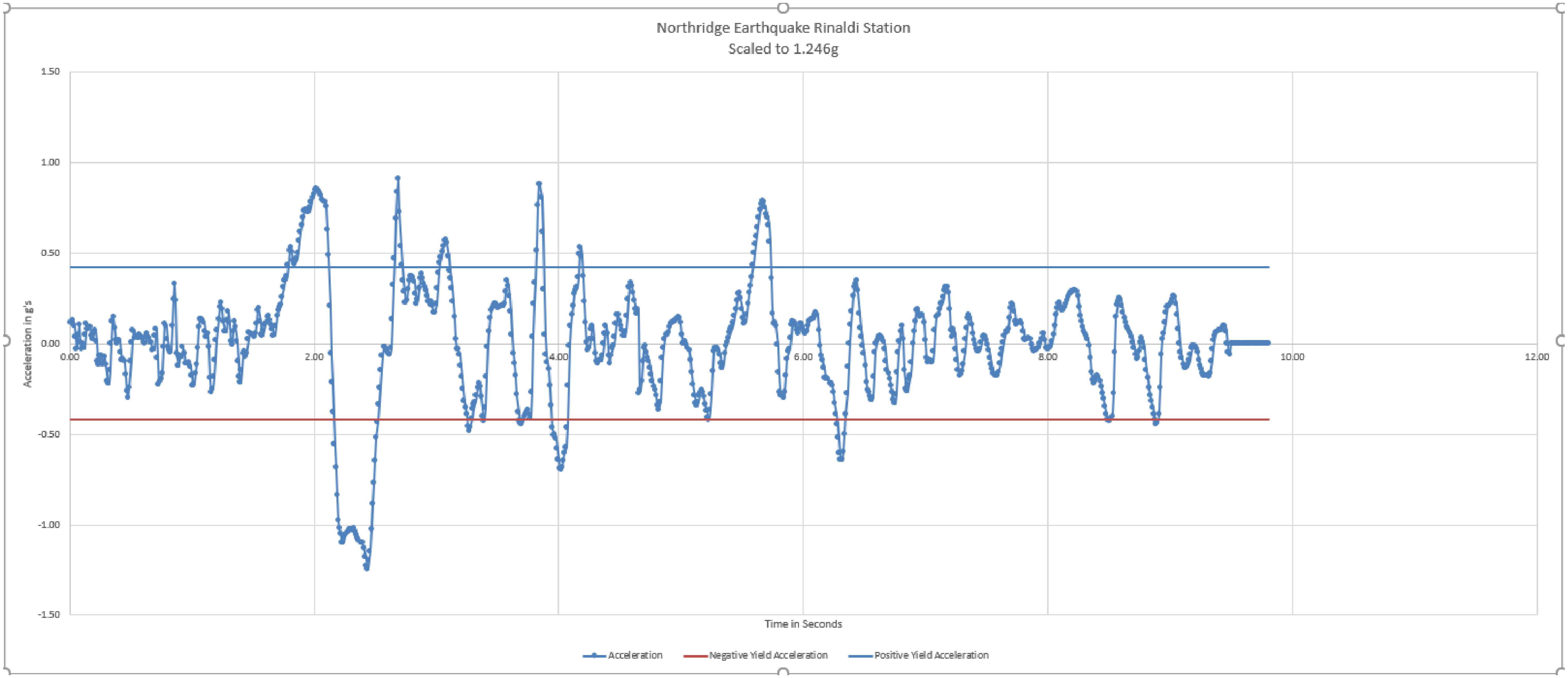
Profile Used in Slope Stability Analysis

FIGURE E-1

Slope yields at a
pseudostatic acceleration
of 0.42g



SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DIST	Slope Stability Result
terraphase engineering	PROJECT: MERRITT COLLEGE HORTICULTURE	
	PROJECT NUMBER: 0034.011.0001	FIGURE E-2



SAFETY FIRST	CLIENT: PERALTA COMMUNITY COLLEGE DIST	Seismogram Used in Newmark Analysis
terraphase engineering	PROJECT: MERRITT COLLEGE HORTICULTURE	
	PROJECT NUMBER: 0034.011.0001	FIGURE E-3

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