APPENDIX D

Geologic Hazards
&
Geotechnical Engineering Report

FOLSOM LAKE COLLEGE

INSTRUCTIONAL BUILDING

Folsom, California

MPE No. 04843-01

MPE

MID PACIFIC ENGINEERING, INC.

January 15, 2020

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Geologic Hazards and Geotechnical Engineering Report

FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING

10 College Parkway
Folsom, California
MPE No. 04843-01
January 15, 2020

INTRODUCTION

We have completed a Geologic Hazards and Geotechnical Engineering Investigation for the proposed Instructional Building project to be constructed at the existing Folsom Lake College campus located at 10 College Parkway in Folsom, California. The purposes of our study have been to investigate the site, soil, groundwater, geologic and seismic conditions at the site, and to prepare Geologic and Geotechnical Engineering conclusions and recommendations for use by the other members of the design team in preparing project plans and specifications for the proposed project. This report presents the results of our work.

Scope of Services

Our scope of work included the following:

1. Site reconnaissance;
2. Review of the following plans:
   - Architectural plans, Sheets A1 through A3, *Folsom Lake College, Los Rios Community College District, Phase 2.1 Instructional Building, Final Project Proposal Pre-Schematic Plans*, dated April 2016;
   - *Grading & Utilities Plan*, Sheets C2.06 and C2.07, prepared by PSOMAS and dated November 5, 2004, referred to, hereinafter, as the 2004 Grading Plan;
3. Review of available historic aerial photographs, topographic maps and groundwater information of the area;
4. Review of geologic maps and fault maps;
5. Review of historic seismicity within 100 kilometers (km) of the site;

6. Subsurface exploration within areas proposed for improvements, including 1) the excavating, logging, and sampling 12 test pits to approximate maximum depths of 3 to 9½ feet below existing ground surface (bgs); 2) the drilling, logging, and sampling two exploratory soil borings to approximate maximum depths of 20½ and 41¼ feet bgs;
7. Collection of bulk and in-situ soil samples at various depths within the test pits;
8. Laboratory testing of selected soil samples;
9. Engineering analyses; and,
10. Preparation of this report.

FIGURES AND ATTACHMENTS

Figure	Title	Figure	Title
1	Vicinity Map	11	Rock Legend
2	Regional Geologic Map	12	Geologic Cross-Section A[1]-A[5]
3	Boring and Test Pit Location Map	13	Regional Fault Map
4 - 5	Logs of Soil Borings	14	Regional Seismicity Map
6 - 9	Logs of Test Pits	15	FEMA Flood Map
10	Unified Soil Classification System		

Appended to this report are:

- Appendix A - General information regarding project concepts; exploratory methods used during our field investigation; and, laboratory test results not included on the boring and test pit logs.
- Appendix B - *Guide Earthwork Specifications* that may be used in the preparation of contract documents.
- Appendix C - Output files from the EQFAULT/EQSEARCH programs.
- Appendix D - A list of references cited.
- Appendix E - *Rippability and Vs100 Class Determination Report*, prepared by Petralogix Engineering, Inc., dated January 11, 2019.

Proposed Development

Based on our review of the Architectural Plans, it is our understanding the project will consist of the construction of an approximately 75,000 square foot instructional building with a plan dimension of approximately 37,500 square feet. It is anticipated that the proposed building will be a two story, steel-frame structure with concrete slab-on-grade ground floors supported on the conventional foundation system. Information regarding structural loads was not available at the time we prepared this report, but we assume the loads will be relatively light to moderate, based on the anticipated construction.

Associated development is anticipated to include construction of new parking lot with associated access driveways, two new fire access roads, underground utilities, light poles, exterior flatwork, and landscaping.

Grading plans were not available at the time this report was prepared; however, for the purposes of this report and based on the sloped site topography, we have assumed earthwork cuts and fills of up to 10 feet in depth.

FINDINGS

Site Description

The project site is within the south-western portion of the Folsom River College campus located at 10 College Parkway in Folsom, California. The approximate location of the project is north latitude 38.6612° and west longitude 121.1290°.

The site is generally bounded to the north by the existing campus buildings and parking lots; to the east by vacant property, beyond which is parking lot; to the south and west by Campus Loop Road (College Parkway), beyond which is vacant property. On the dates of our investigation, the project site was vacant land covered with low growth of vegetation. Underground utilities (sewer line, gas main, and electric line) were noted throughout the project site.

Review of the 2004 Grading Plan, indicates the site ground surface elevations range from approximately +380 feet relative to mean sea level (msl) in the western portion of the site to

+425 feet msl in the eastern portion of the site. Within the proposed building footprint the site ground surface elevations range from approximately +405 feet msl in the western portion of the site to +427 feet msl in the eastern portion of the site Portions of the USGS *Folsom Quadrangle, California* topographic map containing the site and vicinity, is included with this report as Figure 1. The project site topography slopes to the west.

Site History

The project site history was compiled based on review of historical aerial photographs (dated 1952, 1958, 1964, and 1966) and Google Earth images (dated 1993, 1998, 2002 through 2018).

The site was an undeveloped land at least until 2004. The 2005 photograph depicts grading activity on site. The site remained essentially unchanged since 2005. Improvements surrounding the site (campus building and parking lots) were constructed in a period between 2002 and 2008. Campus Loop Road (College Parkway) was constructed in 2005.

GEOLOGIC SETTING

Regional Geology and Structure

The project site lies along the western boundary of the northern portion of the Sierra Nevada geomorphic province of California. The Sierra Nevada is a tilted fault block nearly 400 miles long with the gentle sloping western that disappears under the sediments of the Great Valley geomorphic province. Drainage is primarily west ward along the more gently sloping side of the mountain range.

Site Geology

The California Geological Survey (CGS) *Preliminary Geologic Map of the Sacramento 30'x60' Quadrangle, California*, indicates the project site is underlain by the Jurassic unit of the Copper Hill Volcanics (Map Symbol: Jch) described as mafic to andesitic pyroclastic rocks, lava and pillow lava with felsic pyroclastic and porphyritic rocks. The subsurface conditions encountered in our test pits and borings were generally consistent with those typically mapped as volcanic rock.

The United States Department of Agriculture, Natural Resources Conservation Service website (http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx), indicates the site is underlain by Argonaut-Auburn complex, 3 to 8 percent slopes. The Argonaut soil is moderately deep, well drained, has very slow permeability, and formed in material weathered from meta-andesite and metamorphic rocks. The Auburn soil is shallow or moderately deep, well drained, has moderate permeability, and formed in material weathered from metabasic and metasedimentary rocks.

SUBSURFACE SOIL CONDITIONS

Fill soils were encountered in every boring drilled and in nine test pits excavated. Fill soils consist of silty, sandy, fine to coarse gravel, with cobbles and extend to depth of approximately ½ to 5 feet bgs. Fill soils are in loose condition based on the observed sloughing and caving of the test pit sidewalls excavated in the fill soils. The native soils encountered consist of predominantly sandy, clayey silts and silty, clayey sands and extend to depths of 1½ to 6½ feet bgs. The thickness of these soils range from 1½ to 2½ feet. These soils are underlain by completely to highly weathered andesite (volcanic rock) and highly to slightly weathered basalt (volcanic rock) of the Copper Hill Volcanics Formation (map symbol: Jch) to the maximum depth explored 41¼ feet bgs.

For soil conditions at a specific location, please refer to the Logs of Soil Borings (Figures 4 and 5) and the Logs of Test Pits (Figures 6 through 9). An explanation of the symbols and classification system used on the Logs is presented on Figures 10 and 11. Graphic illustrations of the subsurface conditions encountered in the borings and test pits are presented on the geologic cross-section (Figure 12).

GROUNDWATER

Permanent groundwater was not encountered in test pits excavated on December 18, 2019 and borings advanced on December 27 and 30, 2019, to the maximum depth explored of 41¼ feet bgs. Review of the *Sacramento County Groundwater Elevation Maps* produced by the County of Sacramento, Water Resources Division for the period from 2000 through 2017 indicates that the highest and lowest groundwater elevations were +150 feet msl in 2004 and +120 feet msl in 2006, respectively. Considering the lowest site elevation +380 feet msl, it is anticipated that the groundwater at the site is deeper than 200 feet bsg.

Seepage or perched water was observed in one of the test pits (TP-12) at a depth of five feet bsg, at the contact of the fill soils and native soils.

Groundwater levels may fluctuate beneath the site depending on the time of year and rainfall amounts. Therefore, groundwater conditions presented in this report may not be representative of those which may be encountered during or subsequent to construction.

REGIONAL SEISMICITY

FAULTING

The project site is not located across the mapped trace of any known fault, nor was there any indication of surface rupture or fault-related surface disturbance at the site during our review of aerial photographs, site reconnaissance, or geotechnical investigation.

The site is not located within an Alquist-Priolo Earthquake Fault Zone as currently designated by the State of California (DMG Special Publication No. 42, revised 1997). The nearest Earthquake Fault Zone is the Mount George Fault of Green Valley Fault System, located approximately 61.5 miles (99.0 kilometers) west of the project site. A Regional Fault Map (Figure 13) is included with this report.

According to the United States Geological Survey (USGS), 2008 National Seismic Hazard Maps – Source Parameters website, (https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm), the closest Type A or Type B faults to the site are the Great Valley 4a and Great Valley 3, located approximately 48.0 miles (77.2 kilometers) west and 49.8 miles (80.3 kilometers) west-northwest of the project site.

Using the USGS Earthquake Hazards Program, 2008 National Seismic Hazard Maps-Source Parameters, we have prepared Table 1 containing CGS Class A and B faults and fault rupture segments within 62 miles (100 kilometers) of the site that are considered capable of producing earthquakes with maximum moment magnitudes (M_W) 6.5 or greater. The M_{wmax} value represents the maximum earthquake believed possible for each fault.

Table 1 - Faults and Fault Rupture Segments Influential to Los Rios-Folsom Lake College

Fault Name	Maximum Magnitude (M_W)	Distance To Site Miles (Kilometers)
Great Valley 4a, Trout Creek	6.6	48.0 (77.2)
Great Valley 3, Mysterious Ridge	7.1	49.8 (80.3)
Great Valley 4b, Gordon Valley	6.8	50.6 (81.4)
Great Valley 5, Pittsburg Kirby Hills	6.7	52.8 (85.0)
Hunting Creek-Berryessa	7.1	59.7 (96.0)
West Tahoe	7.1	59.8 (96.2)
Green Valley Connected	6.8	60.1 (96.7)

The Foothills Fault system (Geodetic zone of distributed shear (C Zone) # 1) utilized in the preparation of the USGS 2008 National Seismic Hazard Maps is located approximately 1.9 miles east of the site. Minimum and maximum moment magnitudes of 6.5 and 7.6, respectively, were assigned to this zone by the USGS.

Review of the CGS California Fault Activity Map of California (2010) database indicates that the nearest fault to the site with the activity in Quaternary time is the Late Quaternary Bear Mountains Fault Zone (Rescue lineament) of the Foothills Fault System located approximately 11.5 miles (18.5 kilometers) north-east from the site. The nearest mapped fault to the site is the unnamed Pre-Quaternary Fault located approximately 3.2 miles (5.1 kilometers) east from the site.

In general, and for larger earthquake scenarios, the magnitude that is utilized for reporting to the public (and for site hazard assessment) is the moment magnitude. The moment magnitude is based on the scalar seismic-moment of an earthquake determined by calculation of the seismic moment-tensor that best accounts for the character of the seismic waves generated by the earthquake. The scalar seismic-moment, a parameter of the seismic moment-tensor, can also be estimated via the multiplicative product rigidity of faulted rock x area of fault rupture x average fault displacement during the earthquake (USGS, 2008). Results of a hazard deaggregation conducted utilizing USGS Unified Hazard Tool indicates that the mode magnitude earthquake for the site is 7.1 (Hunting Creek – Berryessa). This is the moment magnitude that should be used for site hazard assessment purposes.

HISTORIC SEISMICITY

Seismological data regarding significant historical earthquakes affecting the site was obtained using the commercially available software program EQSEARCH (Blake, 2000; database updated 2018). The EQSEARCH database was developed by extracting records of events greater than magnitude 5.0 from the DMG Comprehensive Computerized Earthquake Catalog, and supplemented by records from the USGS; University of California, Berkeley; the California Institute of Technology; and, the University of Nevada at Reno. A search radius of 62 miles (100 kilometers) was specified for this analysis. A historic earthquake epicenter map showing earthquakes (magnitude 5.0 or greater) within the project region is presented as Figure 14.

A review of the historical earthquake data indicates that the most significant earthquake shaking (acceleration) experienced at the project site occurred during the 1892 Vacaville-Winters earthquake sequence. The source of these events is attributed to the Midland Fault. The estimated magnitudes of these events ranged from 5.5 to 6.4 and produced estimated site peak ground accelerations of 0.029 to 0.040 g. The closest epicenter is located approximately 40.5 miles (65.1 kilometers) west of the site. An examination of the tabulated EQSEARCH data suggests that the project site has experienced maximum ground shaking equivalent to Modified Mercalli Intensity IV[1] as the result of six earthquakes.

Among the most recent earthquakes, the Mw=5.2 1980 Snow Mountain and the Mw=5.7 1975 Palermo events produced estimated site peak ground accelerations of 0.020 g and 0.025 g, respectively.

The number of earthquakes greater than Mw 5.0 within a 62 mile (100 kilometer) radius of the site is presented in the following table.

[1] IV – Light: Felt indoors by many, outdoors by few during the day. At night, some awakened Dishes, windows, doors, disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.

TABLE OF MAGNITUDES AND EXCEEDANCES	
Earthquake Magnitude	Number of Times Exceeded
5.0	12
5.5	7
6.0	3

Output files from the EQFAULT/EQSEARCH programs are included in Appendix C.

COSEISMIC GROUND DEFORMATION

The California State Legislature passed the Seismic Hazards Mapping Act (SHMA) in 1990 (Public Resources Code Division 2, Chapter 7.8) as a result of earthquake damage caused by the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes. The purpose of the SHMA is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, and other hazards caused by earthquakes (California Geological Survey [CGS] Special Publication [SP] 117).

There are currently three State designated Seismic Hazard Zone maps for Sacramento County.

SITE CLASS

Review of the *Rippability and Vs100 Class Determination Report*, prepared by Petralogix Engineering, Inc, dated January 11, 2019, indicates that the minimum value of the soil shear wave velocity (Vs) in the upper 100 feet of the project site is equal to 2938 feet per second (ft/sec). Based on this value, the site can be classified as Site Class B; however, based on the anticipated grading (more than 10 feet of engineered feet beneath the foundations) and Chapter 1613.5.5 of the 2019 CBC, it is our opinion that Site Class C is most applicable to the soils and rock conditions anticipated to be present upon completion of grading.

We should review the final grading plans and work with the design team to determine if Site Class B can be used appropriately.

SEISMIC CODE PARAMETERS

Section 1613*A* of the 2019 edition of the CBC references ASCE Standard 7-16 for seismic design. The following seismic parameters were determined based on the site latitude and longitude using the web interface developed by the Structural Engineers Association of California (SEAOC) and California Office of Statewide Health Planning and Development (OSHPD) (https://seismicmaps.org/) to retrieve seismic design data from the public domain computer program developed by the USGS. The seismic design parameters summarized in the table below may be used for seismic design of the proposed improvements.

Table 2 - 2019 CBC/ASCE 7-16 Seismic Design Parameters

Latitude: 38.6612° N Longitude: -121.1290° W	ASCE 7-16 Table/Figure	2019 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2	Figure 22-1	Figure 1613*A*.3.1(1)	S_S	0.408 g
1.0 Period MCE	Figure 22-2	Figure 1613*A*.3.1(2)	S_1	0.21 g
Soil Class	Table 20.3-1	Section 1613*A*.3.2	Site Class	C
Site Coefficient	Table 11.4-1	Table 1613*A*.3.3(1)	F_a	1.3
Site Coefficient	Table 11.4-2	Table 1613*A*.3.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16*A*-37	S_{MS}	0.531 g
	Equation 11.4-2	Equation 16*A*-38	S_{M1}	0.315 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16*A*-39	S_{DS}	0.354 g
	Equation 11.4-4	Equation 16*A*-40	S_{D1}	0.21 g
Seismic Design Category	Table 11.6-1	Section 1613*A*.3.5(1)	Risk Category I to IV	D
	Table 11.6-2	Section 1613*A*.3.5(2)	Risk Category I to IV	D

* Calculated using USGS computer program U.S. Seismic Design Maps and the site latitude and longitude.
MCE – Maximum Considered Earthquake
g – Acceleration due to gravity

The site modified peak ground acceleration PGA_M (Equation 11.8-1, ASCE 7-16) is 0.21 g.

Site-specific ground response and ground motion hazard analyses, and/or time history analyses were not part of our work scope.

PRIMARY SEISMIC HAZARDS

Seismic Hazards

No active or potentially active faults are known to cross the project site as indicated by the published geologic maps or aerial photographs reviewed for this project. The project site is not located within an Earthquake Fault Zone, or designated seismic hazard zone; therefore, a site-specific ground motion analysis is not warranted. The project site is located within an area of moderate seismic activity; however, design of the structure in conformance with the 2019 edition of the California Building Code (Title 24 of the California Code of Regulations, Chapter 16A), should be sufficient to prevent significant damage from ground shaking during seismic events resulting from movement on any of the faults or fault systems discussed in this report.

Seismic Sources

Several faults exhibiting activity in the Quaternary time are mapped within 62 miles (100 kilometers) of the project site. These faults and fault systems, their Maximum Magnitude Earthquakes ($M_w$max) and distances to the project site are listed within the FAULTING section of this report. Hazard deaggregation indicates that the causing faults contributing to the estimated site PGA are Hunting Creek-Berryessa Fault System, Great Valley Fault System, Foothills Fault System, and Green Valley Fault.

The Foothills Fault System is regarded as Geodetic zone of distributed shear (C Zone) that is based on poorly constrained Quaternary slip rates across the Bear Mountain and Melones Fault Zones (CDMG, 1996; Woodward-Clyde Consultants, 1978). Wakabayashi and Smith (1994) describe the Foothills Fault Zone as lacking evidence of active crustal shorting and note that deformation along the eastside of the Central Valley is extensional or transtensional.

The Great Valley Fault System extends from the southern San Joaquin Valley in Kern County northward into Tehama County, and serves as the boundary between the Coast Range and the Great Valley Geomorphic Provinces of California. It is characterized by a zone of low-angle, or blind thrust, and reverse faults that do not rupture the ground surface during sizable earthquake events. Although not exposed at the surface, regional studies have suggested that the Great Valley Fault System may be comprised of 18 to 25 segments that range in length from 7 to 35 miles (11.2 to

56.3 kilometers) – with most segment lengths measuring between 12 and 19 miles (19.3 to 30.6 kilometers). Several notable earthquake events have occurred along segments of the Great Valley Fault System, including: the 1892 Mw 6.4 and 6.2 Winters-Vacaville earthquakes, 1983 MW 6.5 Coalinga earthquake, and the 1985 MW 6.1 Kettleman Hills earthquake.

The Hunting Creek-Berryessa is a Holocene dextral strike-slip fault system associated with the larger San Andreas fault system. The Hunting Creek-Berryessa fault system extends from the vicinity of Wilson Valley south-southeast to the Cedar Roughs area west of Lake Berryessa. The fault zone is divided from north to south into the Wilson, Hunting Creek, and Lake Berryessa sections. The Hunting Creek-Berryessa fault system is expressed as a zone of discontinuous fault traces as much as 3.5 km wide. The Hunting Creek-Berryessa fault system locally is delineated by geomorphic evidence of Holocene dextral strike-slip displacement, predominantly along the Hunting Creek fault, which comprises the Hunting Creek section (Bryant, 1982). An investigation by Steffen, Robertson, and Kirsten, and Woodward-Clyde Consultants (1983) demonstrated latest Pleistocene and probable Holocene displacement along traces of the Hunting Creek fault. Slip rates of between 1 and 5 mm/yr are assigned for these fault sections.

The Green Valley Fault is a Holocene dextral strike slip fault. It is characterized by aseismic creep, and has been monitored by Galehouse (1992, 1999) since 1984. Detailed reconnaissance level mapping exists for most of the fault, based on geologic and geomorphic data (Weaver, 1949; Dooley 1973; Sims, and others 1973; Frizzell and Brown, 1976; and Bryant 1982, 1992). Several site-specific studies in compliance with the Alquist-Priolo Act (Hart and Bryant, 1997) have documented the location and approximate age of most recent faulting. Preliminary data from the Lopes Ranch paleoseismic project site indicates the Green Valley Fault has produced multiple surface-rupturing events in the last 2700 years, and has a minimum late Holocene dextral slip rate of 3.8 mm/yr to 4.8 mm/yr based on 1.2 – 1.5 meters (3.9 – 4.9 feet) dextral offsets within a 310 year old paleochannel (Baldwin and Lienkaemper, 1999). Geomorphic expressions of the Green Valley Fault include closed depressions, ponded alluvium, dextrally offset drainages, linear troughs, sidehill benches, and scarps in young alluvium (Dooley, 1973; Frizzell and Brown, 1976; Bryant, 1982, 1992). Bryant (1982, 1991) estimated a long-term Quaternary slip rate of 3 mm/yr, based on unconstrained dextral separation of Pliocene Sonoma Volcanics mapped by Sims, et al (1973).

Surface Fault Rupture

No known faults are mapped crossing the immediate vicinity of the site. The site does not lie within an Earthquake Fault Zone as currently designated by the State of California and no evidence of surface faulting was observed during our historical aerial photography review, site reconnaissance, or geotechnical investigation. It is our opinion that the potential of fault-related surface rupture at the site is low.

Seismic Risk

Hazard deaggregation indicates that the causing faults contributing to the estimated site PGA are Hunting Creek-Berryessa Fault System (M=7.1 event), Great Valley Fault System (M=6.7 event), Foothills Fault System (M=5.5 event), and Green Valley Fault System (M=6.7 event).

SECONDARY HAZARDS

Liquefaction

Liquefaction is a soil strength and stiffness loss phenomenon that typically occurs in loose, saturated cohesionless soils as a result of strong ground shaking during earthquakes. The potential for liquefaction at a site is usually determined based on the results of a subsurface geotechnical investigation and the groundwater conditions beneath the site. Hazards to buildings associated with liquefaction include bearing capacity failure, lateral spreading, and differential settlement of soils below foundations, which can contribute to structural damage or collapse. The site is not located within a State Designated Seismic Hazard Zone for liquefaction.

Considering the historic depth to groundwater (deeper than 100 feet) and site soil conditions (volcanic rock), the potential for soil liquefaction beneath the site is very low and is not considered influential to the site.

Cyclic Softening of clay and clay-like soils

Cyclic softening of clay soils commonly understood as the reduction in soil stiffness and strength due to repeated cyclic loading. This phenomenon is typically observed in soft, saturated soils with Plasticity Index (PI) above 7. The site is underlain predominantly by volcanic rock. The historical depth to groundwater is deeper than 100 feet. Layers of soft, low strength clays were not exposed in our borings or test pits. Therefore, it is our opinion the potential for cyclic softening occurring beneath the site is very low.

Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of gently sloping ground as a result of pore pressure build up or liquefaction in a shallow underlying deposit during an earthquake. Lateral spreading usually occurs on gently sloping ground exposed to a slope or free face. Based on very low potential for liquefaction beneath the site, it is our opinion that the potential for lateral spreading at the site is very low.

Seismically Induced Settlement and Dry Sand Seismic Settlement

The site is not located in a Seismic Hazard Zone for liquefaction as designated by the state of California, which delineates areas of historical occurrence of liquefaction or local geological, geotechnical and groundwater conditions indicating a potential for permanent ground displacement. Based on the fact that the site is underlain by volcanic rock and considering that the loose undocumented fill soils will be recompacted, it is our opinion the potential for site seismically induced ground subsidence or dry sand seismic settlement is very low.

Subsidence & Hydrocollapse

Regional subsidence occurs when large areas of land sink in response to withdrawal of groundwater, petroleum, or natural gas. According to a review of the *Areas of Land Subsidence in California Map* (California Water Science Center), the site is not currently located within an area of land subsidence from groundwater pumping, peat loss, or oil extracting our opinion, the site is not located in an area subject to high subsidence, due to the absence of factors and conditions needed to cause subsidence (excessive withdrawal of groundwater, petroleum, or natural gas).

Providing, that the site is underlain predominantly by volcanic rock, the potential for hydrocollapse of on-site soils is very low.

Landslides and Slope Stability

The site is not located in a Landslide Hazard Zone as designated by the State of California. Considering the gently sloping topography, the potential for development of the landslides or slope instability is negligible. Furthermore, we do not anticipate any permanent site excavations into the underlying rock, and all fills constructed during grading will be properly keyed, benched and compacted as recommended in this report.

Tsunami

The project site is located approximately 2½ miles from the Folsom Lake; therefore, the potential for tsunamis influencing the site is low.

Seiche

The project site is located approximately 2½ miles from the Folsom Lake; therefore, the potential for seiches influencing the site is low.

Flood/Dam Inundation

The site is not located within a Special Flood Hazard Area (SFHA) as designated by the Federal Emergency Management Agency (FEMA). According to the Flood Insurance Rate Maps (FIRM), Panel 0177H, Map Number 06067C0177H, published by FEMA, with an effective date of January 11, 2020, the proposed site improvements lie within Zone X, Areas to be determined to have the 0.2% annual chance floodplain. It is our opinion the site has a minimal risk of flooding (Figure 15).

Review of the maps published by Sacramento Area Flood Control Agency indicates the site is not located in the area of inundation due to levee failure.

Review of the Dam Breach Inundation Map Web Publisher, maintained by Department of Water Resources, indicates that the site is not located in the area prone to inundation due to the dam failure.

According to the *Safety Element of County of Sacramento General Plan* the project site is located in the Folsom Dam failure inundation area.

Volcanic Hazard

Review of the USGS Map of Potential Hazards from Future Volcanic Eruptions in California (Miller, 1989), shows the project site is approximately 80 miles (130 kilometers) east-southeast of Clear Lake Volcanic Area, 125 miles (200 kilometers) northwest of the Mono Lake-Owens Valley Volcanic Area, 125 miles (200 kilometers) south of the Mount Shasta, Medicine Lake Highland, and Lassen Peak Volcanic Area. The closest know area of the Quaternary volcanic eruption (Sutter Buttes) is 46 miles (74 kilometers) northwest of the site. Based on the above information, it is our opinion that a potential for volcanic hazard affect the site is very low.

Naturally Occurring Asbestos (NOA)

Asbestos is the generic term for the naturally occurring fibrous (asbestiform) varieties of six silicate minerals. Asbestos also refers to an industrial product obtained by mining and processing deposits of asbestiform minerals. According to California Geological Survey Open-File Report 2000-19, *A General Location Guide for Ultramafic rocks in California-Areas More Likely to Contain Naturally Occurring Asbestos* (2000), and the USGS Open-File Report 2011-1188, *Reported Historic Asbestos Mines, Historic Asbestos Prospects, and Other Natural Occurrences of Asbestos in California* (2011), the project site does not lie within an area mapped as containing Naturally Occurring Asbestos (NOA) or ultramafic rock in outcrop.

Based on the *Relative Likelihood for the Presence of Naturally Occurring Asbestos In Eastern Sacramento County, California* (CGS Special Report 192, 2006), the site is located in an area mapped as "Moderately Likely to Contain NOA".

The *Geotechnical and Geologic Hazards Investigation* prepared for the Folsom Lake College Athletic Complex by Geocon, dated January 2010, included the sampling and testing of three samples (fill soil, residual soil, and rock) for the presence of asbestos using polarized light microscopy with CARB 435 preparation with a target analytical sensitivity of 0.25%. Their screening level testing did not preclude the possibility of NOA being present at the site; however, asbestos minerals were not present in any of the samples tested.

Radon Gas

Sections 307 and 309 of the Indoor Radon Abatement Act of 1988 (IRAA) directed EPA to list and identify areas of the U.S. with the potential for elevated indoor radon levels. EPA's Map of Radon Zones assigns each of the 3,141 counties in the U.S. to one of three zones based on radon potential. Sacramento County and the project site are located in Zone 3 for radon potential. Zone 3 counties have a predicted average indoor radon screening level less than two pCi/L and are indicated to have a Low Potential for radon.

CONCLUSIONS

Foundation and Structural Support

Of special concern for site development and structural support are the presence of loose, undocumented fill soils, presence of the underground utilities within the proposed building pad, potential for significant differential settlement of the foundations spanning from engineered fill/native soils onto rock, and as well as differential settlement of the foundations bearing on engineered fill soils with differential thicknesses of more than five feet.

Our field investigation indicates that undocumented fill soils are loose and were not placed as engineered fill. In our opinion, these soils must be over-excavated to expose native soils/rock and the excavations backfilled, as necessary to achieve final design grades, with engineered fill to provide adequate and uniform support for the planned structure and other improvements.

In addition, the area proposed for the structure contains existing underground utilities; therefore, proper clearing and removal of the utilities and proper backfilling of excavations is very important to provide adequate and uniform structural support. Removal of the underground utilities will disturb the surface and near-surface soils creating loose and variable soil conditions; therefore, we will recommend all disturbed and/or loose soils within building pad and all site structural areas be over-excavated and replaced with properly moisture conditioned and compacted engineered fill to promote more uniform support for the planned slab-on-grade structures, foundations, pavements, concrete flatwork, and associated improvements.

Grading plans were not available at the time we prepared this report and the depths of cuts and fills at the site were not known. To help reduce the potential for differential settlements we will recommend that foundations for the proposed structure be supported either entirely on weathered rock, or entirely on a minimum of 2 feet of engineered fill. If second options is selected, we will recommend that the differential thickness of the engineered fill below the foundations should not exceed five feet in 40 linear feet.

Specific recommendations for processing and re-compaction are presented in the SITE PREPARATION AND OVER-EXCAVATION section of this report.

Our work indicates that weathered rock and engineered fill, properly placed and compacted in accordance with the recommendations of this report, will be capable of supporting the proposed structure and associated improvements.

Provided the pad preparation and foundation construction are performed as recommended, we estimate total static settlements of foundations to be one inch with differential settlements to be approximately ½-inch in 40 linear feet. In our opinion, the majority of any initial static settlements will occur during construction. We do not anticipate long-term secondary static settlements to occur, based on the soil conditions and the recommended re-compaction.

EXPANSIVE SOILS

The results of our subsurface exploration and laboratory testing program indicate the on-site soils and completely weathered rock exhibit low expansion potential. These soils/rock, when present within the upper portion of the building pad, are capable of exerting low to moderate expansion pressures on building foundations, interior slabs-on-grade and exterior flatwork with variations in soil moisture content, which must be considered in design and construction. Specific recommendations to reduce the effects of expansive soils are presented in this report.

Results of Expansion Index laboratory testing (ASTM 4829) are presented on Figures A1 through A3.

SUITABILITY OF ON-SITE SOILS FOR USE AS FILL

The on-site soils and rock are considered suitable for use as engineered fill materials, provided these materials are free from concentrations of organic debris (roots and root balls), over-size rock, rubble, debris, rubbish, or other deleterious materials and are at the proper moisture content for compaction.

The weathered rock will be suitable for use in engineered fill construction provided it is broken into pieces less than 12 inches in maximum dimension and thoroughly mixed with soil and smaller rocks. Particles larger than 6 inches in size should not be used within five feet from the finished building pad. Particles larger than 6 inches, but less than 12 inches can be used in fills placed deeper than 5 feet below finished grade. Rock fragments greater than 12 inches in diameter should be removed from fill material, prior to placement.

We anticipate the rock will tend to excavate in large pieces during grading, which will require processing/crushing, prior to placement as fill. However, large "floaters" of basalt should be anticipated during pad excavation. The method of processing will be dependent upon the contractor's ability to break the rock into pieces suitable for use as fill and could involve breaking the rock with hydraulic hammers attached to large excavators. Large pieces of rock can sometimes be broken down to suitable sizes with large dozers or large compactors for use as fill, depending on the equipment and methods used

Boulders larger than one foot in diameter may be excavated during grading which would need to be broken down to a suitable size prior to placement as engineered fill. Boulders will be very difficult to break into sizes suitable for use as fill with dozers and compactors, and may require specialized equipment. Large boulders that cannot be broken down into suitable sizes for fill should be removed from the site. If may be possible to place large rocks (up to 24 inches on size) within deeper fills; however, that can only be determined by the Geotechnical Engineer based on review of the final grading plans and final design of the structure and its foundations.

If encountered, removal of rubble, debris, and organic debris from on-site soils may require laborers handpicking the fill materials, and/or screening prior to allowing the soils to be re-used as fill.

On-site soils should not be used within the 12 inches from the finished building pad subgrade due to the shrink/swell potential.

Excavation Conditions

Review of the data contained within the *Rippability and Vs100 Class Determination Report*, prepared by Petralogix Engineering, Inc, dated January 11, 2019, indicates that the p-wave velocity was generally below approximately 6,000 ft/sec to an approximate depth of 15 feet bgs. Based on the *Caterpillar Performance Handbook, Edition 31*, on-site basalt rock with velocities less than about 6000 ft/sec is rippable with a Caterpillar D8R ripper equipped with multi or single shanks. In general, p-wave velocity was between 6,000 ft/sec and 9,000 ft/sec at depths of between approximately 15 and 20 feet bsg. On-site basalt rock with velocities of 6000 to 9000 ft/sec is marginally rippable with D11R ripper equipped with multi or single shanks. Based on the data, below about 15 depth the materials appear more variable and depending on location, the p-wave velocity may increase rapidly and may exceed 10,000 ft/sec (non-rippable). Therefore, caution should be exercised when sizing equipment and excavation methods for depths below 15 feet below ground surface.

In general, we anticipate undisturbed soil sidewalls and recompacted on-site soils for most site excavations will remain stable at near-vertical inclinations for short periods of time without significant caving, unless perched water and/or seepage is encountered, or saturated and/or low cohesion sandy soils are encountered or the exposed soils are allowed to dry. Excavations encountering perched water and seepage will be susceptible to sloughing or caving upon excavation or if left open for an extended period of time requiring sloped excavations and other stabilization methods.

Excavations deeper than five feet that will be entered by workers should be sloped and/or braced in accordance with current OSHA regulations. The contractor must provide an adequately constructed and braced shoring system in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground. If material is stored or heavy equipment is operated near an excavation, stronger shoring would be needed to resist the extra pressure due to the superimposed loads.

SOIL CORROSION POTENTIAL

Representative soil samples were submitted to Sunland Analytical Lab, Inc. for testing to determine pH, resistivity, and sulfate and chloride concentrations to help evaluate the potential for corrosive attack upon reinforced concrete. Results of the corrosion testing performed by Sunland Analytical Lab are summarized in the Table 3.

Table 3 – Soil Corrosivity Testing

Analyte	Test Method	Sample Identification	
		TP7 @ 1'	TP3 @ 3'
Soil pH	CA DOT 643 Modified*	6.13	6.26
Minimum Resistivity	CA DOT 643 Modified*	3,480 Ω-cm	3,750 Ω-cm
Chloride	CA DOT 417	1.9 ppm	2.0 ppm
Sulfate	CA DOT 422	4.7 ppm	4.2 ppm

* = Small cell method
Ω-cm = Ohm-centimeters
ppm = Parts per million

The California Department of Transportation Corrosion Technology Section, Office of Materials and Foundations, Corrosion Guidelines Version 2.0, November 2012, considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples collected: a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or the pH is 5.5 or less. Based on this criterion, the on-site soils are not considered corrosive to reinforced concrete. Table 19.3.1.1 – Exposure Categories and Classes, American Concrete Institute (ACI) 318-19, Section 19.3, as referenced in Section 1904.1 of the 2019 CBC, indicates the severity of sulfate exposure for the samples tested is "not a concern". Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover is maintained over the reinforcement.

Mid Pacific Engineering, Inc. are not corrosion engineers. Therefore, to further define the soil corrosion potential at the site, or to determine the need or design parameters for cathodic protection or grounding systems, a corrosion engineer should be consulted.

Import fills, if used for construction, should be sampled and tested to verify the materials have corrosion characteristics within acceptable limits and generally should be similar to the tested on-site soils.

PAVEMENT SUBGRADE QUALITIES

Based on the results of laboratory testing, on-site soils and completely weathered rock when excavated consist of sandy, clayey silts and silty, clayey sands which when tested in accordance with California Test (CT) 301 are relatively fair quality materials for the support of asphalt concrete pavements possessing Resistance ("R")-values of about 19, (see Figure A5). Based upon the test results, and the natural variability of the soils at the site, it is our opinion that an R-value of 15 is considered appropriate for design of pavements at this site. The underlying rock may possess higher subgrade support qualities; however, that can only be determined after a careful review of final grading plans and additional testing.

PERMANENT GROUNDWATER

Due to the anticipated depth to groundwater, permanent groundwater should not be a significant factor in the design and construction of the proposed improvements at this site.

SEASONAL WATER

The near-surface soils also may be in a near-saturated condition during and for a significant time following the rainy season due to rain water being unable to penetrate through the clayey soils below existing site grade. Earthwork operations attempted following the onset of the rainy season and prior to prolonged drying will be hampered by high soil moisture contents. Heavy, prolonged rainfall events will promote high soil moisture contents and increase the potential for trapped water over impermeable soil and rock layers that could further affect grading operations. If grading operations are to proceed shortly after the rainy season, and before prolonged periods of warm dry weather, the near-surface soils and soils to be used as engineered fill including trench backfill may be at moisture contents where significant and prolonged aeration or lime-treatment may be required to dry the soils to a moisture content where the specified degree of compaction can be achieved. The contractor should anticipate the additional time and effort necessary to achieve a compactable moisture content.

Perched or seepage water may be present within excavations, depending upon the time of year when construction takes place. The need for dewatering of excavations can best be determined during site work when subsurface conditions are fully exposed.

Seasonal moisture and landscape irrigation will result in high soil moisture contents below interior floor slabs throughout their lifetime. Moisture vapor penetration resistance should be a significant consideration in design and construction of interior floor slabs.

Erosion and Winterization

The near-surface on-site soils generally consist of silty sands and sandy silts. In our opinion, the undisturbed soils may be susceptible to erosion by surface run-off that occurs during intense rainfall. As a minimum, erosion control measures including placement of straw bale sediment barriers or construction of silt filter fences in areas where surface run-off may be concentrated would be prudent. The project civil engineer should develop a site-specific erosion and sediment control plan based upon their site grading and drainage plan and the anticipated construction schedule.

All excavation and fill (if any) slopes should be protected from concentrated storm water run-off to minimize potential erosion. Control of water over the slopes may be accomplished by constructing small berms at the top of the slope, constructing V-ditches near the top of the slope, or by grading the area behind the top of the slope to drain away from the slope. Ponding of surface water at the top of the slope or allowing sheet flow of water over the top of the slope should be avoided.

RECOMMENDATIONS

The project is in a preliminary stage of development; therefore, we consider it essential that our office review site, grading, and structural foundation plans to verify the applicability of the following recommendations, perform additional investigations, and provide supplemental recommendations, as conditions dictate.

Our recommendations are contingent upon our office performing the recommended plan reviews and providing a letter indicating that the recommendations of this report are applicable to the proposed construction. Grading plans were not available; therefore, we have assumed that excavations and fills of up to ten feet for development of the planned improvements. The recommendations contained in this report are based upon this assumption.

Based on subsurface conditions encountered in our borings, we anticipate site over-excavation depths of up to five feet bsg will be required to remove undocumented fill soils to expose undisturbed native soils. The exposed subgrade should be scarified to a depth of 12 inches, properly moisture conditioned, and compacted in accordance with the recommendations of this report. The depth of over-excavation will vary across the site both in lateral and depth dimensions, depending on exposed conditions; therefore, we recommend construction bid documents contain a unit price (price per cubic yard) to compensate for the variations in the lateral extents and depths of over-excavation and engineered fill construction.

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and spring months, and will not be compactable without drying by aeration or the addition of lime (or a similar product) to dry the soils. Should the construction schedule require work to continue during the wet months, additional recommendations should be provided by the Geotechnical Engineer retained to provide services during project construction.

SITE CLEARING

Initially, all structural areas of the site should be cleared of underground utilities scheduled for removal and vegetation, and other deleterious materials to expose firm and stable soil conditions as identified by our on-site representative. Backfill of the existing utilities located in the structural area, outside the building pad, should be tested by our representative to determine proper degree of the compaction.

Where practical, the clearing should extend a minimum of five feet beyond the limits of the proposed improvements and structural areas of the site. Existing underground utilities located within proposed building pad should be completely removed and/or rerouted as necessary. Utilities located outside the building area should be properly abandoned (i.e., fully grouted provided the abandoned utility is situated at least 2½ feet below the final subgrade level to reduce the potential for localized "hard spots").

Adequate removal of debris, rubble, and particles large than 12 inches in size may require laborers and handpicking to clear the subgrade soils to the satisfaction of our on-site representative. Depressions resulting from clearing operations and any other loose, disturbed, soft or otherwise unstable materials should be removed to expose a firm, undisturbed soils prior to backfilling with properly placed and compacted engineered fill to restore the areas back to the required grades.

Remaining areas should be stripped of surface vegetation and organically contaminated topsoil; strippings may be stockpiled for later use in landscape areas or disposed of off-site. Strippings should not be used in general fill construction, but may be used in landscaped areas, provided they are kept at least five feet from the building pads, exterior flatwork and pavements, and moisture conditioned and compacted. *Strippings should not be used in landscaped berms that will support sound walls, retaining walls, concrete flatwork, or other at-grade structure.*

It is essential that our representative be present during clearing operations to verify adequate removal of existing utilities, and determine the need for over-excavation of disturbed soil areas. It is essential that excavations resulting from clearing operations be left as shallow dish-shaped depressions for proper location and to allow proper access with compaction equipment during grading operations. If clearing and removal of structures takes place without direct observation by the Geotechnical Engineer, deeper cross-ripping and/or over-excavation of the disturbed areas, building pad or structural areas affected will be required.

Site Preparation and Over-Excavation

Provided MPE is present during clearing operations and the excavations for removal of subsurface elements are left as dish shaped depressions so that our representative can verify adequate and complete removal, pad preparation can proceed as recommended below. If this is not the case and MPE is not present during site clearing operations or if excavations are backfilled without our observation and testing, all building and structural pads (building/structural area plus five feet beyond) will require deeper processing or over-excavation and re-compaction.

To reduce the potential for differential settlement of building foundations, building pads constructed partially by cut and partially by fill that exceed five feet in thickness, and fill differentials that exceed five feet should be avoided. Building pads with either of these conditions may require over-excavation so that the fill differential across the building pad does not exceed five feet. We should review the grading plans and work with the contractor to determine the areas, if any, requiring over-excavation. It should be noted that such sub-excavations will require excavation of hard rock, depending on final grading plans. Due to the existence of rocky materials at the site, we recommend a performance specification be followed for the compaction of rocky materials. For densities comparable to 90 percent relative compaction, subgrades consisting of rocky materials should be compacted by at least three complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor, to the satisfaction of our on-site representative. For densities comparable to 95 percent relative compaction, rocky materials should be compacted by at least five complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor. *The number of passes is considerable a minimum and passes should be added as required by the Geotechnical Engineer to achieve a stable and unyielding subgrade condition.* One complete coverage is defined as the effort necessary to assure that every square foot of the subgrade is compacted.

Undocumented Fill Soils Over-Excavation

Following site clearing operations, all structural areas (building pad, pavement, exterior flatwork) and areas to receive engineered fill should be over-excavated to remove the undocumented fills to expose native undisturbed soils, as identified by the Geotechnical Engineer. The over-excavations should extend a minimum of five feet horizontally beyond the proposed structural areas and building lines. The bottom of all over-excavations, where native soils or completely weathered rock are present, should be ripped and cross-ripped to a minimum depth of 8 inches, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the ASTM D1557 maximum dry density, or if rocky, subgrades should be compacted in accordance with the performance specification.

Building Pad Over-Excavation

If the option to support the proposed structure entirely on engineered fill is selected, the entire building pad should over-excavated to provide a minimum of two feet of engineered fill below the foundations. Additional over-excavation will be required to reduce the differential thickness of the fill below the foundations to within five feet in 40 linear feet. The over-excavations should extend a minimum of five feet horizontally beyond the proposed structure lines, and should include areas of exterior columns, areas supporting exterior flatwork and pavements within five feet of the proposed structure. The bottom of all over-excavations, where native soils or completely weathered rock are present, should be ripped and cross-ripped to a minimum depth of 8 inches, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the ASTM D1557 maximum dry density, or if rocky, subgrades should be compacted in accordance with the performance specification.

If the option to support the proposed structure entirely on weathered rock is selected, the entire building pad should over-excavated to provide a minimum of 12 inches of granular, non-expansive engineered fill below the proposed slab-on-grade. The bottom of all over-excavations, where native soils or completely weathered rock are present, should be ripped and cross-ripped to a minimum depth of 8 inches, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the ASTM D1557 maximum dry density, or if rocky, subgrades should be compacted in accordance with the performance specification.

Since the site is underlain by weathered rock materials and depending on final grading plans, consideration should be given to removing the undisturbed rock materials to a depth of at least two feet below final pad grade (or to the depth of the deepest utilities) with subsequent placement of engineered fill to achieve final pad elevations. Although removal of the rock materials is not required for support or stability reasons, it may prove to be economical in consideration of time and effort saved during foundation and utility construction.

Other Structural Areas outside the Undocumented Fill Area and Building Pad

All other structural areas, where native soils or completely weathered rock are present, should be ripped and cross-ripped to a minimum depth of 12 inches, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the ASTM D1557 maximum dry density, or if rocky, subgrades should be compacted in accordance with the performance specification. The extent of scarification and compaction should extend a minimum of three feet horizontally beyond the proposed structural improvements lines. The compacted subgrades must be in a stable and unyielding condition for proper structural support.

MPE should review the final plans to verify the applicability of these recommendations and determine the need for revised recommendations.

Compaction operations should be undertaken with a heavy, self-propelled, sheepsfoot compactor (Caterpillar 825, or equivalent-size compactor) and should be performed in the presence of our representative who will evaluate the performance of the subgrade under compactive load and identify loose or unstable soils that could require additional excavation and/or compaction. Loose, soft, or unstable soils, as identified by our representative in the field, should be cleaned out to firm, undisturbed and stable soils, as determined by our representative, and should be restored to grade with engineered fill compacted in accordance with the recommendations of this report. Difficulty in achieving subgrade compaction or unusual soil instability may be indications of loose fill associated with past subsurface items. Should these conditions exist, the materials should be excavated to check for subsurface structures and the excavations backfilled with engineered fill. We recommend construction bid documents contain a unit price (price per cubic yard) for all excess excavation due to loose, soft, or unsuitable materials and replacement with engineered fill.

ENGINEERED FILL CONSTRUCTION

On-site soils and rock materials that are predominately less than 12 inches in maximum diameter may be used as engineered fill if they do not contain debris, organics or other deleterious materials. Rocks greater than 12 inches in diameter should be broken into pieces less than 12 inches in maximum dimension if used as engineered fill material. Excessive concentrations of rocks should be avoided. Rocks should be spread and thoroughly mixed with soils to reduce the chances of voids being created within fill. On-site soils and rock materials should be uniformly moisture conditioned and compacted to the satisfaction of our on-site representative in accordance with the recommendations presented in the following paragraphs.

Although it may not be possible to eliminate all rock greater than 6 inches from fill material without screening the fill material prior to placement and compaction, we recommend that the use of rock exceeding 6 inches in diameter be minimized in the upper five feet of fill placed within a building pad. Large rocks, greater than 6 inches in size but no more than 12 inches in size, should be placed in deeper fills, as approved by our on-site representative, or should be removed from the fill.

It should be noted that without the use of screening, it is almost impossible to assure a certain particle size within the fill; therefore, the possibility exists that particle sizes larger than those specified could be incorporated into the fill. This could result in larger (wider) than the planned foundation and/or utility trench excavations. The owner and contractor should discuss the implications of larger particle sizes in the fill versus the increased cost for screening and/or the use of select fill materials.

Fill soils should be placed in level lifts not exceeding six inches in compacted thickness, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the ASTM D1557 maximum dry density. Fill soils placed deeper than five feet below finished subgrade should be compacted to at least 95 percent of the ASTM D1557 maximum dry density. Rocky fills should be placed in maximum 12-inch lifts with each lift being compacted by least three complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor for densities comparable to 90 percent relative compaction. For densities comparable to 95 percent relative compaction, each lift of rocky materials should be compacted by at least five complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor. *The number of passes is considerable a minimum and passes should be added as required by the Geotechnical Engineer to achieve a stable and unyielding subgrade condition.* One complete coverage is defined as the effort necessary to assure that every square foot of the subgrade is compacted. Compactive effort should be applied uniformly across the full width of the fill. Large rocks that cannot be properly incorporated into the engineered fill should be removed from the fill.

Fill placed on sloping ground steeper than five horizontal to one vertical (5:1) should be benched during placement of engineered fill. Each lift should be benched into the slope and should consist of a level terrace excavated at least twelve inches into the slope. For every three feet of vertical height of fill, a larger bench should be constructed, extending at least five feet into the existing slope. Taller slopes may require a wider bench placed at mid-slope height. The Geotechnical Engineer retained for construction should determine the need for, and depth and dimensions of, a base key upon which to begin fill construction based on a review of the final plans and site conditions. In general, base keys should be at least 15 feet wide and extend at least two feet into competent weathered rock. Revised recommendations may be needed based on final plans.

Import fill materials, if required, should be granular in nature, with a Plasticity Index not exceeding 15 and a maximum particle size of 6 inches. Imported fill materials, if required, ideally should be granular with a Plasticity Index of 15 or less; Expansion Index of 20 or less; and, free of particles greater than three inches in maximum dimension. Clean, open graded gravels (such as crushed rock or pea gravel) and other such materials are *not* acceptable for fill construction. The contractor also should supply appropriate documentation for imported fill materials indicating the materials are free of known contamination and have corrosion characteristics within acceptable limits. Imported soils should be tested and approved by the Geotechnical Engineer office prior to being transported to the site.

The upper 12 inches of final building pad subgrades and exterior flatwork should consist of 12 inches of approved imported, or on-site, non-expansive granular materials, or Class 2 aggregate base.

The upper 12 inches of final building pad subgrades should be thoroughly moisture conditioned and uniformly compacted to at least 90 percent of the ASTM D1557 maximum dry density. or if too rocky to test, by at least three complete coverages of a Caterpillar 825 compactor (or equivalent), regardless of whether final grade is achieved by filling, excavation, or is left at the existing grade.

The upper six inches of pavement subgrades and exterior slab subgrades supporting vehicle loadings should be uniformly compacted to at least 95 percent of the ASTM D1557 maximum dry density, or if too rocky to test by at least five complete coverages with a Caterpillar 825 (or equivalent) compactor, and must be stable under construction traffic prior to placement of aggregate base. Final subgrade processing and compaction should be performed just prior to placement of aggregate base, after construction of underground utilities is complete.

Permanent excavation and fill slopes should be constructed at a gradient of two horizontal to one vertical (2: 1) or flatter. Slopes constructed by fill and excavation should be protected from erosion by suitable methods such as hydroseeding prior to the rainy season. Brow ditches at the top are very effective at redirecting runoff water away from slopes.

Site preparation should be accomplished in accordance with the recommendations of this section and the *Guide Earthwork Specifications* provided in Appendix B. It is essential that a representative from our office be present on a nearly full-time basis during site preparation and all grading operations to verify complete removal of undocumented fills and/or unstable soil deposits, to observe the earthwork construction, perform compaction testing and verify compliance with our recommendations and the job specifications.

UTILITY TRENCH BACKFILL

Utility trench backfill should be mechanically compacted in maximum six-inch lifts. Trench backfill should be brought to uniform moisture content above the optimum moisture and each lift mechanically compacted to at least 90 percent of the maximum dry density. The upper six inches of trenches in pavement areas should be compacted to at least 95 percent of the maximum dry density. Rock over 12 inches in diameter should not be used as trench backfill material and rock over three inches in diameter should not be used as initial backfill to avoid impact damage to utility lines. Particle size for backfill within the building pad should be limited to six inches in size or less; smaller particle sizes may be required depending on the equipment being used for compaction. Jetting of trench backfill as a means of compaction is not acceptable. We recommend that native soil be used as trench backfill within the perimeter of the building foundations to help minimize soil moisture variations beneath the structure. The native soil backfill should extend at least three feet horizontally beyond perimeter foundation lines. The upper 12 inches of backfill material for trenches within building pad and slab-on-grade subgrades should be non-expansive granular soils or aggregate base.

Rocky backfill material should be properly moisture conditioned and uniformly compacted in six to 12-inch lifts using mechanical compaction methods. The lift thickness and number of passes to achieve proper compaction will depend on the size of the rocky material and the type of compaction equipment used.

We recommend that underground utility trenches that are aligned nearly parallel with foundations be at least three feet laterally from the outer edge of foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at a 1:1 (horizontal to vertical) inclination below the bottom of the foundations. In addition, trenches parallel to foundations should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

Pipe bedding, shading and trench backfill and compaction within municipal streets should conform to jurisdictional requirements.

Foundation Design

We are providing design soil values for the analysis of proposed foundations, and suggested minimums for dimensions, but only from a Geotechnical Engineering perspective. The project Structural Engineer should determine final foundation design width and depth dimensions as well as concrete strength and reinforcing requirements, based on their specific structural design, which should include an appropriate factor of safety applied to the overall design.

Total and differential settlements (static and seismic) of 1-inch and ½-inch in 40 linear feet, respectively, should be anticipated for the design of the proposed foundations.

Foundations for the proposed structure should be supported either entirely on weathered rock, or entirely on a minimum of two feet of engineered fill. If second option is selected, the differential thickness of the engineered fill below the foundations should not exceed five feet in 40 linear feet.

Provided the building pad is over-excavated and re-compacted as recommended, the proposed structure may be supported upon continuous and/or isolated spread foundations extending at least 18 inches into the prepared building pad, or at least 18 inches below lowest adjacent soil grade, whichever is deeper. Continuous foundations should be at least 15 inches wide; isolated foundations should be at least 24 inches wide. Foundations must be continuous around the perimeter of the building to help minimize moisture migration beneath the structure.

The following bearing pressure values may be used for shallow spread and continuous foundation design. The weight of foundation concrete extending below grade may be disregarded in sizing computations. The recommended factors of safety for various Allowable Stress Design (ASD) load combinations are presented in Table 4 below for the design in accordance with 2019 CBC 1605A.1.1, assuming the structure would be designed for a system overstrength factor (Ω_0) of 3. For foundations designed using ASD, the factor of safety for soil bearing pressure shall not be less than the overstrength factor.

Table 4 – Allowable Bearing Pressures

Load Condition	Ultimate Bearing Pressure (psf)	Minimum Factor of Safety	Allowable Bearing Pressure (psf)
Dead plus Live Loads	12,000	4	3,000
Total Loads (Including Wind or Seismic)	12,000	3	4,000

We recommend that all foundations be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. As a guide minimum only, continuous foundations should contain *at least* two No. 4 steel reinforcing bars placed one each, near the top and bottom of the foundations. *The project designer should determine the actual foundation reinforcement based on their specific structural design requirements, including the use of slab ties to provide structural continuity and integrity of the slab and foundation system.*

Resistance to lateral displacement of shallow foundations may be computed using an allowable friction factor of 0.35 multiplied by the effective vertical load on each foundation. Additional lateral resistance may be achieved using an allowable passive earth pressure against the vertical projection of the foundation equal to an equivalent fluid pressure of 350 psf per foot of depth. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

If foundations are constructed near the top of sloping ground, the foundations should be at least 10 feet away from the top of the slope (horizontal distance) for the full passive pressure and bearing capacity to be applicable.

It is an essential requirement that foundation excavations be observed by a representative of MPE to verify competent and uniform bearing conditions and evaluate the need for any modifications to these recommendations as may be required by specific circumstances. The observations should take place prior to placement of reinforcing steel but following cleaning of the excavations. To account for any re-compaction of foundation bottoms or deepening of foundations that might be required, we suggest bid documents include a unit price for additional compaction or foundation excavation and concrete that may be required.

INTERIOR FLOOR SLAB SUPPORT

Interior concrete slab-on-grade floors can be suitably supported upon the 12 inches of imported, non-expansive soil subgrade prepared and constructed in accordance with the recommendations in this report and maintained in that condition (at or near optimum conditions). From a Geotechnical standpoint, interior concrete slab-on-grade floors should be a minimum of four inches thick and, as a minimum, should be reinforced with chaired No. 3 reinforcing bars on 18-inch center-to-center spacing, located at mid-slab depth. *This slab thickness and reinforcement is suggested as a guide "minimum" only; final concrete slab thickness, compressive strength, reinforcement and joint spacing should be determined by the Architect or Structural Engineer based on anticipated slab loading, uses, and performance expectations.*

It is emphasized that thicker slabs with greater reinforcing will be needed in areas supporting higher loads or where increased performance is desired.

Temporary loads exerted during construction from vehicle traffic, cranes, forklifts, and storage of palletized construction materials should be considered in the design of the slab-on-grade floors. Proper and consistent location of the reinforcement at mid-slab is essential to its performance. The risk of uncontrolled shrinkage cracking is increased if the reinforcement is not properly located within the slab.

Floor slabs may be underlain by a layer of free-draining crushed rock, serving as a deterrent to migration of capillary moisture. The crushed rock layer should be at least four inches thick and graded such that 100 percent passes a one-inch sieve and none passes a No. 4 sieve. Additional moisture protection may be provided by placing a plastic water vapor retarder (at least 10-mils thick) directly over the crushed rock. The plastic water vapor retarder should meet or exceed the minimum specifications as outlined in ASTM E1745. Consideration should be given to using a thicker, higher quality membrane for additional moisture protection, such as a 15-mil thick Stego vapor barrier or other similar product. The membrane should be installed so that there are no holes or uncovered areas. All seams should overlap and be sealed with manufacturer-approved tape, continuous at the laps to create vapor tight conditions. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be sealed or caulked per manufacturer's recommendations. An optional, thin layer of clean sand above the membrane is acceptable, as an aid to curing of the slab concrete.

If heavier floor loads are anticipated and/or increased support is desired, the crushed rock section (if used) beneath interior slab-on-grade floors could be replaced with a thicker section of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557.

Floor slab construction over the past 25 years or more has included placement of a thin layer of sand over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern for water trapped within the sand. As a consequence, we consider the use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

The recommendations presented above are intended to mitigate any significant soils-related cracking of the slab-on-grade floors. More important to the performance and appearance of a Portland cement concrete slab is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized and the spacing of control joints.

FLOOR SLAB MOISTURE PENETRATION RESISTANCE

It is considered likely that floor slab subgrade soils will become wet to near-saturated at some time during the life of the structure. This is a certainty when slab subgrades are constructed during the wet seasons or when constantly wet ground or poor drainage conditions exist adjacent to structure. For this reason, it should be assumed that all slabs intended for moisture-sensitive floor coverings require protection against moisture or moisture vapor penetration. Standard practice includes the gravel and vapor retarder membrane, as discussed above. However, the gravel and membrane offer only a limited, first-line of defense against soil-related moisture. Recommendations contained in this report concerning foundation and floor slab design are presented as *minimum* requirements, only from the geotechnical engineering standpoint.

It is emphasized that the neither use of sub-slab crushed rock and sheet plastic membrane will not "moisture proof" the slab, nor does it assure that slab moisture transmission levels will be low enough to prevent damage to floor coverings or other building components. If increased protection against moisture vapor penetration of slabs is desired, a concrete moisture protection specialist should be consulted. The design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

Retaining Walls

We should review the project plans as they are developed to determine the applicability of the following recommendations and provide modified recommendations, as needed.

Retaining walls capable of slight rotation should be designed to resist active earth pressures equivalent to a fluid weighing 40 psf per foot of depth for horizontal backfill. Retaining walls that are fixed at the top and not capable of rotation should be designed to resist at-rest earth pressures equivalent to a fluid weighing 60 psf per foot of depth for horizontal backfill. For retaining walls with backfill sloped at a gradient on steeper than two horizontal to one vertical (2: 1), add 20 psf per foot of depth to the values for horizontal backfill.

Foundations for retaining structures should be designed in accordance with the recommendations contained in the FOUNDATION DESIGN section of this report. Passive resistance for retaining wall foundations constructed on sloping ground should be computed below a depth at which at least five feet of engineered fill or native soil is present in front of the foundation, as measured horizontally from the exterior edge of the foundation to the face of the slope. *It is essential that these foundation excavations be observed by a representative of MPE prior to placement of reinforcement to verify uniformity and acceptable bearing conditions, as well as to determine the need for additional deepening based on exposed conditions.*

The recommended active and at-rest pressure values assume fully drained wall backfill conditions. Drainage behind walls may consist of a drainage blanket (Class 2 permeable material, Caltrans Specification Section 68-2.02F(3)) at least one foot wide extending from the base of wall to within one foot of the top of the wall. Weep holes or perforated rigid pipe should be provided at the base of the wall to drain accumulated water. Drain pipes, if used, should be placed in the drainage blanket with perforations down and should slope to discharge at no less than a two percent fall. Washed ½-inch to ¾-inch crushed rock may be used in lieu of the Class 2 permeable material, provided the rock and drainpipe are completely enveloped in an approved nonwoven geotextile filter fabric.

Wall backfill should consist of granular soils compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. Rocky backfill material should be approved by our representative prior to use and should not contain rock greater than six inches in diameter.

Additional Retaining Wall Design Parameters

In addition to the design parameters in the above paragraphs, the following soil design parameters may be used in the design of reinforced earth (Keystone-type walls), or alternate design retaining walls, such as rockery walls. We should review the project plans, when complete, to verify the applicability of the following parameters.

Cohesion (c): 0 psf
Internal Angle of Friction (Φ): 35°
Compacted Moist Unit Weight of Soil (γ_m): 135 pcf

These parameters assume that the retained soils at the wall locations will consist of a compacted engineered fill mixture of on-site soils. We assume the wall backfill, including the reinforced zone, consisting of imported, non-expansive soils properly compacted. Modified recommendations and additional laboratory testing will be needed if materials vary from those assumed.

EXTERIOR FLATWORK

Areas to receive exterior concrete flatwork should be ripped and cross-ripped to a minimum depth of 12 inches, moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the ASTM D1557 maximum dry density. The upper 12 inches of exterior flatwork subgrades should consist of approved on-site or imported granular (non-expansive) soils or aggregate base. Uniform moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks. Practices recommended by the Portland Cement Association for proper placement and curing of concrete should be followed during exterior concrete flatwork construction. Some seasonal movement of flatwork should be anticipated. *Areas adjacent to slabs-on-grade should not be allowed to lay fallow to reduce problems associated with seasonal moisture content variations.* For increased support and performance, the exterior slabs may be underlain by a minimum four inches of Class 2 aggregate compacted to 95 percent relative compaction.

The Architect or Structural Engineer should determine the final thickness, strength, reinforcement, and joint spacing of exterior slab-on-grade concrete; however, we offer the following suggested minimum guidelines. Exterior flatwork should be at least four inches thick and be constructed independent of perimeter building foundations and isolated column foundations by the placement of a layer of felt material between the flatwork and the foundation. Reinforcement should consist of at least steel reinforcing bars, placed mid-depth of the slab. Slabs supporting vehicle loads should be designed as pavements with thicker slabs underlain by aggregate base. Thicker slabs constructed with thickened edges to at least twice the slab thickness should be constructed where light wheeled traffic or intermittent light loading is expected over the slabs.

Site Drainage

Control of surface water on this site is essential to proper performance of the planned improvements. Final site grading should be accomplished to provide positive drainage of surface water away from building, pavements, and structures and prevent ponding of water adjacent to foundations, slabs or pavements. Proper control of surface water drainage is essential to the performance of foundations, slabs-on-grade, and pavements. The ground adjacent to the planned building and structures should be sloped away from the structures at a gradient no less than two percent for a distance of at least 10 feet.

We recommend using full-roof gutters, with downspouts from roof drains connected to rigid non-perforated piping directed to an appropriate drainage point away from the structures, or discharging onto paved surfaces leading away from the structures and foundations. Concentrated storm water discharge collected from roof downspouts or surface drains should not be allowed to drain on unprotected slopes adjacent to structure. The ground should be graded to drain positively away from all flatwork and building structure. Ponding of surface water should be avoided near pavements, foundations, and flatwork. Landscape berms, if planned, should be constructed in such a manner as to promote drainage away from the buildings.

Seepage could be encountered during site excavation, or in permanent excavation slopes at the site, due to the low permeability of the on-site rock. Such seepage could require either temporary or permanent drainage features such as surface or subsurface drains. The need for such drains and their proper design and location should be determined during construction when subsurface conditions are fully exposed.

All excavations and fill slopes (if any) should be protected from concentrated storm water run-off to minimize potential erosion. Control of water over the slopes may be accomplished by constructing V-ditches near the top of slopes, or by grading the area behind the top of slope to drain away from the slope. Ponding of surface water or allowing sheet flow of water over any open excavation must be avoided.

Pavement Design

Grading plans were not available at the time we prepared this report; therefore, the materials to be exposed at final pavement subgrades is not known. In preparing this report and due to the near-surface soils primarily consisting of fair quality sandy silts, it is our opinion that an R-value of 15 should be used for pavement design (Figure A5). Depending on final grading plans, it is possible that grading may expose rock materials with potentially higher subgrade quality in which case revised pavement sections may be possible but would require additional testing to verify subgrade qualities and revised grading. The pavement sections have been calculated for a range of traffic indices using the design procedures contained in Chapters 600 to 670 of the 6th Edition of the *California Highway Design Manual.* The project Civil Engineer should determine the appropriate traffic index based on anticipated traffic conditions. Additional pavement sections for other traffic indices can be provided upon request.

Traffic Index (TI)	Pavement Subgrade R-value = 15	
	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2½*	8
5.0	2½	9
	3*	8

Traffic Index (TI)	Pavement Subgrade R-value = 15	
	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
6.0	2½	13
	3½*	11
6.5	3	14
	4*	12
7.0	3	15
	4*	13

* = Asphalt concrete thickness includes the Caltrans Safety Factor.

We emphasize that the performance of a pavement is critically dependent upon uniform compaction of the subgrade soils, as well as all engineered fill and utility trench backfill within the limits of the pavements. Materials used for pavement construction should conform to the appropriate sections of the most recent editions of the Sacramento County Standards and the Caltrans *Standard Specifications.*

It has been our experience that pavement failures may occur where a non-uniform or disturbed subgrade soil condition is created. Subgrade disturbances can result if pavement subgrade preparation is performed prior to underground utility construction and/or if a significant time period passes between subgrade preparation and placement of aggregate base. Therefore, we recommend that pavement subgrade preparation, i.e. scarification, moisture conditioning and compaction, be performed just prior to aggregate base placement.

The upper six inches of final pavement subgrades should be uniformly moisture conditioned to at least the optimum moisture content and compacted to at least 95 percent relative compaction if the subgrades consist of soil, or by *at least* five complete coverages by a Caterpillar 825 (or equivalent) if the subgrades are rocky. Subgrades must be properly compacted and stable prior to placing AB. Pavement subgrades should be proof-rolled with a loaded water truck and must be stable under construction traffic prior to placement of aggregate base. All aggregate base (AB) should be compacted to at least 95 percent of the maximum dry density. The AB should be proof rolled with a loaded water truck. Any areas of observed instability should be stabilized and recompacted as necessary to achieve the compaction requirements above. Earthwork construction within the limits of the pavements should be performed in accordance with the recommendation contained within this report. Materials quality and construction of the structural section should conform to the applicable provisions of the Caltrans Standard Specifications, latest editions.

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using a Portland cement concrete (PCC) section in areas subjected to concentrated heavy wheel loading, such as entry driveways, truck maneuvering areas, and in front of trash enclosures. At the time this report was prepared, the need for, and locations of, PCC pavements had not yet been determined. Therefore, when more information is available regarding uses, loading and potential subgrade conditions, we should review the information and provide specific thicknesses as applicable. For preliminary purposes, it may be assumed that Portland cement concrete slabs in areas of entry driveways and in front of trash enclosures should be at least 6 inches thick and be underlain by at least 6 inches of 95 percent compacted Class 2 aggregate base. Thicker slabs will be needed in areas of frequent bus traffic, in heavy duty areas, or areas subjected to high traffic frequencies by heavy trucks or equipment. In these areas, Portland cement concrete slabs with a minimum thickness of 7 inches and underlain by at least 6 inches of 95 percent compacted Class 2 aggregate base may be needed. These sections are preliminary and subject to revision based on review of additional information regarding loadings and traffic frequencies.

We suggest the concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards. Reinforcing for crack control, if desired, should consist of No. 4 reinforcing bars placed on maximum 24-inch centers each way throughout the slab. Reinforcement must be located at mid-slab depth to be effective. Construction of Portland cement concrete pavements should be performed in accordance with applicable American Concrete Institute (ACI) or PCA standards. Portland cement concrete utilized in pavements should attain a compressive strength of at least 3500 psi at 28 days.

Pavement Drainage

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Consideration should be given to using full-depth curbs between landscaped areas and pavements to serve as a cut off for water that could migrate into the pavement base materials or subgrade soils. Geotextile water barriers also could be used to inhibit migration of water into pavement base materials, if extruded curbs are used. Proprietary geotextile moisture barriers and curb details should be reviewed and approved by our office prior to construction. Weep holes are recommended in parking lot drop inlets to allow accumulating water moving through the aggregate base to drain from beneath the pavements.

Earthwork construction within the limits of the pavements should be performed in accordance with the recommendation contained within this report.

EARTHWORK TESTING AND OBSERVATION

Site preparation should be accomplished in accordance with the recommendations of this report and the appended *Guide Earthwork Specifications*. Representatives of Mid Pacific Engineering, Inc. must be present during site preparation and all grading operations to observe and test the fills to verify compliance with our recommendations and the job specifications. In the event that MPE is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide this service should indicate in writing that they agree with the recommendations of this report, and prepare supplemental recommendations as necessary.

A final report by the "Geotechnical Engineer" should be prepared upon completion of the project indicating compliance with or deviations from this report and the project plans and specifications. Please be aware that the title Geotechnical Engineer is restricted in the State of California to a Civil Engineer authorized by the State of California to use the title "Geotechnical Engineer."

FUTURE SERVICES

We recommend that our firm be given the opportunity to review the final plans and specifications to verify that the intent of our recommendations has been implemented in those documents. Testing and approval of proposed import sources is an essential requirement to qualify the proposed soils for use as engineered fill for this project. This sampling and testing should be completed well in advance of the proposed start of construction.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed construction, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used our best engineering judgment based upon the information provided and the data generated from our investigation. This report has been prepared in accordance with generally accepted standards of practice existing in northern California at the time of the report. No warranty, either express or implied, is provided.

If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at the test pit or boring locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

Mid Pacific Engineering, Inc., should be retained to review the final plans and specifications to verify that the intent of our recommendations has been implemented in those documents.

We emphasize that this report is applicable only to the proposed construction and the investigated site and should not be utilized for construction on any other site.

The conclusions and recommendations of this report are considered valid for a period of two years. If design is not completed and construction has not started within two years of the date of this report, the report must be reviewed and updated, as necessary.

Mid Pacific Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
MARTIN S. OSIER
No. 89860
Exp. 6/30/21
CIVIL
STATE OF CALIFORNIA

Martin S. Osier, PE
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
DANIEL C. SMITH
No. 2530
Exp. 6/30/21
GEOTECHNICAL
STATE OF CALIFORNIA

Daniel C. Smith, GE
Principal Engineer

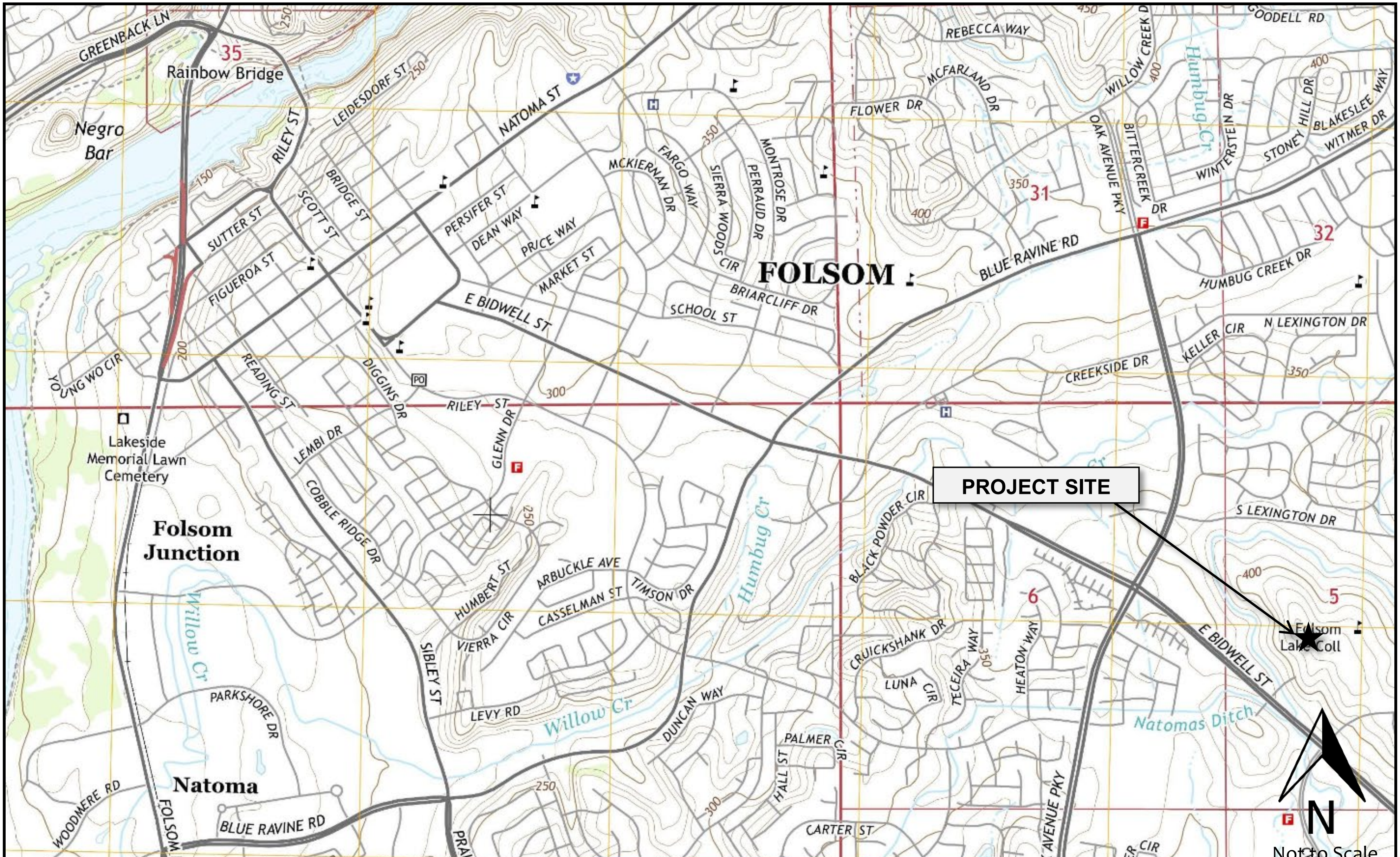
REGISTERED GEOLOGIST
VASILIY V. PARFENOV
No. 2355
CERTIFIED ENGINEERING GEOLOGIST
Exp. 6/30/21
STATE OF CALIFORNIA

Vasiliy V. Parfenov, CEG
Senior Geologist

PROFESSIONAL GEOLOGIST
DANIEL E. KRAMER
No. 2588
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

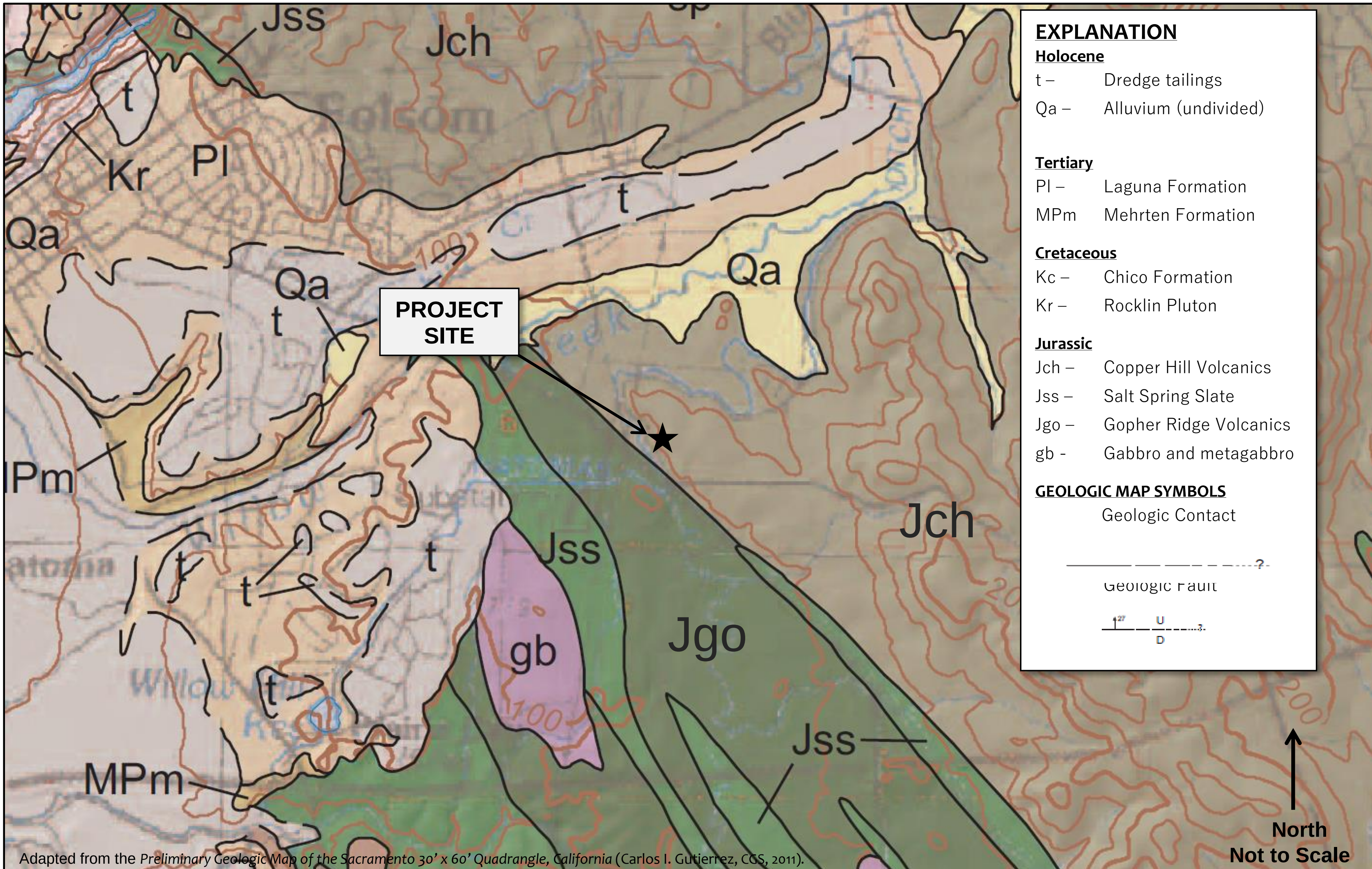
Daniel E. Kramer, CEG
Engineering Geologist

FIGURES



NOTES: Adapted from USGS National Topographic Map, *Folsom Quadrangle, California – Sacramento County,* 7.5-Minute Series, 2015.

MID PACIFIC ENGINEERING, INC.	**VICINITY MAP** **FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING** 10 College Parkway Folsom, California	**FIGURE 1** Date: 01/20 MPE No. 04843-01



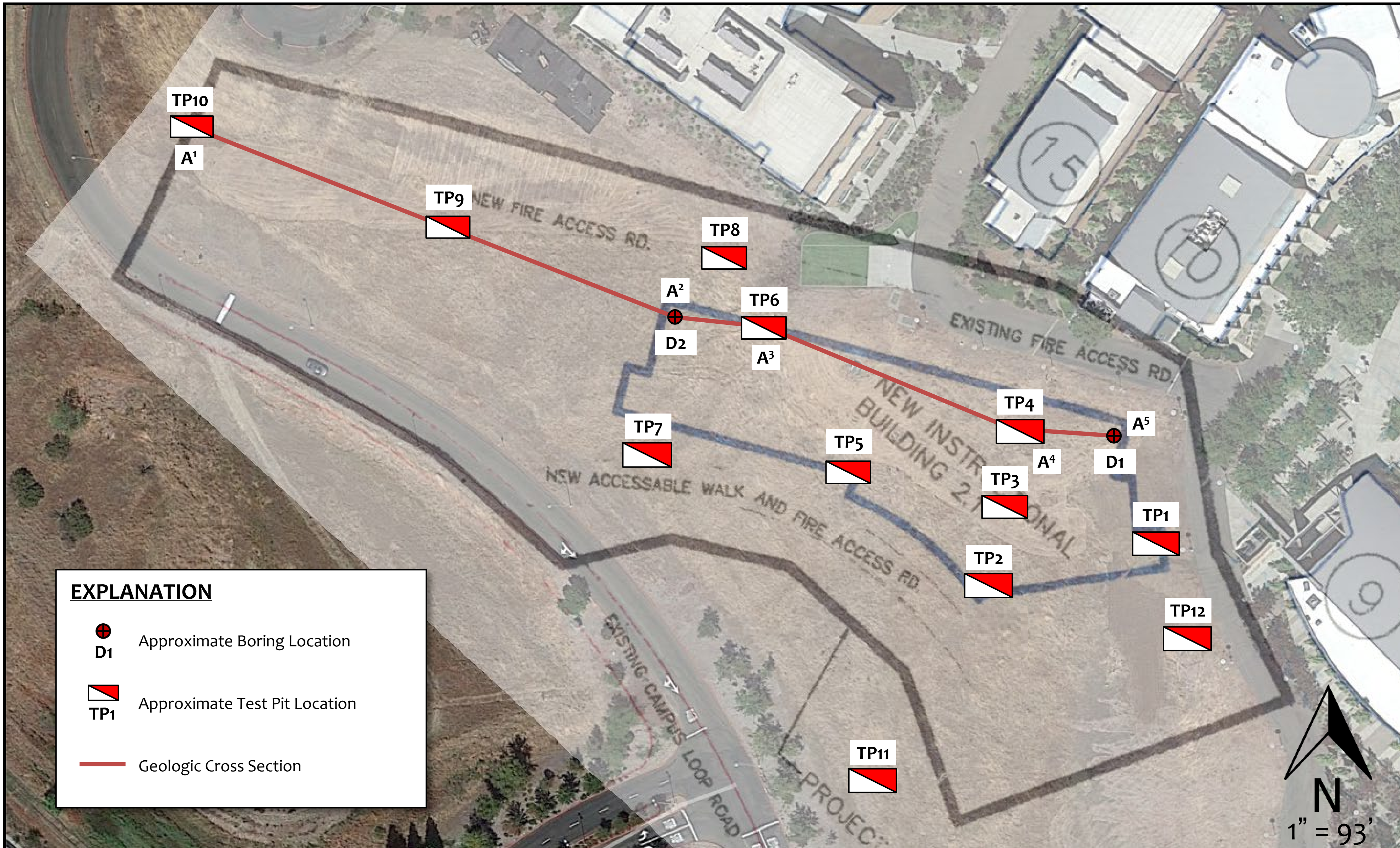
Adapted from the *Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle, California* (Carlos I. Gutierrez, CGS, 2011).

MPE
MID PACIFIC ENGINEERING, INC.

REGIONAL GEOLOGIC MAP

FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING
10 College Parkway
Folsom, California

FIGURE 2
Date: 01/20
MPE No. 04843-01



NOTES: Adapted from *FPP Pre-Schematic Plans, Sheet A1, dated April 2016*

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BORING AND TEST PIT LOCATION MAP

FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING

10 College Parkway
Folsom, California

FIGURE 3

Date: 01/20

MPE No. 04843-01

Project: Folsom Lake College Instructional Building	LOG OF SOIL BORING D1
Project Location: 10 College Parkway, Folsom, CA	
MPE Number: 04843-01	Sheet 1 of 2

Date(s) Drilled: 12/27/2019	Logged By: VVP	Checked By:
Drilling Method: Hollow Stem Auger/HXB Wireline	Drilling Contractor: V&W Drilling	Total Depth of Drill Hole: 41½ Feet
Drill Rig Type: CME 55 HT	Diameter(s) of Hole, inches: 7⅝ Inches	Approx. Surface Elevation, MSL: + 427 Feet
Groundwater Depth [Elevation]: Not Encountered	Sampling Method(s): 140 Lb Hammer/30" Drop	Drill Hole Backfill: Soil Cuttings
Remarks		

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: BLOWS PER FOOT	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Reddish brown, moist, fine to coarse sandy, fine to coarse gravel (GM-FILL) with cobbles						
			Reddish brown, moist, stiff, clayey silt (CL-ML)		D1-1	14			UCC 0.56 tsf
			Brown, completely weathered, soft, andesite (Rx)						
	5				D1-2	>100			ϕ=17° C=10 psf
	10		no recovery		D1-3	>50			
	15				D1-3	50/3"			
			Auger Refusal at 16¼ feet						
			Gray, highly weathered, hard, intensely fractured, basalt (Rx) Core run: 16¼' - 21¼' Recovery: 30% RQD: 0%						
	20		slightly weathered Core run: 21¼' - 26¼' Recovery: 34% RQD: 7%						
	25		Core run: 26¼' - 31¼' Recovery: 31% RQD: 12%						

Project: Folsom Lake College Instructional Building	LOG OF SOIL BORING D1
Project Location: 4700 College Oak Parkway, Folsom, CA	
MPE Number: 04843-01	Sheet 2 of 2

Date(s) Drilled	12/27/2019	Logged By	VVP	Checked By	
Drilling Method	Hollow Stem Auger/HXB Wireline	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	41½ Feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, MSL	+ 427 Feet
Groundwater Depth [Elevation]	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Soil Cuttings
Remarks					

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	30		Core run: 31¼' - 36¼' Recovery: 29% RQD: 10%						Compressive Strength 172 tsf
			loss of drilling fluid at 34 feet						
	35		Core run: 36¼' - 41¼' Recovery: 87% RQD: 0%						
	40								
			Bottom of boring at 41¼ feet						
	45								
	50								

Project: Folsom Lake College Instructional Building	LOG OF SOIL BORING D2
Project Location: 10 College Parkway, Folsom, CA	
MPE Number: 04843-01	Sheet 1 of 1

Date(s) Drilled	12/30/2019	Logged By	VVP	Checked By	
Drilling Method	Hollow Stem Auger/HXB Wireline	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	20½ Feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, MSL	+ 410 Feet
Groundwater Depth [Elevation]	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Soil Cuttings
Remarks					

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Reddish brown, moist, silty fine to coarse sandy, fine to coarse gravel (GM-FILL)						
			Reddish brown, moist, stiff, clayey silt (CL-ML)		D2-1	20			UCC 0.23 tsf
			Brown, completely weathered, soft, andesite (Rx)						
	5		no recovery		D2-2	50			
			Auger refusal at 9½ feet						
	10		no recovery		D2-3	50/1"			
			Brown, completely weathered, moderatley hard, very intensely fractured andesite (Rx) Core run: 9½' - 12½' Recovery: 8½% RQD: 0%						
			Gray, highly weathered, hard, very intensely to intensely fractured, andesite (Rx) Core run: 12½' - 16' Recovery: 45% RQD: 0%						
	15		Core run: 16' - 17½' Recovery: 33% RQD: 0%						
			Core run: 17½' - 20½' Recovery: 17% RQD: 0% very intensely fractured						
	20								
			Bottom of boring at 20½ feet						
	25								

LOGS OF TEST PITS

Excavated on: December 18, 2019
Takeuchi TB260 with a 24-inch bucket
Logged by: P. Porata

Test Pit 1 **(38.661252, -121.128608)**
MSL = +425 feet

0' – 2½'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, with cobbles up to 1 foot in size (GM-FILL)
2½' – 4'	Reddish brown, moist, silty, clayey sand (SC)
4' – 5½'	Brown, completely weathered, very soft, andesite (Rx)
5½' – 7'	Brown, highly weathered, aphanitic, hard, andesite (Rx)

Refusal on andesite at 7 feet.

Test Pit 2 **(38.661194, -121.128978)**
MSL = +419 feet

0' – 1'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, with cobbles up to 1 foot in size (GM-FILL)
1' – 3'	Reddish brown, moist, clayey silt (CL-ML)
3' – 5½'	Brown, completely weathered, soft, andesite (Rx) with highly weathered, hard, basalt pillow lavas
5½' – 8½'	Brown, highly weathered, aphanitic, hard, andesite (Rx)

Refusal on andesite at 8½ feet.

Test Pit 3 **(38.661316, -121.128963)**
MSL = +420 feet

0' – 1'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, with cobbles up to 1 foot in size (GM-FILL)
1' – 2½'	Reddish brown, moist, clayey silt (CL-ML)
2½' – 5'	Brown, completely weathered, soft, andesite (Rx) weathers to clayey sand
5' – 5½'	Brown, highly weathered, aphanitic, hard, andesite (Rx)

Refusal on andesite at 5½ feet.

LOGS OF TEST PITS

Excavated on: December 18, 2019
Takeuchi TB260 with a 24-inch bucket
Logged by: P. Porata

Test Pit 4 **(38.661463, -121.128928)**
MSL = +422 feet

0' – 1½'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, with cobbles up to 1 foot in size, with construction debris (GM-FILL)
1½' – 4'	Reddish brown, moist, clayey silt (CL - ML)
4' – 6'	Brown, completely weathered, soft, andesite (Rx) with highly weathered, hard, basalt pillow lavas
6' – 9½'	Brown, highly weathered, aphanitic, soft, andesite (Rx)

Refusal on highly weathered, hard, basalt pillow lavas at 9½ feet.

Test Pit 5 **(38.661383, -121.129340)**
MSL = +413 feet

0' – 2'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, with cobbles up to 1 foot in size, with construction debris (GM-FILL)
2' – 3½'	Reddish brown, moist, clayey silt (CL-ML)
3½' – 5'	Brown, completely weathered, soft, andesite (Rx) with highly weathered, hard, basalt pillow lavas
5' – 6'	Brown, highly weathered, aphanitic, soft, andesite (Rx) with highly weathered, hard, basalt pillow lavas

Refusal on highly weathered, hard, basalt pillow lavas at 6 feet.

Test Pit 6 **(38.661658, -121.129546)**
MSL = +413 feet

0' – ½'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel (GM-FILL)
½' – 2½'	Reddish brown, moist, clayey silt (ML)
2½' – 4'	Brown, completely weathered, soft, andesite (Rx)
4' – 5'	Brown, highly weathered, aphanitic, moderately soft, andesite (Rx) with highly weathered, hard, basalt pillow lavas

Refusal on highly weathered, hard, basalt pillow lavas at 5 feet.

LOGS OF TEST PITS

Excavated on: December 18, 2019
Takeuchi TB260 with a 24-inch bucket
Logged by: P. Porata

Test Pit 7 **(38.661414, -121.129831)**
MSL = +401 feet

0' – 2'	Reddish brown, moist, clayey silt, with fine to coarse gravel, some cobbles (CL-ML)
2' – 4'	Brown, completely weathered, soft, andesite (Rx)
4' – 5'	Brown, highly weathered, aphanitic, hard, andesite (Rx)

Refusal on andesite at 5 feet.

Test Pit 8 **(38.661794, -121.129640)**
MSL = +415 feet

0' – ½"	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel (GM-FILL)
½" – 2'	Reddish brown, moist, clayey silt (CL-ML)
2' – 4'	Brown, highly weathered, aphanitic, moderately hard, very intensely to intensely fractured, andesite (Rx)

Refusal on andesite at 4 feet.

Test Pit 9 **(38.661860, -121.130328)**
MSL = +401 feet

0' – 1½'	Reddish brown, moist, clayey silt, with fine to coarse gravel, some cobbles (CL-ML)
1½' – 4'	Brown, highly weathered, aphanitic, hard, andesite (Rx)

Refusal on andesite at 4 feet.

Test Pit 10 **(38.662067, -121.130968)**
MSL = +393 feet

0' – 2'	North sidewall - Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, some cobbles (GM-FILL) South sidewall - Reddish brown, moist, clayey silt (CL-ML)
2' – 3'	Brown, highly weathered, aphanitic, hard, andesite (Rx)

Bottom of the test pit at 3 feet.

LOGS OF TEST PITS

Excavated on: December 18, 2019
Takeuchi TB260 with a 24-inch bucket
Logged by: P. Porata

Test Pit 11 **(38.660800, -121.129287)**
MSL = +412 feet

0' – 1½'	Reddish brown, moist, clayey silt (CL-ML)
1½' – 3½'	Brown, highly weathered, aphanitic, hard, intensely fractured (fractures deep 45 degrees NE, at 85 degrees) andesite (Rx)

Bottom of the test pit at 3½ feet.

Test Pit 12 **(38.661065, -121.128528)**
MSL = +424 feet

0' – 5'	Reddish brown, moist, silty, fine to coarse sandy, fine to coarse gravel, some cobbles in 1' in size (GM-FILL)
5' – 6½'	Reddish brown, moist, clayey silt, with fine to coarse gravel (CL-ML); water seepage at 5 feet
6½' – 7½'	Light gray, completely weathered, soft, andesite (Rx) weathers to clayey sand
7½' – 9½'	Brown, highly weathered, aphanitic, hard, andesite (Rx) with highly weathered, hard, basalt pillow lavas

Refusal on highly weathered, hard, basalt pillow lavas at 9½ feet.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		SYMBOL	CODE	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well graded gravels or gravel - sand mixtures, little or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, little or no fines
		GM-FILL		Artificially placed fill material of silty gravels, gravel - sand - silt mixtures
		GC		Clayey gravels, gravel - sand - silt mixtures
	SANDS (50% or more of coarse fraction < no. 4 sieve size)	SW		Well graded sands or gravelly sands, little or no fines
		SP		Poorly graded sands or gravelly sands, little or no fines
		SM		Silty sands, sand - silt mixtures
		SC		Clayey sands, sand clay mixtures
FINE GRAINED SOILS (More than 50% of soil < no. 200 sieve size)	SILTS & CLAYS LL< 50	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL		Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL ≥ 50	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		Inorganic clays of high plasticity, fat clays
		OH		Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler

= Continuous HXB Wireline Core

=SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests
- PI = Plasticity Index
- EI =Expansive Index
- UCC = Unconfined Compression Test
- TR = Triaxial Compression Test
- GR = Gradation Analysis (Sieve)
- K = Permeability Test

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL coarse (c) fine (f)	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND coarse (c) Medium (m) fine (f)	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

MPE
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UNIFIED SOIL CLASSIFICATION SYSTEM
FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING
10 College Parkway
Folsom, California

FIGURE 10
Date: 01/20
MPE No. 04843-01

FRACTURING	
LOG TERM	DEFINITION
Very Wide	> 6 feet
Wide	2 to 6 feet
Moderately	8 to 24 inches
Closely	2 1/2 to 8 inches
Very Closely	3/4 to 2 1/2 inches

ROCK QUALITY DESIGNATION (RQD)	
RQD (%)	ROCK QUALITY
90 to 100	Excellent
75 to 90	Good
50 to 75	Fair
25 to 50	Poor
0 to 25	Very Poor

WEATHERING	
LOG TERM	DESCRIPTION/DEFINITION
Fresh	No visible sign of decomposition or discoloration. Rings under hammer impact
Slightly Weathered	Slight discoloration inwards from open fractures; otherwise similar to fresh
Moderately Weathered	Discoloration throughout. Strength less than fresh rock, specimens cannot be broken by hand or scraped with knife
Highly Weathered	Specimens can be broken by hand with effort and shaved with knife. Textures becoming indistinct but fabric preserved
Completely Weathered	Mineral decomposed to soil but fabric and structure preserved. Specimens easily crumbled or penetrated.

COMPETENCY			
CLASS	LOG TERM	DESCRIPTION/DEFINITION	APPROXIMATE RANGE OF UNCONFINED COMPRESSIVE STRENGTHS (tsf)
I	Extremely Strong	Many blows with geologic hammer required to break intact specimens	>2000
II	Very Strong	Hand held specimens break with pick end of hammer under more than one blow	1000 to 2000
III	Strong	Hand held specimens can be broken with singer, moderate blow with pick end of hammer	500 to 1000
IV	Moderately Strong	Specimens can be scraped with knife; light blow with pick end of hammer causes indentations	250 to 500
V	Weak	Specimens crumble under moderate blow with pick end of hammer	10 to 250
VI	Friable	Specimens crumble in hand	N/A

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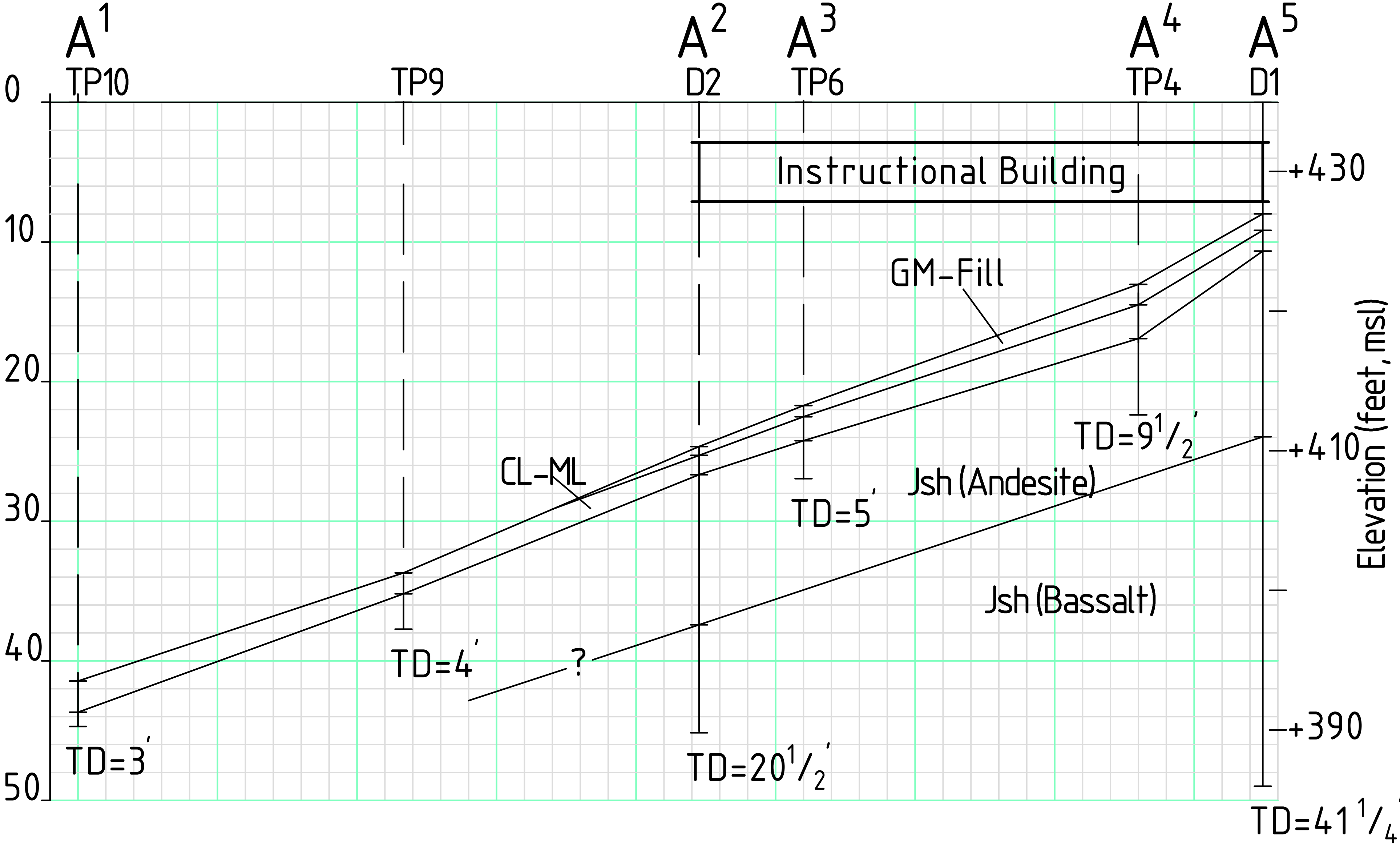
ROCK LEGEND

FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING
10 College Parkway
Folsom, California

FIGURE 11
Date: 01/20
MPE No. 04843-01



Scale: hor 1":80'
ver 1":10'

MPE
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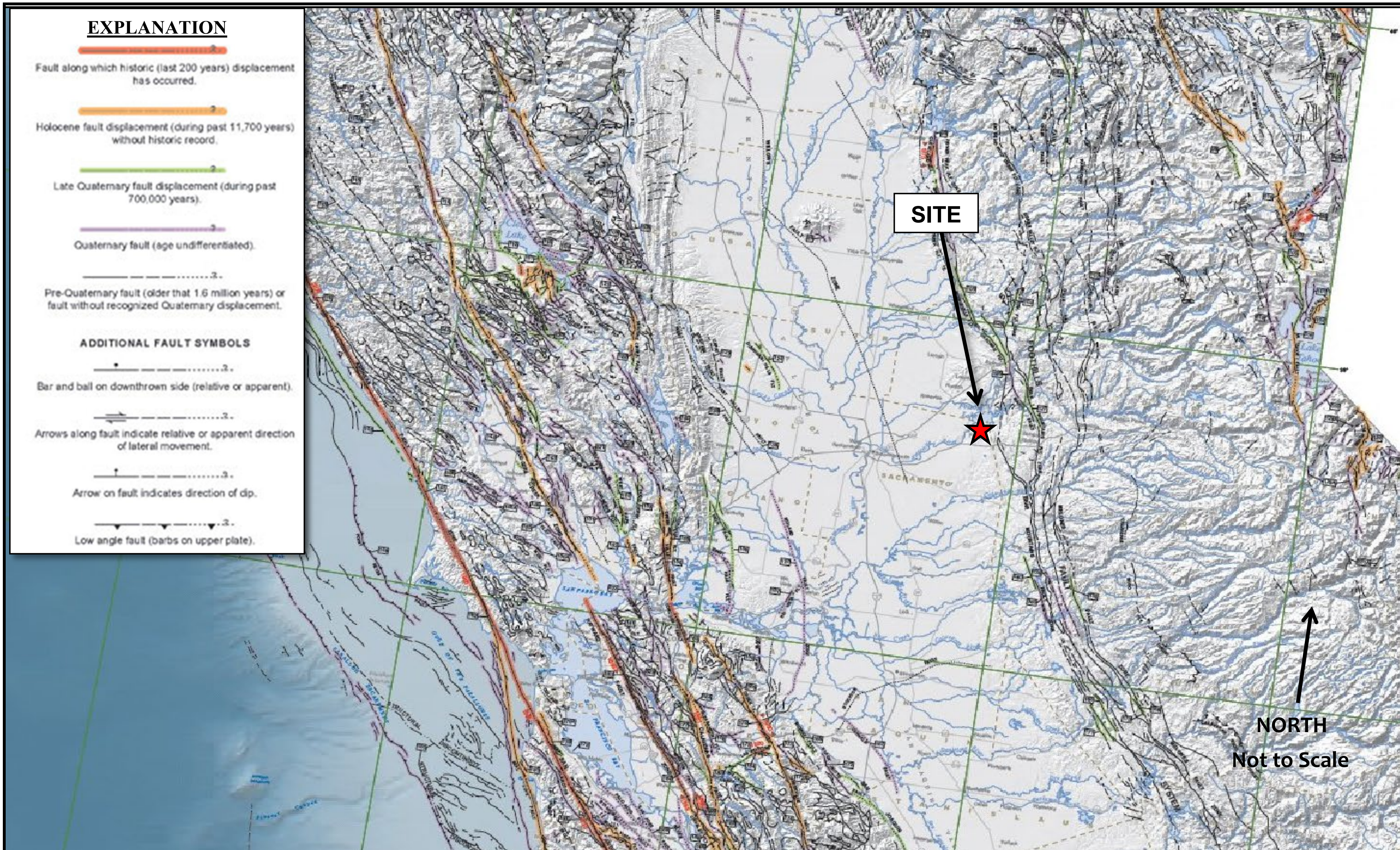
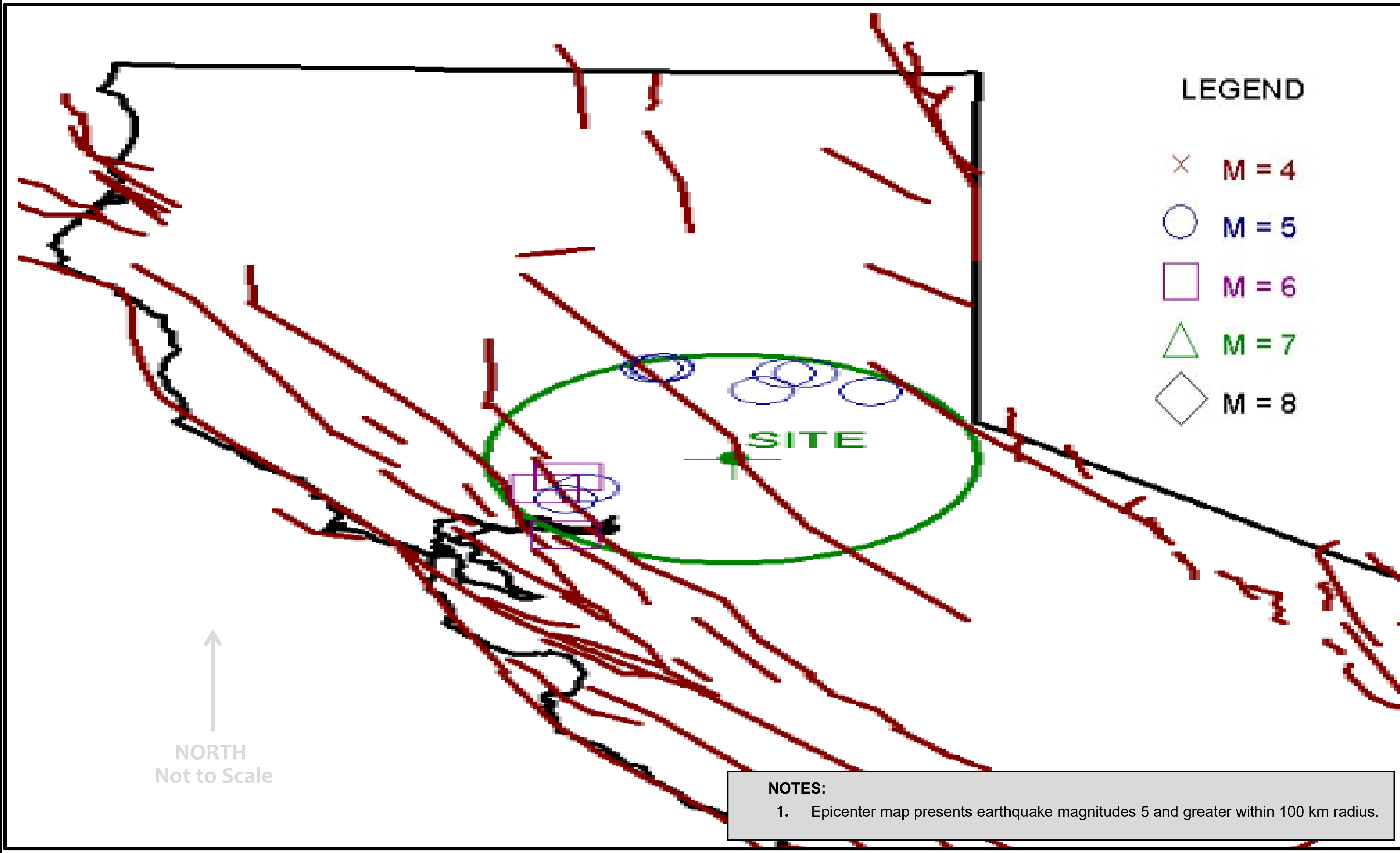
GEOLOGIC CROSS SECTION A^1 – A^5
FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING
10 College Parkway
Folsom, California

FIGURE 12
Date:01/20
MPE No. 04843-01



Adapted from: *Fault Activity Map of California 2010.* California Geological Survey, Geologic Data Map No. 6. Compilation and Interpretation by C.W. Jennings and W.A. Bryant

MPE MID PACIFIC ENGINEERING, INC.	**REGIONAL FAULT MAP** **FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING** 10 College Parkway Folsom, California	**FIGURE 13** Date: 01/20 MPE No. 04843-01



NOTES:

1. Epicenter map presents earthquake magnitudes 5 and greater within 100 km radius.

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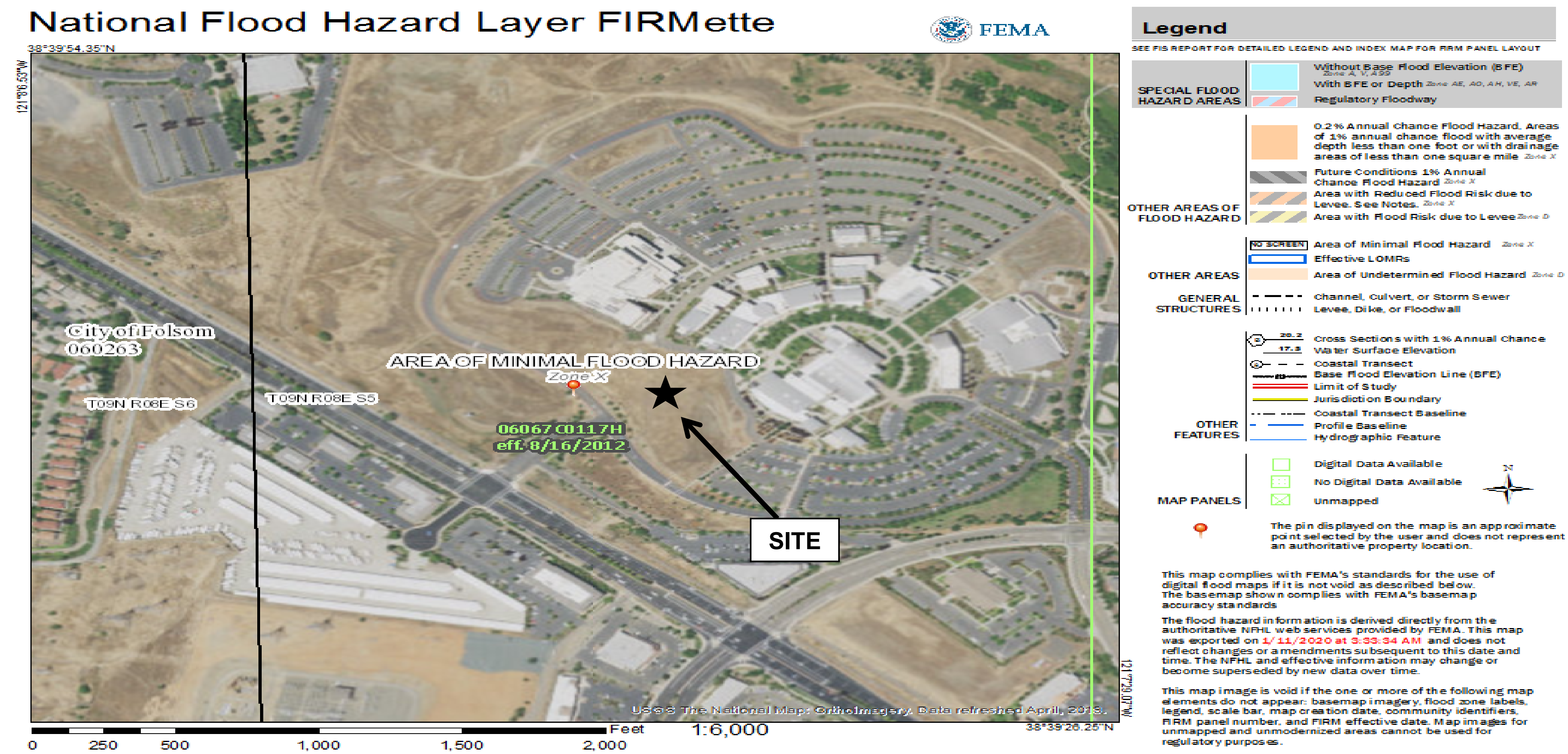
EARTHQUAKE EPICENTER MAP

FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING
10 College Parkway
Folsom, California

FIGURE 14
Date: 01/20
MPE No. 04843-01



FLOOD HAZARDS

ZONE X – Site shown as being protected from the 1-percent chance flood hazard by a levee system.
Overtopping or failure of any levee system is possible.
ZONE AE – Base Flood Elevations Determined

N
Not to Scale

Adapted from: Federal Emergency Management Agency (FEMA), Flood Insurance Rate Map (FIRM), Map Number 06077C0117H, dated January 11, 2020

MPE MID PACIFIC ENGINEERING, INC.	**FEMA FLOOD MAP** **FOLSOM LAKE COLLEGE INSTRUCTION BUILDING** 10 College Parkway Folsom, California	**FIGURE 15** Date: 01/20 MPE No. 04843-01

APPENDICES

APPENDIX A

APPENDIX A

A. GENERAL INFORMATION

The performance of a Geotechnical Engineering Investigation and Geologic Hazards Report for the Instructional Building project to be constructed at the existing Folsom Lake College campus located at 10 College Parkway in Folsom, California, was authorized by Josef Meyer with the Los Rios Facilities Management on December 3, 2019, whose mailing address is 3753 Bradview Drive, Sacramento, California 95827; telephone (916) 856-3457.

B. FIELD EXPLORATION

On December 18, 2019, twelve test pits were excavated at the approximate locations indicated on Figure 3, utilizing a Takeuchi TB260, track-mounted excavator equipped with 24-inch bucket. The test pits were excavated to the maximum depths of approximately 3 to 9½ feet below existing site grades. On December 27 and 30, 2019, two soil borings were drilled at the approximate locations indicated on Figure 3, utilizing a CME-55 High Torque, truck-mounted drill rig. Test borings were advanced utilizing 7⅝-inch diameter, hollow-stem augers till practical refusal at depths of 9½ and 16¼ feet below existing site grades. After auger refusal, the borings were advanced utilizing 3.65-inch diameter continuous core HXB Wireline system to the maximum depths of 20½ and 41¼ feet below existing site grades.

Within portions of the borings advances with augers, at various intervals, relatively undisturbed soil/rock samples were recovered with a 2½-inch O.D., 2-inch I.D. or with 3-inch O.D., 2½-inch I.D. Modified California samplers (ASTM D3550), or with a 2-inch O.D., 1⅜-inch I.D. SPT sampler (ASTM D1586) driven by a 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each 6-inch interval was recorded with the sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, being designated the penetration resistance or "blow count" for that particular drive.

The samples obtained with the modified California samplers were retained in 2-inch diameter by 6-inch long, thin-walled brass tubes contained within the sampler. The samples obtained with the SPT sampler were retained in sealed plastic bags. Immediately after recovery, the field engineer visually classified the soil/rock in the tubes or SPT- sampler. The ends of the tubes were sealed or soils from the SPT sampler were placed in the sealed plastic bags to preserve the natural moisture contents. Disturbed bulk samples of the surface materials also were obtained at various locations and depths. Immediately after recovery, the field engineer visually classified continuous rock cores, 2.406-inch diameter, and determined percentage recovery and Rock Quality Designation for each core run. Core samples were placed in core boxes. Soil/rock samples and rock cores were taken to our laboratory for additional classification (ASTM D2488) and selection of samples for testing.

The Logs of Soil Borings, Figures 4 and 5 and the Logs of Test Pits, Figures 6 through 9, contain descriptions of the soils/rock encountered in each test pit or boring. A Boring Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 10.

C. LABORATORY TESTING

Selected samples of the soils/rock were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216), unconfined compression strength (ASTM D2166), compressive strength (ASTM D7012), and triaxial shear strength (ASTM D2850). The results of these tests are included on the boring logs at the depth each sample was obtained.

Three bulk samples of the soil/rock were subjected to percent passing the 200 sieve (ASTM D1140) and Atterberg limits (AASTM D4318) tests. The results of these tests are presented on Figure A1.

Three bulk samples of the soils/rock were subjected to an Expansion Index testing (ASTM D4829). The results of these tests are presented on Figures A2 through A4.

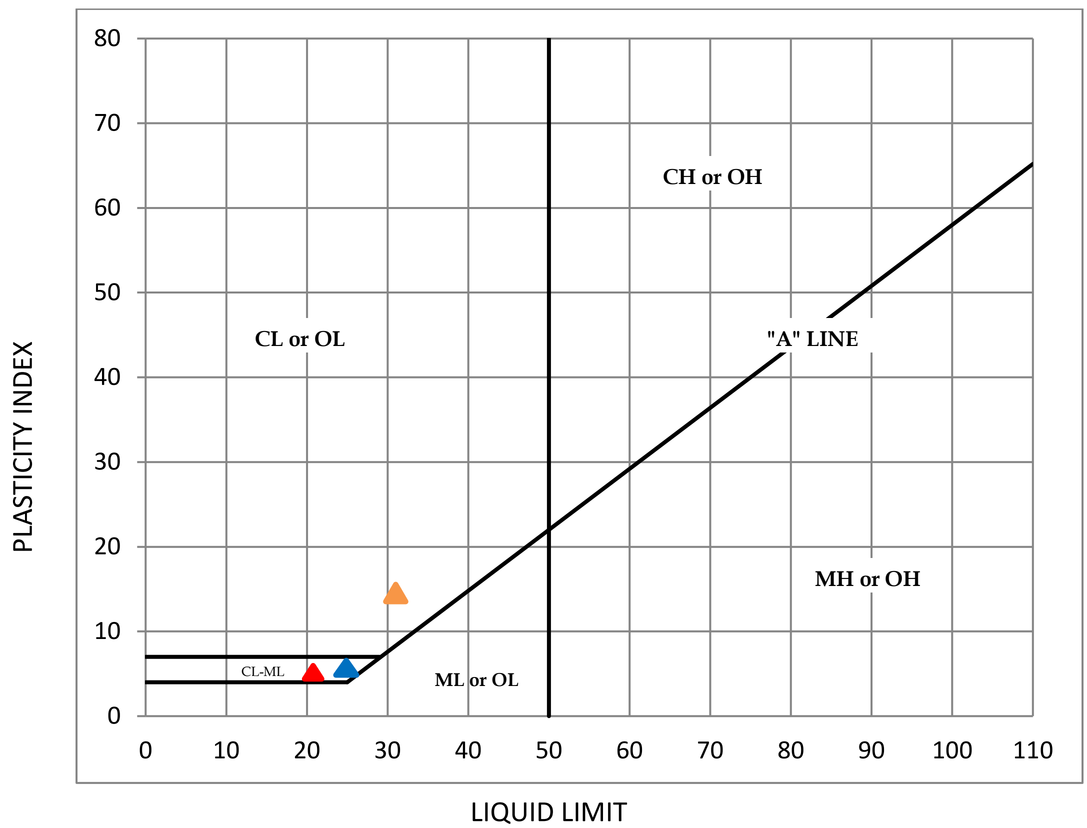
One bulk sample of the anticipated pavement subgrade soils was subjected to Resistance ("R-") value testing. The results of the test were used in the pavement design and presented on Figure A5.

Two samples of near-surface soils were submitted to Sunland Analytical in Rancho Cordova, California, for corrosivity testing in accordance with No. 643 (Modified Small Cell), CT 532, CT 422, and CT 417. The analytical results are presented in the text of the report.

ATTERBERG LIMITS
(ASTM D4318 and D1140)

Symbol	Sample & Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Percent Passing #200 Sieve
▲ (red)	TP1 (@3')	21	16	5	43.4
▲ (orange)	TP3 (@3')	31	17	14	33.0
▲ (blue)	TP12 (@7')	25	19	6	29.7

PLASTICITY CHART



MPE	**ATTERBERG LIMITS TEST RESULTS** **FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING** 10 College Parkway Folsom, California	**FIGURE A1** Date: 01/20 MPE No. 04843-01

EXPANSION INDEX TEST RESULTS
(ASTM D4829)
(UBC 18-2)

Material Description: Reddish brown, silty, clayey sand (SC)
Location: TP1 (@3')

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
TP1 (@3')	6.2	14.2	119	31

CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
Above 130	Very High

EXPANSION INDEX TEST RESULTS
(ASTM D4829)
(UBC 18-2)

Material Description: Brown, completely weathered rock (Rx) weathers to clayey sand
Location: TP3 (@3')

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
TP3 (@3')	8.4	19.0	115	46

CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
Above 130	Very High

EXPANSION INDEX TEST RESULTS
(ASTM D4829)
(UBC 18-2)

Material Description: Light gray, highly weathered metavolcanic rock (Rx) weathers to clayey sand
Location: TP12 (@7')

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
TP12 (7')	8.2	20.2	116	27

CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
Above 130	Very High

RESISTANCE VALUE TEST RESULTS
(ASTM D2844, CT 301)

Material Description: Brown, completely weathered rock (Rx) weathers to clayey sand

Location: TP3 (@3')

Specimen No.	Dry Unit Weight (pcf)	Moisture at Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
1	124.0	15.3	357	43	20
2	118.5	16.7	161	30	15
3	121.0	14.2	581	69	29

R-value at 300 psi exudation pressure = 19

APPENDIX B

APPENDIX B

GUIDE EARTHWORK SPECIFICATIONS

FOLSOM LAKE COLLEGE INSTRUCTIONAL BUILDING

10 College Parkway

Sacramento, California

MPE No. 04843-01

PART 1: GENERAL

1.1 SCOPE

A. General Description

This item shall include clearing of all surface and subsurface structures associated with previous development of the site, existing structures, septic systems, leach lines, concrete slabs, foundations, asphalt concrete, utilities to be relocated or abandoned including all associated backfill, trees, demolition debris, rubbish, rubble, rubbish and associated items; preparation of surfaces to be filled, filling, spreading, compaction, observation and testing of the fill; and all subsidiary work necessary to complete the grading of the building areas to conform with the lines, grades and slopes as shown on the accepted Drawings.

B. Related Work Specified Elsewhere

1. Trenching and backfilling for sanitary sewer system: Section ______.
2. Trenching and backfilling for storm drain system: Section ______.
3. Trenching and backfilling for underground water, natural gas, and electric supplies: Section ______.

C. Geotechnical Engineer

Where specific reference is made to "Geotechnical Engineer" this designation shall be understood to include either him or his representative.

1.2 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by at the site. Streets and adjacent property shall be fully protected throughout the operations.

B. In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.

C. Any construction review of the Contractor's performance conducted by the Geotechnical Engineer is not intended to include review of the adequacy of the Contractor's safety measures, in, on or near the construction site.

D. Adjacent streets and sidewalks shall be kept free of mud, dirt or similar nuisances resulting from earthwork operations.

E. Surface drainage provisions shall be made during the period of construction in a manner to avoid creating a nuisance to adjacent areas.

F. The site and adjacent influenced areas shall be watered as required to suppress dust nuisance.

1.3 GEOTECHNICAL REPORT

A. A Geologic Hazards and Geotechnical Engineering Report (MPE No. 04843-01; dated January 15, 2020) has been prepared for this site by Mid Pacific Engineering, Inc., Geotechnical Engineers. A copy is available for review at the office of Mid Pacific Engineering, Inc., 840 Embarcadero Drive, Suite 20, West Sacramento, California 95605.

B. The information contained in this report was obtained for design purposes only. The Contractor is responsible for any conclusions he/she may draw from this report; should the Contractor prefer not to assume such risk, he/she should employ their own experts to analyze available information and/or to make additional borings upon which to base their conclusions, all at no cost to the Owner.

1.4 EXISTING SITE CONDITIONS

The Contractor shall be acquainted with all site conditions. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor's operations subsequent to the discovery of such unshown utilities.

1.5 SEASONAL LIMITS

Fill material shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until field tests indicate that the moisture contents of the subgrade and fill materials are satisfactory.

PART 2: PRODUCTS

2.1 MATERIALS

A. All fill shall be of approved local materials from required excavations, supplemented by imported fill, if necessary. Approved local materials are defined as local soil and rock materials with a maximum particle size of approximately six inches (6") in the upper five feet (5') from the finished subgrade, and twelve inches (12") below five feet (5') from the finished subgrade; free from significant quantities of rubble, rubbish and vegetation; and, having been tested and approved by the Geotechnical Engineer prior to use.

B. Imported fill materials shall be approved by the Geotechnical Engineer; shall meet the above requirements; shall have plasticity indices not exceeding fifteen (15), when tested in accordance with ASTM D4318; shall have a maximum Expansion Index not exceeding twenty (20) when tested in accordance with ASTM D4829; and, shall be of three-inch (3") maximum particle size. Import fill shall be clean of contamination with appropriate documentation. All imported materials shall be approved by the Geotechnical Engineer prior to being transported to the site.

C. Asphalt concrete, aggregate base, aggregate subbase, and other paving products shall comply with the appropriate provisions of the *State of California (Caltrans) Standard Specifications* Standards, latest editions.

PART 3: EXECUTION

3.1 LAYOUT AND PREPARATION

Lay out all work, establish grades, locate existing underground utilities, set markers and stakes, set up and maintain barricades and protection of utilities-all prior to beginning actual earthwork operations.

3.2 CLEARING, GRUBBING AND PREPARING BUILDING PADS AND PAVEMENT AREAS

A. The site shall be cleared of existing structures designated for removal including but not limited to flatwork, asphalt concrete, utilities to be relocated or abandoned including all associated backfill, demolition debris, rubbish, rubble and other unsuitable materials. Subsurface utilities to be relocated or abandoned shall be removed from within and to at least five feet beyond the perimeter of the proposed structural areas; remaining piping beyond the structure that is not removed shall be plugged. Excavations and depressions resulting from the removal of such items, as well as any existing excavations or loose soil deposits, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soil and backfilled with suitable materials in accordance with these specifications.

B. The upper twelve inches (12") of soil subgrades within areas of removed flatwork, pavements, and trees shall be ripped and cross-ripped to expose any remaining remnants, roots, rubble and debris. All exposed rubble, roots, rubble and debris shall be removed from the subgrades. Hand picking of exposed roots, rubble and debris shall be performed by the Contractor to adequately clear the grades.

C. Following site clearing operations, undocumented fill soils in the structural areas shall be over-excavated to the depths and lateral extents as recommended in the Geotechnical Engineering Report.

D. If the option to support the proposed structure entirely on engineered fill is selected, the entire building pad shall be over-excavated to provide a minimum of two feet (2') of engineered fill below the foundations. Additional over-excavation shall be required to reduce the differential thickness of the fill below the foundations to within five feet (5') in 40 linear feet (40').

E. If the option to support the proposed structure entirely on weathered rock is selected, the entire building pad shall be over-excavated to provide a minimum of twelve inches (12") of granular, non-expansive engineered fill below the proposed slab-on-grade.

F. The surfaces upon which fill is to be placed, as well as at-grade areas or areas achieved by excavation, shall be plowed or scarified to a depth of at least eight inches (8") until the surface is free from ruts, hummocks or other uneven features which would tend to prevent uniform compaction by the selected equipment.

G. Subgrade preparation and compaction shall extend at least five feet (5') beyond the proposed structure lines, or as required by the Geotechnical Engineer based on the exposed soil and site conditions.

H. When the moisture content of the subgrade is below that required to achieve the specified density, and that minimum content recommended in the geotechnical report, water shall be added until the proper moisture content is achieved.

I. When the moisture content of the subgrade is too high to permit the specified compaction to be achieved, the subgrade shall be aerated by blading or other methods until the moisture content is satisfactory for compaction.

J. After the foundations for fill have been cleared, plowed or scarified, they shall be disced or bladed until uniform and free from large clods, brought to the proper moisture content and compacted to not less than ninety percent (90%) for all structural areas of the maximum dry density as determined by the ASTM D1557-91 Compaction Test. Soils compaction shall be performed using a heavy, self-propelled sheepsfoot compactor (Caterpillar 825 or equivalent) capable of providing compaction to the full depth of soils scarification/ripping.

K. Rocky soils that are too rocky to test using conventional methods shall be compacted using a performance specification based on the following criteria. For densities comparable to ninety percent (90%) relative compaction, subgf-grades shall be compacted by at least three (3) complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor, to the satisfaction of our on-site representative. For densities comparable to 95 percent relative compaction, rocky materials shall be compacted by at least five (5) complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor. The number of passes shall be considered the minimum and passes shall be added as required by the Geotechnical Engineer to achieve a stable and unyielding subgrade condition. One complete coverage is defined as the effort necessary to assure that every square foot of the subgrade is compacted.

L. Compaction operations shall be performed in the presence of the Geotechnical Engineer who will evaluate the performance of the materials under compactive load. Unstable soil deposits, as determined by the Geotechnical Engineer, shall be excavated to expose a firm base and grades restored with engineered fill in accordance with these specifications.

3.3 PLACING, SPREADING AND COMPACTING FILL MATERIAL

a. The selected soil fill material shall be placed in layers which when compacted shall not exceed six inches (6") in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to promote uniformity of material in each layer. Rocky fill materials shall be placed in maximum twelve inch (12") lifts.

b. When the moisture content of the fill material is below that required to achieve the specified density, water shall be added until the proper moisture content of at least the optimum is achieved.

c. When the moisture content of the fill material is too high to permit the specified degree of compaction to be achieved, the fill material shall be aerated by blading or other methods until the moisture content is satisfactory.

d. After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted to at least ninety percent (90%) of the ASTM D1557 maximum dry density for fills placed in the upper five feet (5') from the finished subgrade and to at least ninety-five percent (95%) of the ASTM D1557 maximum dry density for fills placed below five feet (5') from the finished subgrade. Compaction shall be undertaken with a heavy, self-propelled sheepsfoot compactor (Caterpillar 825 compactor, or an equivalent sized compactor), capable of achieving the specified density and shall be accomplished while the fill material is at the required moisture content. Each layer shall be compacted over its entire area until the desired density has been obtained. Rocky soils that are too rocky to test using conventional methods shall be compacted using a performance specification based on the following criteria. For densities comparable to ninety percent (90%) relative compaction, each lift of fill shall be compacted by at least three (3) complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor, to the satisfaction of the Geotechnical

Engineer. For densities comparable to 95 percent relative compaction, each lift of rocky materials shall be compacted by at least five (5) complete coverages (passes) with a Caterpillar 825 compactor, or an equivalent sized self-propelled sheepsfoot compactor. The number of passes shall be considered the minimum and passes shall be added as required by the Geotechnical Engineer to achieve a stable and unyielding subgrade condition. One complete coverage is defined as the effort necessary to assure that every square foot of the subgrade is compacted.

e. Engineered fill placed on the sloping ground steeper than five (5) horizontal to one (1) vertical shall be properly benched into the slopes. Each bench shall consist of a level terrace excavated at least twelve inches (12") into the slope. For every three feet (3') of vertical height of fill, a larger bench shall be constructed, extending at least five feet (5') into the existing slope. Taller slopes shall require a wider bench placed at mid-slope height.

f. The filling operations shall be continued until the fills have been brought to the finished slopes and grades as shown on the accepted Drawings.

3.4 FINAL SUBGRADE PREPARATION

The upper twelve inches (12") of final building pad subgrades and the upper six inches (6") of all final subgrades supporting pavement sections shall be brought to a uniform moisture content, and shall be uniformly compacted to not less than:

building pad	90%
pavement areas	95%

of the ASTM D1557 maximum dry density, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

Rocky soils that are too rocky to test using conventional methods shall be compacted using a performance specification defined in this guide specification and as recommended in the Geotechnical Engineering Report.

3.5 TRENCH BACKFILL

Utility trench backfill shall be placed in lifts of no more than six inches (6") in compacted thickness. Each lift shall be compacted to at least ninety percent (90%) compaction, as defined by ASTM D1557, except that backfill supporting sidewalks, streets or other public pavement shall be compacted to comply with applicable County of Sacramento Standards, latest editions. The upper six inches in pavement areas, the minimum compaction should be ninety-five (95%) percent of ASTM D1557. The upper 12 inches of trench backfill in structural areas (i.e. building pads, exterior flatwork, pavements) should consist of ninety-five percent (95%) compacted material.

3.6 TESTING AND OBSERVATION

a. Grading operations shall be observed by the Geotechnical Engineer, serving as the representative of the Owner.

b. Field density tests shall be made by the Geotechnical Engineer after compaction of each layer of fill. Additional layers of fill shall not be spread until the field density tests indicate that the minimum specified density has been obtained.

c. Earthwork shall not be performed without the notification or approval of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least two (2) working days prior to commencement of any aspect of the site earthwork.

d. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory, as determined by the Geotechnical Engineer and the Architect/Engineer. No deviation from the specifications shall be made except upon written approval of the Geotechnical Engineer or Architect/Engineer.

APPENDIX C

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                         **********************
                         *                    *
                         *    E Q F A U L T   *
                         *                    *
                         *    Version 3.00    *
                         *                    *
                         **********************

                       DETERMINISTIC ESTIMATION OF
                  PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 04843-01
                                                     DATE: 12-30-2019

JOB NAME: LOS RIOS-FOLSOM LAKE COLLEGE

CALCULATION NAME: NEHRP C

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE:  38.6612
   SITE LONGITUDE:  121.1290

SEARCH RADIUS:   62  mi

ATTENUATION RELATION:   2) Boore et al. (1997) Horiz. - NEHRP C (520)
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   DISTANCE MEASURE:  cd_2drp
   SCOND:   1
   Basement Depth:  .10 km      Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED:  C:\Program Files\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km):  0.0
```

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
FOOTHILLS FAULT SYSTEM 1	1.9(3.1)	6.5	0.395	X
FOOTHILLS FAULT SYSTEM 2	13.2(21.3)	6.5	0.151	VIII
FOOTHILLS FAULT SYSTEM 3	20.9(33.6)	6.5	0.107	VII
GREAT VALLEY 3	47.2(75.9)	6.9	0.071	VI
GREAT VALLEY 4	47.2(76.0)	6.6	0.060	VI
FOOTHILLS FAULT SYSTEM 4	48.0(77.3)	6.5	0.057	VI
GREAT VALLEY 5	48.3(77.8)	6.5	0.056	VI
HUNTING CREEK - BERRYESSA	59.4(95.6)	7.1	0.054	VI
WESTERN NEVADA ZONE 1	59.5(95.7)	7.3	0.060	VI
CONCORD/GV (GVN)	59.8(96.3)	6.0	0.030	V
CONCORD/GV (GVS+GVN)	59.8(96.3)	6.5	0.039	V
CONCORD/GV (CON+GVS+GVN)	59.8(96.3)	6.7	0.044	VI
CONCORD/GV (FLOATING)	59.8(96.3)	6.2	0.034	V
CONCORD/GV (GVS)	61.5(99.0)	6.2	0.033	V
CONCORD/GV (CON+GVS)	61.5(99.0)	6.6	0.040	V

****

-END OF SEARCH- 15 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE FOOTHILLS FAULT SYSTEM 1 FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.9 MILES (3.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3953 g

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*************************
*                       *
*    E Q S E A R C H    *
*                       *
*     Version 3.00      *
*                       *
*************************
```

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 04843-01

DATE: 12-30-2019

JOB NAME: LOS RIOS-FOLSOM LAKE COLLEGE

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 38.6612
SITE LONGITUDE: 121.1290

SEARCH DATES:
START DATE: 1800
END DATE: 2018

SEARCH RADIUS:
62.0 mi
99.8 km

ATTENUATION RELATION: 2) Boore et al. (1997) Horiz. - NEHRP C (520)
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
ASSUMED SOURCE TYPE: BT [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: .10 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 0.0

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	38.4000	121.8000	04/30/1892	0 9 0.0	0.0	5.50	0.038	V	40.5(65.1)
T-A	39.2500	121.0000	12/01/1867	712 0.0	0.0	5.00	0.029	V	41.2(66.4)
DMG	38.5000	121.9000	04/21/1892	1743 0.0	0.0	6.20	0.053	VI	43.1(69.3)
DMG	38.3000	121.9000	05/19/1902	1831 0.0	0.0	5.50	0.033	V	48.6(78.1)
DMG	38.4000	122.0000	04/19/1892	1050 0.0	0.0	6.40	0.052	VI	50.4(81.1)
DMG	39.4000	120.9000	03/03/1909	12 0 0.0	0.0	5.00	0.024	V	52.5(84.4)
UNR	39.2450	120.4960	11/28/1980	182112.4	1.5	5.20	0.027	V	52.7(84.8)
DMG	39.4000	120.8000	06/23/1909	724 0.0	0.0	5.50	0.031	V	54.0(86.9)
USG	39.4330	121.4750	08/02/1975	2059 0.0	5.1	5.20	0.025	V	56.4(90.8)
USG	39.4490	121.4730	08/02/1975	202216.2	4.1	5.20	0.025	V	57.4(92.4)
USG	39.4360	121.5230	08/01/1975	202012.0	8.8	5.70	0.032	V	57.5(92.6)
DMG	38.0000	121.9000	05/19/1889	1110 0.0	0.0	6.00	0.036	V	61.9(99.6)

****

-END OF SEARCH- 12 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2018

LENGTH OF SEARCH TIME: 219 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 40.5 MILES (65.1 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.4

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.053 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

a-value= 0.010
b-value= 0.288
beta-value= 0.662

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	12	0.05479
4.5	12	0.05479
5.0	12	0.05479
5.5	7	0.03196
6.0	3	0.01370

APPENDIX D

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APPENDIX E

Petralogix Engineering, Inc.
26675 Bruella Road, Galt, Ca 95632
(T) 209-400-5729
dkramer@petralogix.com
www.petralogix.com

January 11, 2020

Martin Osier, PE
Mid Pacific Engineering, Inc.
840 Embarcadero Drive, Suite 20
West Sacramento, CA 95605
martinosier@midpacificeng.com

Project No. 2019-00096

SUBJECT: Rippability and Vs100 Site Class Determination
Folsom Lake College
Folsom, California

Dear Martin,

We have completed our site rippability and shear wave site assessment in order to assist in characterization of the upper 100 feet to achieve proper site classification. Below you will find a description of our investigation activities including but not limited to conducted field investigations, data processing, data analysis, and final conclusions. We appreciate the opportunity to work with you and MPE on this project. Please feel free to contact our firm with any questions or comments regarding our services and the findings or conclusions detailed in this letter report.

INTRODUCTION

Petralogix performed a total of three (3) individual refraction (P-Wave) and refraction microtremor (REMI) lines. The locations of our survey lines are shown on Plate 2. Plates 3 through 5 (Appendix A) show 2D P-Wave tomography values for each of the survey line locations. From this the site is considered to be marginally to non-rippable below 10 to 15 feet, respectively. Plates 6 through 8 (Appendix A) show 1D derived Vs100 values (in feet per sec) for each of the survey line locations. From this the site can be characterized as a Site Class B. The exact Latitude and Longitudes for transect lines was taken using a Trimble GeoXH 6000, and locations were differentially corrected using Pathfinder Software. The methods which were used to investigate the subsurface soils are more thoroughly explained below:

Standard P-Wave Refraction

Three (3) Standard P-Wave Refraction lines were conducted across the site. The seismic refraction method of geophysical testing is used to obtain vertical p-wave profiles for soil property characterization. It is based on the principle that different materials within the earth have different sound wave propagation characteristics. When a signal source, or shot, is produced, a measurement of its arrival time to a receiver, or geophone, can be measured. When the source is activated, a seismic wave that moves through the interior of the earth is produced (as opposed to surface waves that travel near the earth's surface). This wave is known as a body wave. When this wave encounters a transition into denser material, at some depth, it is refracted into the lower layer as a head wave. Head waves are elastic waves that enter a high-velocity medium (refractor) near the critical angle and travel in the high-velocity medium nearly parallel to the refractor surface before returning to the surface of the Earth. The objective in refraction surveys is to measure the arrival times of head waves as a function of source receiver distance so that the depth to the refractors in which they traveled can be determined (R. E. Sheriff[1]).

Refraction surveys generally consisted of 4.5 Hz geophones spaced 5 feet on center, with hammer shot points at 90, 60, and 30 feet before geophone 1, at geophone 1, and every 30 feet from geophone from 1 through 48. Additional shots were generally taken past geophone 48 at 30 foot intervals. The survey line geometry yielded a total survey line length of between 270 and 320 feet. A total of 11 to 14 records were recorded for each location line. A sample recording rate of 0.250 milliseconds was used for a total time of 0.500 seconds.

<u>Data Processing:</u>

For the Refraction survey, data records are input and analyzed using SeisOpt 2D 6.0 software (Optim Software). SeisOpt@2D uses only the first-arrival travel times and the survey geometry to derive subsurface velocity information. For this reason, accurate picks are important. It uses a nonlinear optimization technique called adaptive simulated annealing that involves forward modeling. Test velocity models are created, through which travel times are calculated. These calculated travel times are compared with the observed data. Testing every possible velocity model would take far too long, so SeisOpt@2D uses Optim's proprietary algorithm to search through only a small percentage of the many possible models, yet still finds the best model. It is called an optimization because the discrepancy, or error, between the calculated and observed travel times is optimized. In this case, the optimal solution is the velocity model with the minimum travel-time error. From this a 2D image is created that shows the compression wave velocity (P-wave) for the entire survey line.

<u>Refraction Microtremor (REMI)</u>

Three (3) Refraction Microtremor (REMI) were conducted across the site. The purpose of these surveys was to find the approximate shear (s-wave) wave velocities for onsite soils. Using this data, we can then assist in providing proper site classification (Vs100). The Shear Wave Refraction Microtremor Technique of geophysical testing (or REMI) is applied to obtain vertical s-wave profiles. Testing is performed using the same equipment as that used for standard Refraction Surveying. The source for this technique is ambient noise (microtremors) which are present within the earth at all times. These noises are generated by both natural and cultural processes. Additional noise can be added to the survey in an active manner by such means as jogging along the survey line, or by striking a steel plate during the survey. Two-dimensional profiling can also be performed using this method by compiling 1D segments at selected intervals and interpolated values between known REMI 1D locations.

<u>Data Acquisition:</u>

For REMI surveying two (2) Geode 24-Bit Acquisition Systems were used for the surveys. The Refraction Microtremor surveys consisted of 4.5 Hz geophones spaced 5 feet on center, yielding a total survey line length of 235 feet. For the Remi survey, a total of 20 records were recorded for each location at a sampling rate of 2

[1] R.E. Sheriff, L.P. Geldart, 1995, Exploration Seismology, 2nD Edition, University of Cambridge.

milliseconds (0.002s) for a total time of 30 seconds. Down-line distances were measured using a survey tape to within approximately 0.1 feet.

Data Processing:

For the REMI survey, data was collected on a field computer and then converted into a spectral energy shear wave frequency versus shear wave velocity (or slowness) image using both SeisOpt Remi 4.0 software (Optim Software). From the created data images, a number of values are picked that represent the lower boundary of the spectral energy shear velocity versus frequency trend. These picked values are plotted in a second module of the aforementioned program. Dispersion inversion (automated equation analysis) software then derives multiple layers and s-wave velocity conditions for the survey line. From this the most likely scenario for the site is interpreted. It must be understood that this type of interpretation may not result in a unique solution. From this a 1D image is created that shows the sum-averaged shear wave velocity for the length of analyzed survey line.

FINDINGS AND CONCLUSIONS

For all P-Wave Refraction Lines (1, 2, and 3), the p-wave velocity was below 5,000 ft/sec for the upper 15 to 20 feet of material. Below that it rapidly increases depending on exact location. Below 15 feet the p-wave velocity may exceed 10,000 feet per second. Volcanic and meta-volcanic rocks in this velocity range become marginally rippable. Depending on exact equipment, materials may become non-rippable using standard grading equipment. If cuts and excavation are to exceed 10 to 15 feet, materials will be encountered that are difficult to rip using standard equipment. Caution should be exercised when sizing equipment and excavation methods for depths below 15 feet below ground surface.

For Line #1 the Vs100 estimations were (REMI) 2938 ft/sec. For Line #2 the Vs100 estimations were (REMI) 3471 ft/sec. For Line #3 the Vs100 estimations were (REMI) 3337 ft/sec. Based on these values for the site we considered this to be a Site Class B.

LIMITATIONS

The professional findings contained in this geophysical assessment are strictly based on a limited testing over a large site, and are also based on the information provided regarding the proposed construction, and the geophysical sounding locations assessed. Furthermore, the analysis, conclusions and recommendations contained in this report are based on the site conditions as they existed at the time we performed our investigation.

Herein, it is assumed that the geophysical test locations are representative of the subsurface conditions throughout the site, however, it should be noted that they are non-unique in many cases. Without direct evidence a level of uncertainty exists. It is standard practice to perform test drilling in areas of hazard concern, and without this information a full evaluation cannot be completed.

If there is a substantial lapse of time between the submission of this report and the start of the work at the site for test drilling, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we urge that our report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only for the project and site studied. This report should not be used after 3 years.

Our professional services were performed, our findings obtained, and our professional opinions are in accordance with generally accepted geologic principles and practices. This warranty is in lieu of all other warranties either expressed or implied. Our findings do not constitute a guarantee or warranty, expressed or implied.

If you have any questions do not hesitate to call us to discuss in more detail. We appreciate the opportunity to work on this project. As a company that values long-term relationships, we look forward to being able to help you be a sustainable and long-term success and provide the best and most affordable services available.

Warm Regards,

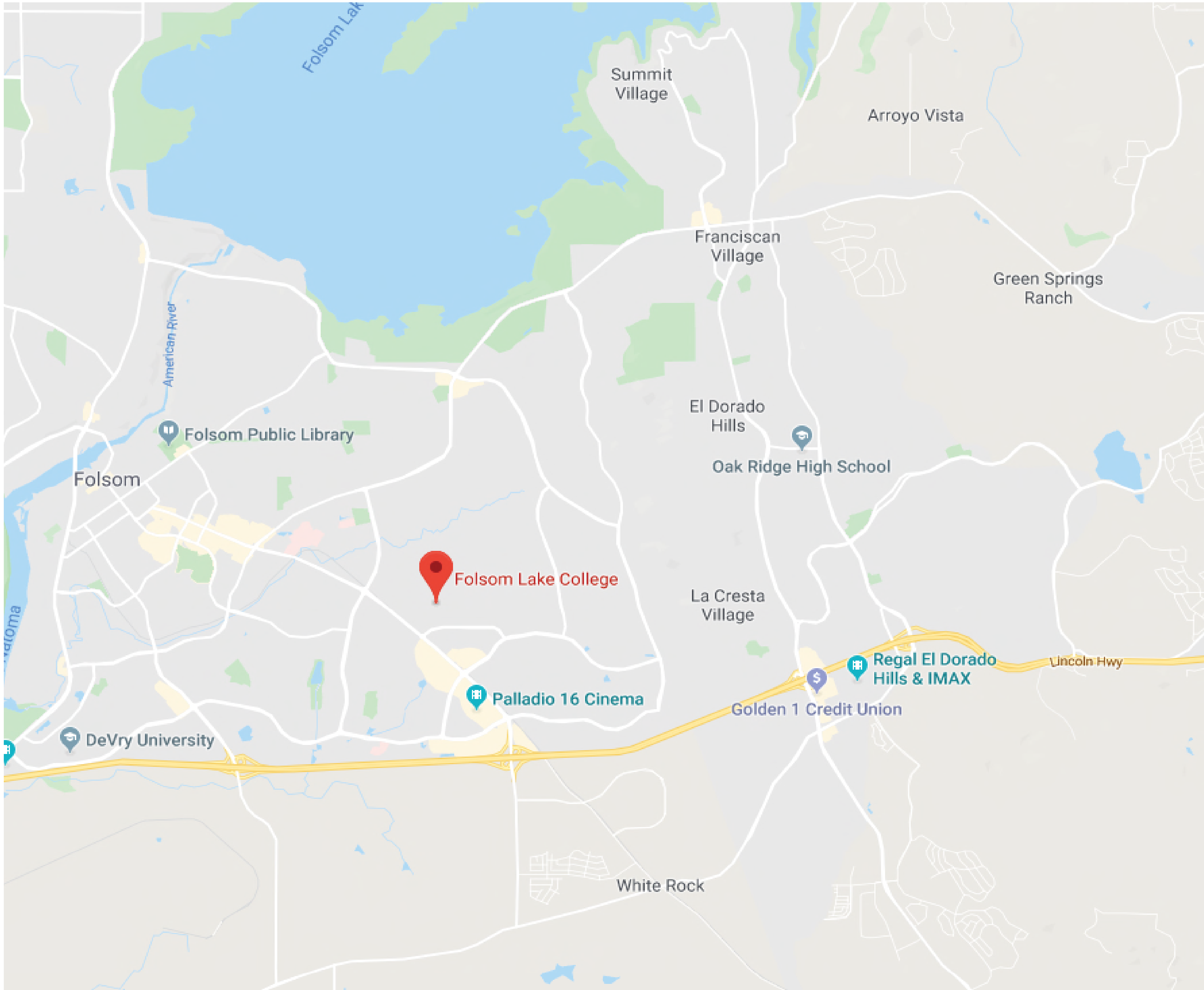
PROFESSIONAL GEOPHYSICIST
DANIEL E.
KRAMER
Pgp 1078
STATE OF CALIFORNIA

Daniel E. Kramer, President
Professional Geologist 8657
Certified Engineering Geologist 2588
Professional Geophysicist 1078

Appendix A

Folsom Lake Project - Geophysical Survey

Vicinity Map



DATE: 01-11-20

JOB NUMBER: 2019-00096

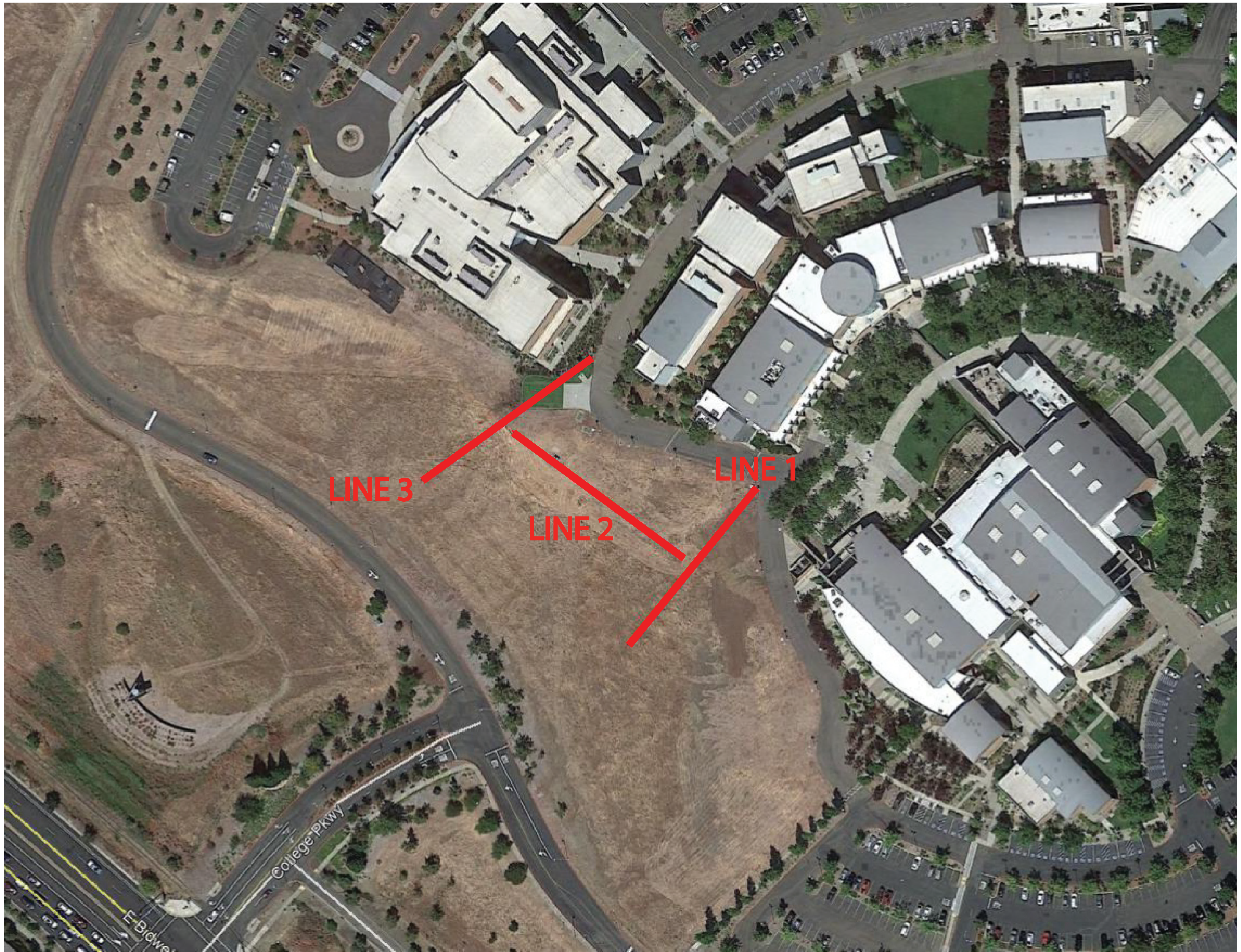
SCALE: Not to Scale

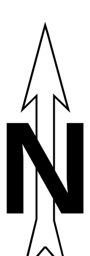
DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 1

Folsom Lake Project - Geophysical Survey
Site Survey Map





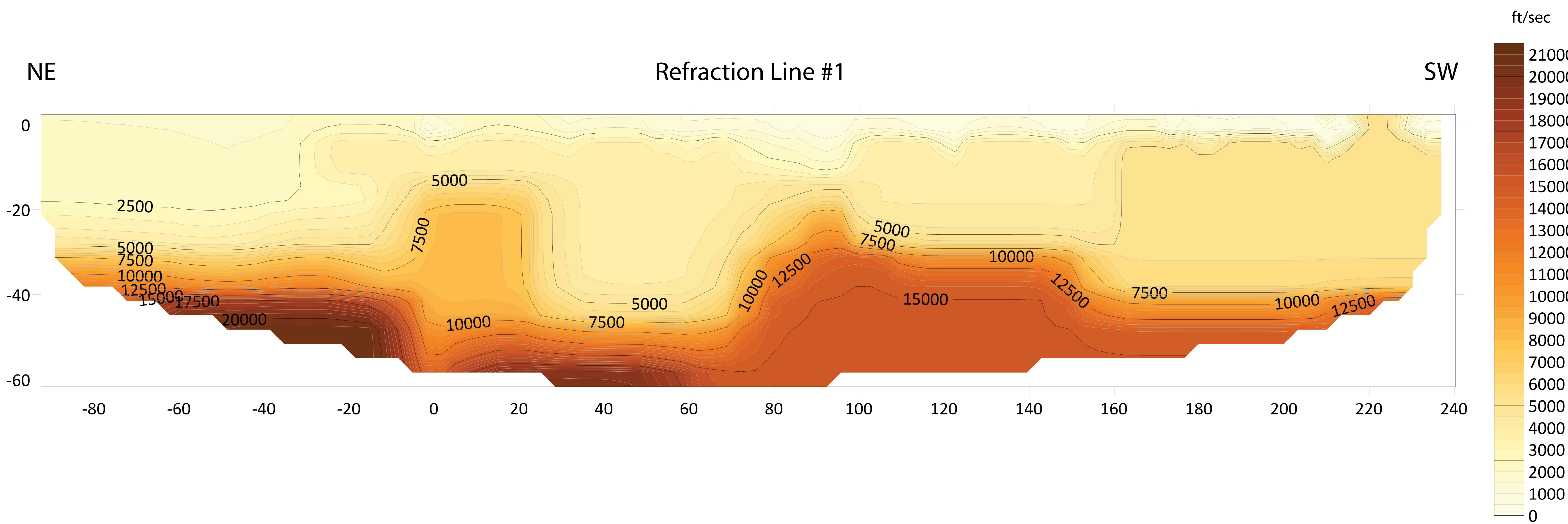
LEGEND

Survey Line Locations

REF-REMI Lines

DATE: 01-11-20
JOB NUMBER: 2019-00096
SCALE: Not to Scale
DRAWN BY: DK
CHECKED BY: DK
PLATE NO. 2

Folsom Lake Project - Geophysical Survey



DATE: 01-11-20

JOB NUMBER: 2019-00096

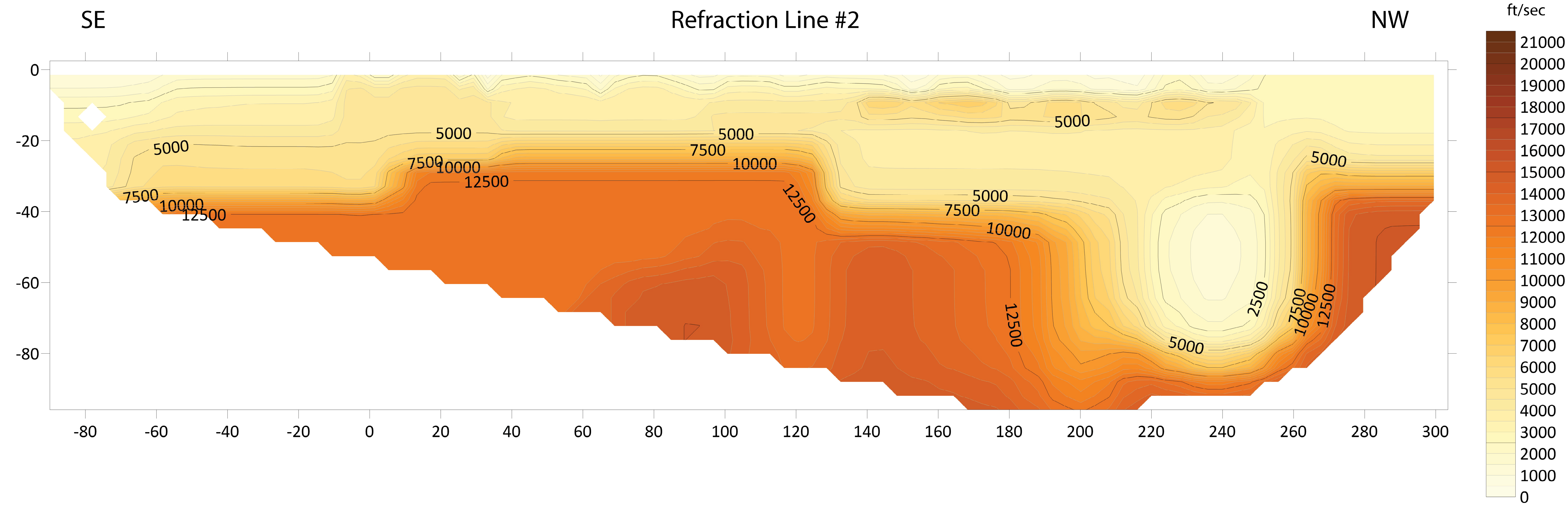
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 3

Folsom Lake Project - Geophysical Survey



DATE: 01-11-20

JOB NUMBER: 2019-00096

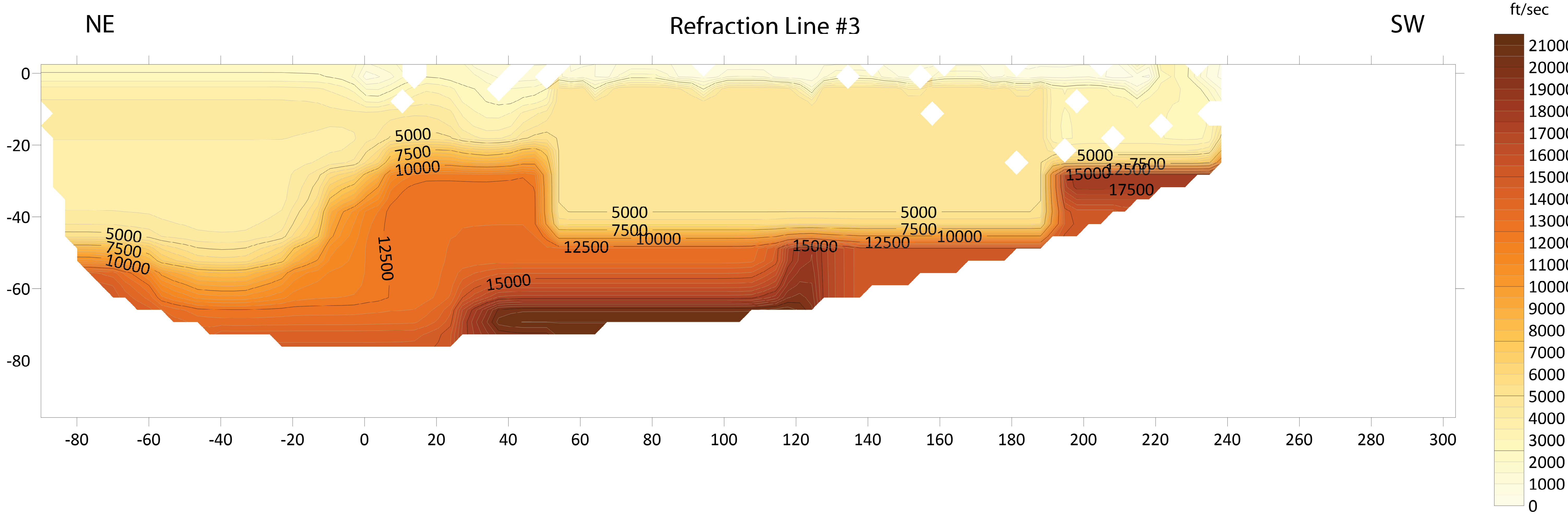
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 4

Folsom Lake Project - Geophysical Survey



DATE: 01-11-20

JOB NUMBER: 2019-00096

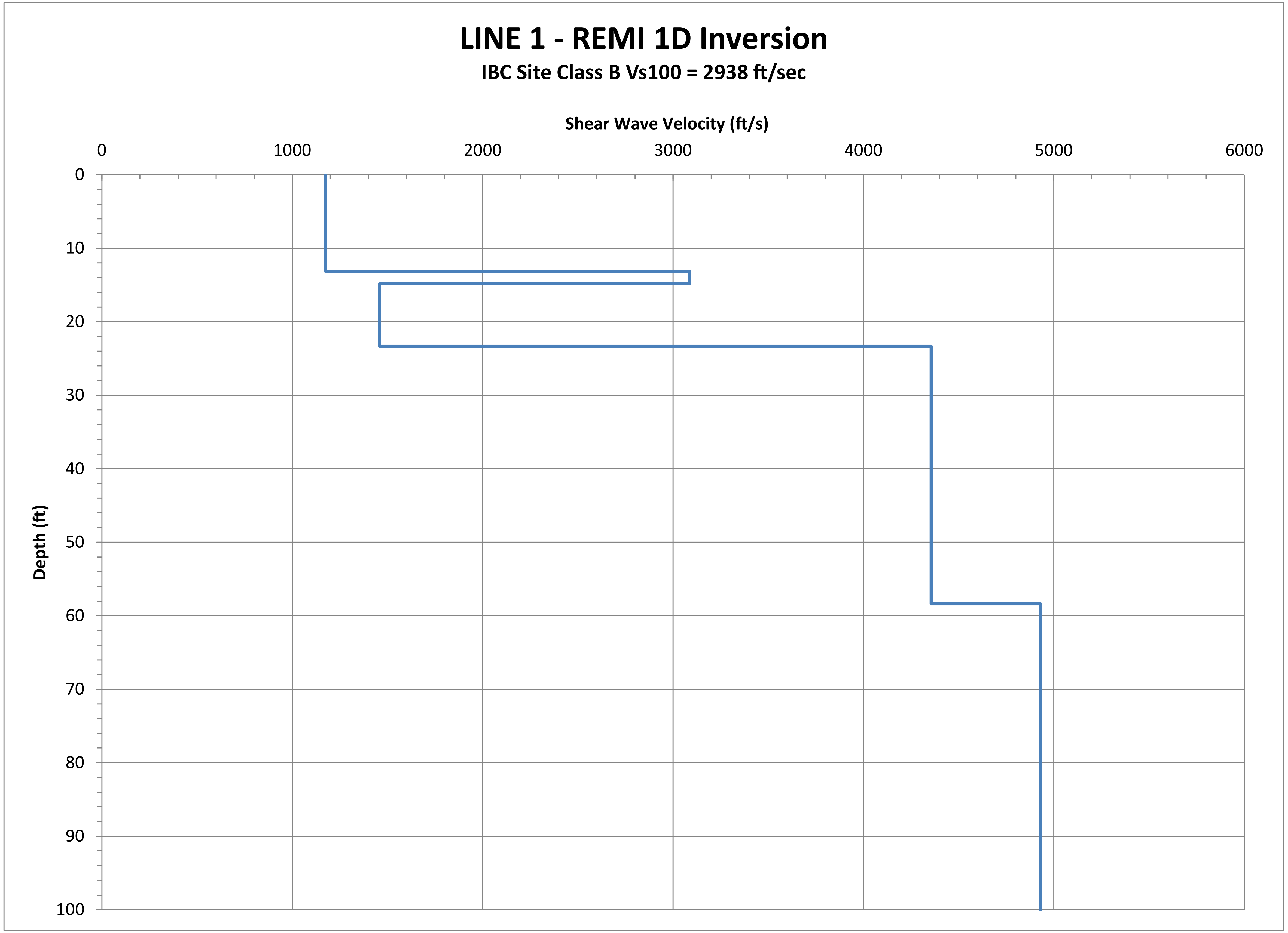
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 5

Folsom Lake Project - Geophysical Survey



DATE: 01-11-20

JOB NUMBER: 2019-00096

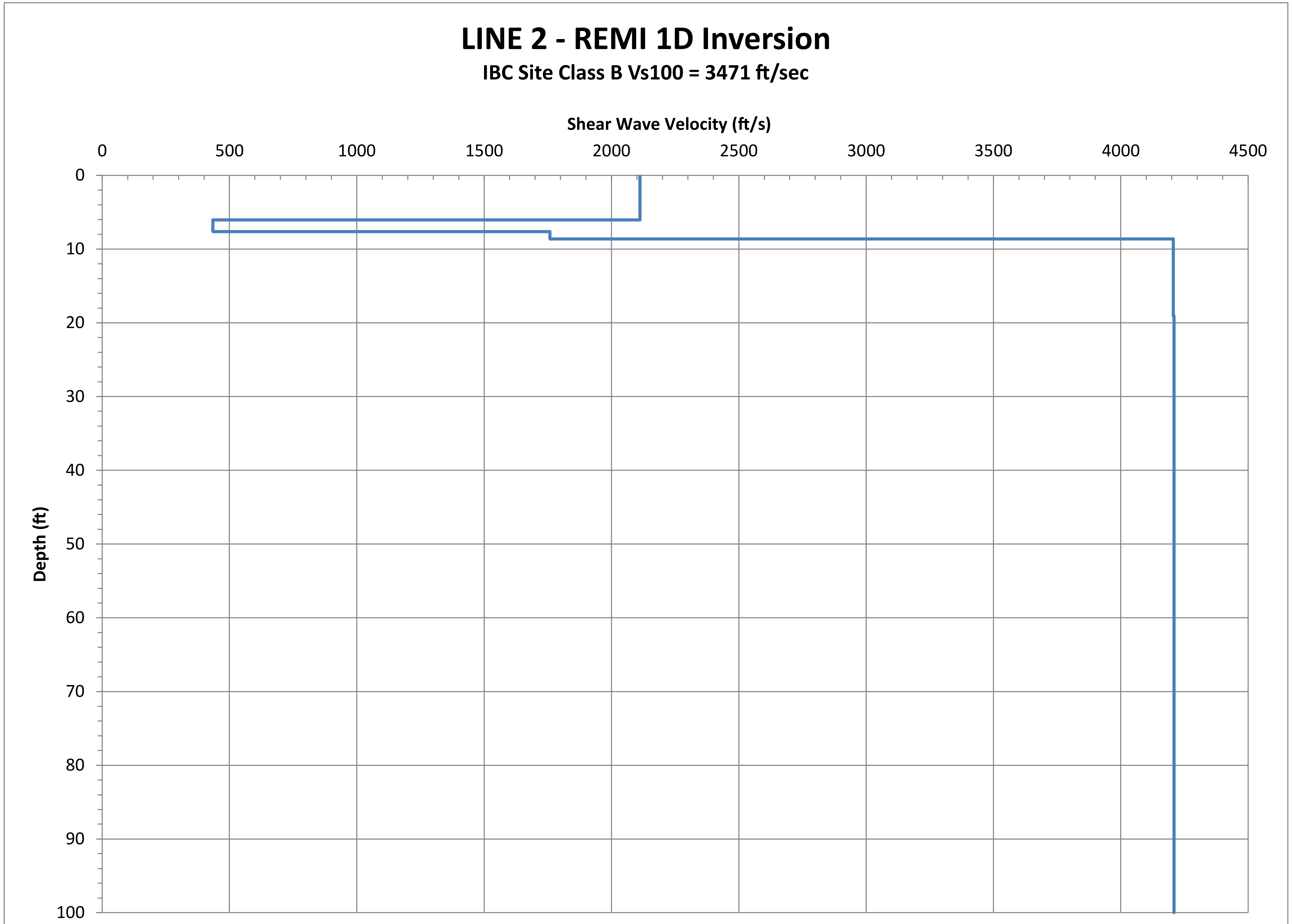
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

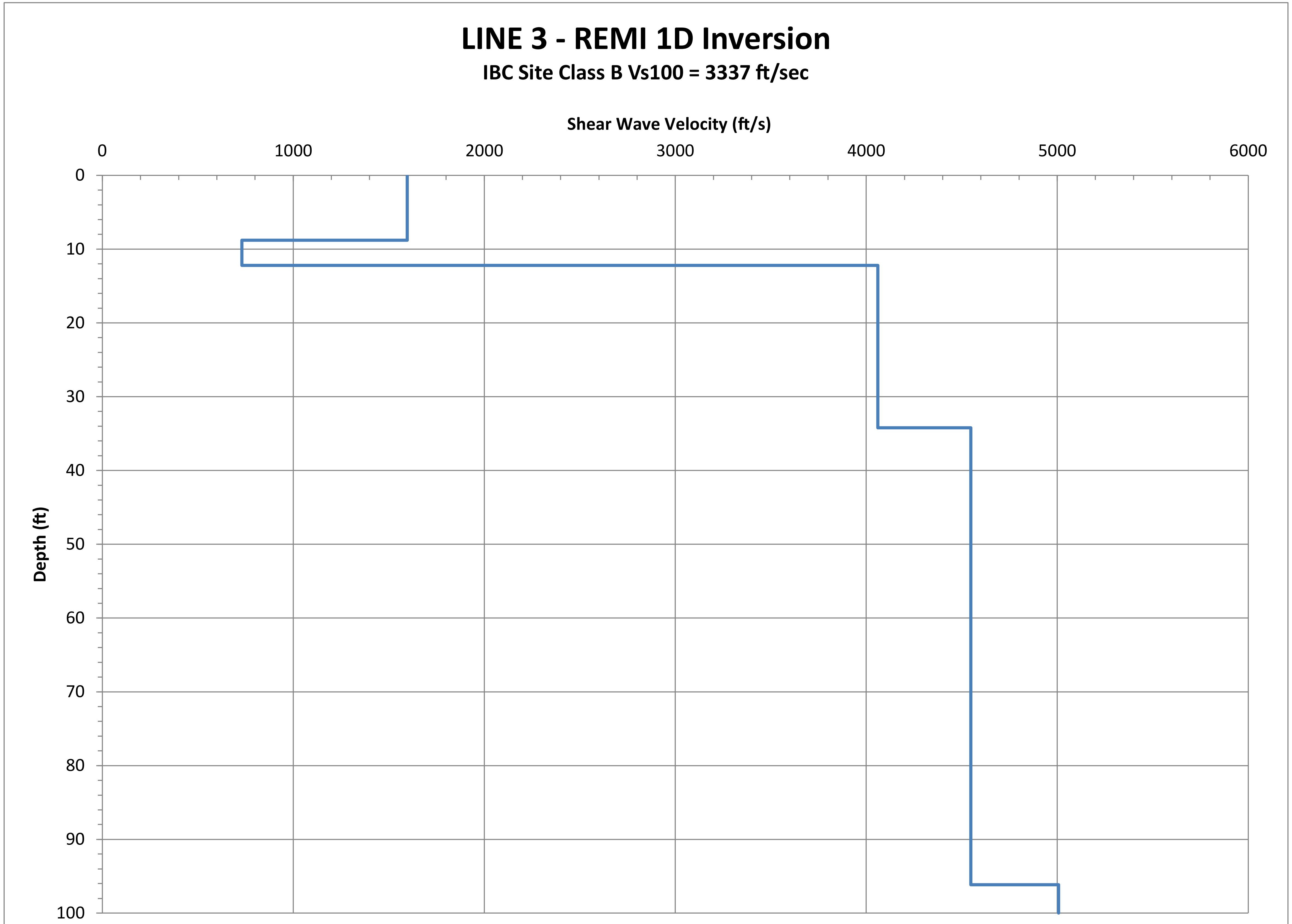
PLATE NO. 6

Folsom Lake Project - Geophysical Survey



DATE: 01-11-20
JOB NUMBER: 2019-00096
SCALE: Not to Scale
DRAWN BY: DK
CHECKED BY: DK
PLATE NO. 7

Folsom Lake Project - Geophysical Survey



DATE: 01-11-20

JOB NUMBER: 2019-00096

SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 8