APPENDIX D

Geologic Hazards
&
Geotechnical Engineering Report

AMERICAN RIVER COLLEGE
TECHNICAL EDUCATION BUILDING

Sacramento, California
MPE No. 04842-01

MID PACIFIC ENGINEERING, INC.

January 15, 2020

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Geologic Hazards and Geotechnical Engineering Report

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING

4700 College Oak Drive
Sacramento, California
MPE No. 04842-01
January 15, 2020

INTRODUCTION

We have completed a Geologic Hazards and Geotechnical Engineering Investigation for the proposed Technical Education Building to be constructed at the existing American River College campus located at 4700 College Oak Drive in Sacramento, 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:
    - Project site plan, transmitted to the office of MPE via e-mail on dated November 22, 2019 (Figure 3);
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, including the drilling, logging, and sampling nine exploratory soil borings to approximate maximum depths of 3½ to 51½ feet below existing ground surface (bgs) within area proposed for the structure;

7. Collection of bulk and in-situ soil samples at various depths within the borings;
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	14	Geologic Cross-Section A^1-A^4
2	Regional Geologic Map	15	Geologic Cross-Section B^1-B^2
3	Boring Location Map	16	Regional Fault Map
4 - 12	Logs of Soil Borings	17	Earthquake Epicenter Map
13	Unified Soil Classification System	18	FEMA Flood Map

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 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 - Output of LiqSVs Analyses.
- Appendix E - A list of references cited.
- Appendix F - *V_s100 Class Determination* report, prepared by Petralogix, dated January 10, 2020.

PROPOSED DEVELOPMENT

Based on our review of the aforementioned plan, it is our understanding the project will consist of the construction of an L-shaped building with an overall footprint of approximately 45,000 square feet (sf). It is anticipated that the proposed building will be a one- and two-story, steel frame structure, with a concrete slab-on-grade ground floor, supported on a conventional foundation system. Information regarding structural loads was not available at the time we prepared this report, but we assume the loads will be light to moderate based on the on the anticipated construction.

Associated development is anticipated to include construction of a new parking lot, underground utilities, light poles, exterior flatwork, and landscaping.

Grading plans were not available at the time we prepared this report; however, for the purposes of preparing this report and based on the relatively level site topography, we anticipate earthwork cuts of up to one foot and fills of up to three feet in depth will be constructed to achieve final pad elevations.

FINDINGS

SITE DESCRIPTION

The project site is within the north-eastern portion of the American River College campus located at 4700 College Oak Drive in Sacramento, California. The approximate location of the project is north latitude 38.6512° and west longitude 121.3455°.

The site is generally bounded to the north by a parking lot and the irregular shaped Technical Education/Auto/Welding building; to the east by Arcade Creek Nature Area and Environmental Resources structures; to the south by Health & Education North structures and Child Development Center; and, to the west by the existing driveway, beyond which are parking lot and Student Center. On the dates of our investigation, the project site was occupied by four standing alone buildings (irregular shaped Technical Education/Auto/Welding building, Physics and Engineering building, cross-shaped Office building, and storage building), as well as storage areas, paved area, exterior flatwork, and landscaped areas. Mature trees and numerous underground utilities are present throughout the site. It appears that landscaped areas were extensively irrigated.

Topography across the site is relatively flat with an average surface elevation of approximately +90 feet relative to mean sea level (msl), based on review of the topographic information presented on the *United States Geological Survey (USGS) 7.5 Minute Series Topographic Map of the Citrus Heights Quadrangle, California (1975)*. Portions of the USGS map containing the site and vicinity, is included with this report as Figure 1. The project site topography gently slopes to the east.

SITE HISTORY

The project site history was compiled based on review of historical aerial photographs (dated 1947, 1957, 1964, 1966, and 1975), Google Earth images (dated 1993, 1998, 2002 through 2018), and USGS historical maps (1951 (1952 and 1956 photorevisions), and 1967 (1969 photorevision).

The site was an undeveloped land at least until 1956. The portions of the Auto/Welding building were under construction on 1957 aerial photo. By 1964, eastern Auto/Welding portion of the building and portions of Physics and Engineering building have been constructed. By 1966, the surrounding parking lots have been constructed. By 1975 all buildings on site have been constructed. The site remained essentially unchanged since 1993.

GEOLOGIC SETTING

REGIONAL GEOLOGY AND STRUCTURE

The project site lies in the northern portion of the Great Valley geomorphic province of California. The Great Valley is an alluvial plain, approximately 50 miles wide and 400 miles long, between the Coast Ranges to the West and Sierra Nevada geomorphic provinces to the East. Within the northern portion, the Great Valley is drained by the Sacramento River, which enters San Francisco Bay. The eastern border is the west-sloping Sierran bedrock surface, which continues westward beneath alluvium and older sediments. The western border is underlain by east-dipping Cretaceous and Cenozoic strata that form a deeply buried synclinal trough, lying beneath the Great Valley along its western side.

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 Early Pleistocene Turlock Lake Formation (Map Symbol: Qtl), described as alluvial-fan deposits derived from glaciated drainage basins and consist of predominantly sand with silt, and minor gravel.

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 Urban land (227), Liveoak sandy clay loam, 0 to 2 percent slopes, occasionally flooded (172), and Liveoak-Urban land complex, 0 to 2 percent slopes (173). Urban land consists of large areas covered by impervious surfaces or structures. The soil beneath the impervious structures may have been altered during construction. The Liveoak sandy clay loam, 0 to 2 percent slopes, occasionally flooded are very deep, well-drained soil on narrow, high flood plains and formed in alluvium derived from granitic rocks. These soils possess moderate permeability, slow runoff, and slight hazard of erosion. The soil is occasionally flooded for very brief or brief periods during prolonged, high-intensity storms. Liveoak-Urban land complex, 0 to 2 percent slopes is similar to the Liveoak sandy clay loam soils, except that channeling and construction of diversions have reduced the hazard of flooding.

SUBSURFACE SOIL CONDITIONS

The near-surface soils encountered in the test borings consist of predominantly soft to medium stiff clays to depths of 1 to 5½ feet bgs underlain by interbedded layers of very stiff to hard and variably cemented clayey silts, medium dense to dense and variably cemented sandy silts, medium dense to dense silty sands, and medium dense clayey sands to the maximum depth explored 51½ feet bgs. Fill soils consisting of sandy silts were encountered in one test boring D9 and extended to a depth of at least three feet.

For soil conditions at a specific location, please refer to the Logs of Soil Borings (Figures 4 through 12). An explanation of the symbols and classification system used on the Logs is presented on Figure 13. Graphic illustrations of the subsurface conditions encountered in the borings are presented on the geologic cross-sections (Figures 14 and 15).

GROUNDWATER

Groundwater was not encountered in borings advanced on November 23 and 24, 2019, to the maximum depth explored of 51½ feet bgs. Review of the *Depth to Groundwater Maps* produced by the California Department of Water Resources for the period from 2011 through 2018 indicates that the depth to groundwater ranged from 120 to 125 feet bgs.

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 the Green Valley Fault System, located approximately 50.6 miles (81.5 kilometers) southwest of the project site. A Regional Fault Map (Figure 16) 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 fault to the site is the Great Valley 4a, Trout Creek Fault, located approximately 37.6 miles (60.5 kilometers) west 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_{wMAX}) 6.5 or greater. The maximum magnitude value represents the maximum earthquake believed possible for each fault.

Table 1 - Faults and Fault Rupture Segments Influential to American River College

Fault Name	Maximum Magnitude (M_W)	Distance To Site Miles (Kilometers)
Great Valley 4a, Trout Creek	6.6	35.5 (57.2)
Great Valley 3, Mysterious Ridge	7.1	36.0 (57.9)
Great Valley 4b, Gordon Valley	6.8	36.0 (57.9)
Great Valley 5, Pittsburg Kirby Hills	6.7	38.3 (61.7)
Hunting Creek-Berryessa	7.1	48 (77.2)
Green Valley Connected	7.0	48.7 (77.2)
West Napa	6.7	57.6 (92.7)
Greenville Connected	7.0	55.2 (88.8)
Great Valley 2	6.5	59.4 (95.6)

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 13.5 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 Mountain Fault Zone (Rescue lineament) of the Foothills Fault System located approximately 24.7 miles (39.8 kilometers) northeast from the site. The nearest mapped fault to the site is the concealed Pre-Quaternary Willows Fault Zone located approximately 5.0 miles (8.0 kilometers) southwest 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 or greater) within the project region is presented as Figure 17.

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 they produced estimated site peak ground accelerations of 0.048 to 0.067 g. The closest epicenter is located approximately 30.1 miles (48.4 kilometers) southwest 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 VI[1] as the result of these earthquakes.

Among the most recent earthquakes, the 2000 Yountville (Mw=5.0) and the 2014 South Napa (Mw=6.0) events produced estimated site peak ground accelerations of 0.021g and 0.037g, 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.

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[1] VI – Strong: Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight..

TABLE OF MAGNITUDES AND EXCEEDANCES	
Earthquake Magnitude	Number of Times Exceeded
5.0	16
5.5	9
6.0	4

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

A shear wave site assessment for characterization of the upper 100 feet was performed for this site by Petralogix (reference *V_s100 Site Class Determination* report, dated January 10, 2020). Results of the assessment are included in Appendix F.

Based on the average value of the soil shear wave velocities (REMI Average of 1327/ft/sec and MASW Average of 1768 ft/sec) for the upper 100 feet of the project site, it is our opinion that Site Class C is most applicable to the soil conditions on site.

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: 38.6512° N Longitude: -121.3455° 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	0.469 g
1.0 Period MCE	Figure 22-2	Figure 1613*A*.3.1(2)	S_1	0.228 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}	0.61 g
	Equation 11.4-2	Equation 16*A*-38	S_{M1}	0.343 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16*A*-39	S_{DS}	0.404 g
	Equation 11.4-4	Equation 16*A*-40	S_{D1}	0.228 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 0.238 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 a 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 east side 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 #5310) demonstrated latest Pleistocene and probable Holocene displacement along traces of the Hunting Creek fault. Slip rate of between 1 and 5 mm/yr assigned for the 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=7.03 event), Great Valley Fault System (M=7.02 event), Foothills Fault System (M=6.01 event), and Green Valley Fault System (M=6.77 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), 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 by predominantly medium dense to very dense and partially cemented silty sands and sandy silts. The historical depth to groundwater is deeper than 50 feet. The isolated layers of near-surface soft clays, where present, will be over-excavated and recompacted. 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

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 generally stiff/dense soils encountered in the borings during our geotechnical exploration, it is our opinion the potential for site seismically induced ground subsidence is low.

Dry Sand Seismic Settlement

Dry sand seismic settlement can be evaluated using the method of Pradel (1998). This method is a simplified method based on earlier work by Tokimatsu and Seed (1987) applicable to sands. Modelling of the soil conditions encountered in the borings D1 and D6 using the LiqSVs software, utilizing SPT data, site ground motion of 0.24g, and earthquake magnitude of 7.1 indicates total dry sand seismic-induced settlements of less than approximately $^{1}/_{8}$-inch for both borings. Conservatively, total and differential settlements of ⅛-inch and ⅛-inch in 40 linear feet, respectively, should be anticipated for the design. Output files of LiqSVs software are presented in the Appendix D.

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).

Landslides and Slope Stability

The site is not located in a Landslide Hazard Zone as designated by the State of California. Considering the essentially flat site topography, the potential for development of the landslides or slope instability is negligible.

Tsunami

The project site is well inland and there are no significant bodies of standing water near the site; therefore, the potential for tsunamis influencing the site is negligible.

Seiche

There are no significant bodies of standing water near the site; therefore, the potential for seiches influencing the site is negligible.

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 01182H, Map Number 06067C0088H-

06067C0089H, published by FEMA, with an effective date of August 16, 2012, 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 18).

Review of the maps published by Sacramento Area Flood Control Agency indicates the site is not located in the area of inundation due to the 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 78 miles (126 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 45 miles (72 kilometers) north 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.

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

The site contains existing buildings, pavements and exterior flatwork, and trees; therefore, proper clearing and removal of existing improvements and proper backfilling of excavations is very important to provide adequate and uniform structural support. Demolition of existing buildings and site clearing operations 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.

It should be noted the soils exposed immediately beneath existing buildings, flatwork and pavements, and soils within grass covered (and irrigated) areas may be wet, soft or unstable requiring additional over-excavation to expose a firm base or a stabilized subgrade on which to begin engineered fill placement.

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

Our work indicates that undisturbed and re-compacted native soils and engineered fill, properly placed and compacted in accordance with the recommendations of this report, will be capable of supporting the proposed structures and associated improvements.

Provided the over-excavation, processing, and re-compaction of on-site disturbed soils is 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 surface and near-surface native soils exhibit medium expansion potential. In our opinion these soils, when present within the upper portion of the building pad, are capable of exerting moderate to high 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 and A2.

SUITABILITY OF ON-SITE SOILS FOR USE AS FILL

The on-site soils 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. 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. We will recommend the upper 12 inches of building pad and exterior flatwork subgrades consist of non-expansive, granular on-site or imported soils, or Class 2 aggregate base. Alternatively, the upper 12 inches of the building pad could be lime-treated. Expansive clays will not be allowed within the upper 12 inches of building pad or exterior flatwork fills, unless lime-treated.

EXCAVATION CONDITIONS

Based on our field investigation, the on-site native soils should be readily excavatable with conventional earthmoving and trenching equipment typically used in the area. Excavations encountering the variably cemented soils will be slower to excavate; although, special trenching and excavation equipment are not anticipated for this project.

In general, we anticipate soil sidewalls 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	
		D2 @ 0-3’	D4 @ 0-3’
Soil pH	CA DOT 643 Modified*	6.63	6.71
Minimum Resistivity	CA DOT 643 Modified*	1,740 Ω-cm	2,390 Ω-cm
Chloride	CA DOT 417	13.3 ppm	25.6 ppm

Sulfate	CA DOT 422	6.2 ppm	15.1 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, majority of the near-surface soils consist of silty and sandy clays which when tested in accordance with California Test (CT) 301 are poor quality materials for the support of asphalt concrete pavements possessing Resistance (“R”)-value of 18, (see Figure A3). Based upon the test results, and considering the natural variation in soils, it is our opinion that an R-value of 10 is considered appropriate for design of pavements at this site.

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.

However, it is possible that perched or seepage water may be present within excavations, depending upon the time of year when construction takes place due to surface water becoming trapped over the on-site clayey and cemented soils.

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 and underlying cemented 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 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. Localized dewatering, if required, can likely be accomplished by using sump pumps.

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 clays and 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 of up to one foot and fills of up to three feet for development of the planned improvements. The recommendations contained in this report are based upon this assumption.

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.

Our review of available historical photographs provides a limited site history. Therefore, unknown buried structures or remnants of former structures may be present on-site and may be encountered during construction. If encountered, these structures should be removed and the resulting cavities or holes should be backfilled with properly moisture conditioned and compacted engineered fill as described in this report.

SITE CLEARING

Initially, all structural areas of the site should be cleared of demolition debris and rubble, pavements, foundations, slabs-on-grade, underground utilities scheduled for removal, trees, vegetation, and other deleterious materials to expose firm and stable soil conditions as identified by our on-site representative.

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 pads 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”).

Trees designated for removal should include the entire root ball and all surface roots larger than ½-inch in diameter. Adequate removal of debris, rubble, and tree roots 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 and former structures, as well as trees and roots, 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, or depressions are not left open as recommended, deeper cross-ripping and/or over-excavation of the disturbed areas, building pads 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 and/or over-excavation and re-compaction.

The contractor should anticipate additional sub-excavation, backfilling and reworking of the areas containing existing or former structures. We recommend construction bid documents contain a unit price (price per cubic yard) for additional excavation of unsuitable materials and replacement with engineered fill.

The depth and lateral extent of site disturbance is not yet known; therefore, it will be essential that our office be present on-site to observe the site conditions during the clearing to determine the depths and lateral extents of sub-excavations required to provide uniform structural support. As a minimum, all disturbed areas will require sub-excavation to depths that will expose firm, undisturbed native soils. Actual depths will vary, but sub-excavations of least one to three feet should be anticipated.

Following site clearing operations, the bottoms of all excavations and sub-excavations 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.

All other structural areas 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 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.

Special care should be taken when compacting near to the existing structures to prevent damage to the existing structures. Vibratory compaction should not be used near the existing structures.

Remnants of Former Construction

The potential exists that remnants from former construction and loose and/or unstable, undocumented fills associated with former site development may be present on the site and extend deeper than the recommended depth of ripping and/or sub-excavations. If loose or unstable fills are exposed during compaction operations, those areas exhibiting instability should be excavated to expose a firm base and backfilled with engineered fill. Our representative should be present during the grading operations to identify and verify adequate removal of exposed structures and loose fills and observe and test proper backfilling of required excavations.

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 815, 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

Engineered fill should be placed in horizontal lifts not exceeding six inches in compacted thickness. Engineered fill should be brought to at least the optimum moisture content and compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. Compaction operations should be undertaken with a heavy, self-propelled, sheepsfoot compactor capable of providing proper compaction to the full depth of each lift of fill. Additional passes with the compactor shall be added, as required by the Geotechnical Engineer, to achieve a firm, stable and unyielding subgrade condition. Compactive effort should be applied uniformly across the full width of fill construction. Care must be taken when compacting at the edges of the over-excavations, to ensure that the fills are uniformly tied into the adjacent sloping ground by benching into undisturbed native soil. Each lift of engineered fill should be properly benched into adjacent side slopes, if present, to remove loose soils and promote uniformity.

The on-site soils will be suitable for use as engineered fill if the materials are at a workable moisture content and free of rubbish, rubble, debris and concentrations of organics, and have a maximum particle size of three inches or less. Hand picking of exposed roots, rubbish, debris, and over-sized material should be performed by the Contractor to adequately clear the grades and properly prepare and clear the soils proposed as fill, prior to use.

The upper 12 inches of building pad and exterior flatwork subgrades should consist of approved imported, non-expansive, granular soils, or Class 2 aggregate base. Alternatively, the upper 12 inches of the building pad and flatwork subgrades could be lime-treated. Clays should not be used within the upper 12 inches of building pad or exterior flatwork fills, unless lime-treated.

Imported fill material, if required, should consist of well-graded granular soils or well-graded aggregates with a Plasticity Index of 15 or less, an Expansion Index of 20 or less and should have no 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. The imported materials should be sampled, tested, and approved before being transported to the project site. Samples should be submitted to the Geotechnical Engineer at least two weeks prior to planned importation to the site.

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, 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.

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. 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 pads and slab-on-grade subgrades should be non-expansive granular soils or aggregate base.

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 1⅛-inch and ⅝-inch in 40 linear feet, respectively, should be anticipated for the design of the proposed foundations.

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 (Ω0) 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 minimum, continuous foundations should contain *at least* four No. 4 steel reinforcing bars placed two each, near the top and bottom of the foundations. The project designer should determine the need for additional reinforcement based on structural 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 0.25 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 300 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.

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 minimum of 12 inches of imported, non-expansive soil subgrades 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.

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 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.

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

Due to the near-surface soils primarily consisting of silty and sandy clays, it is our opinion that an R-value of 10 should be used for pavement design (Figure A3).

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 = 10	
	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2½	9
5.0	2½	10
	3*	9
6.0	2½	14
	3½*	12
6.5	3	15
	3½*	14
7.0	3	16
	4*	14

* = 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. 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 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.

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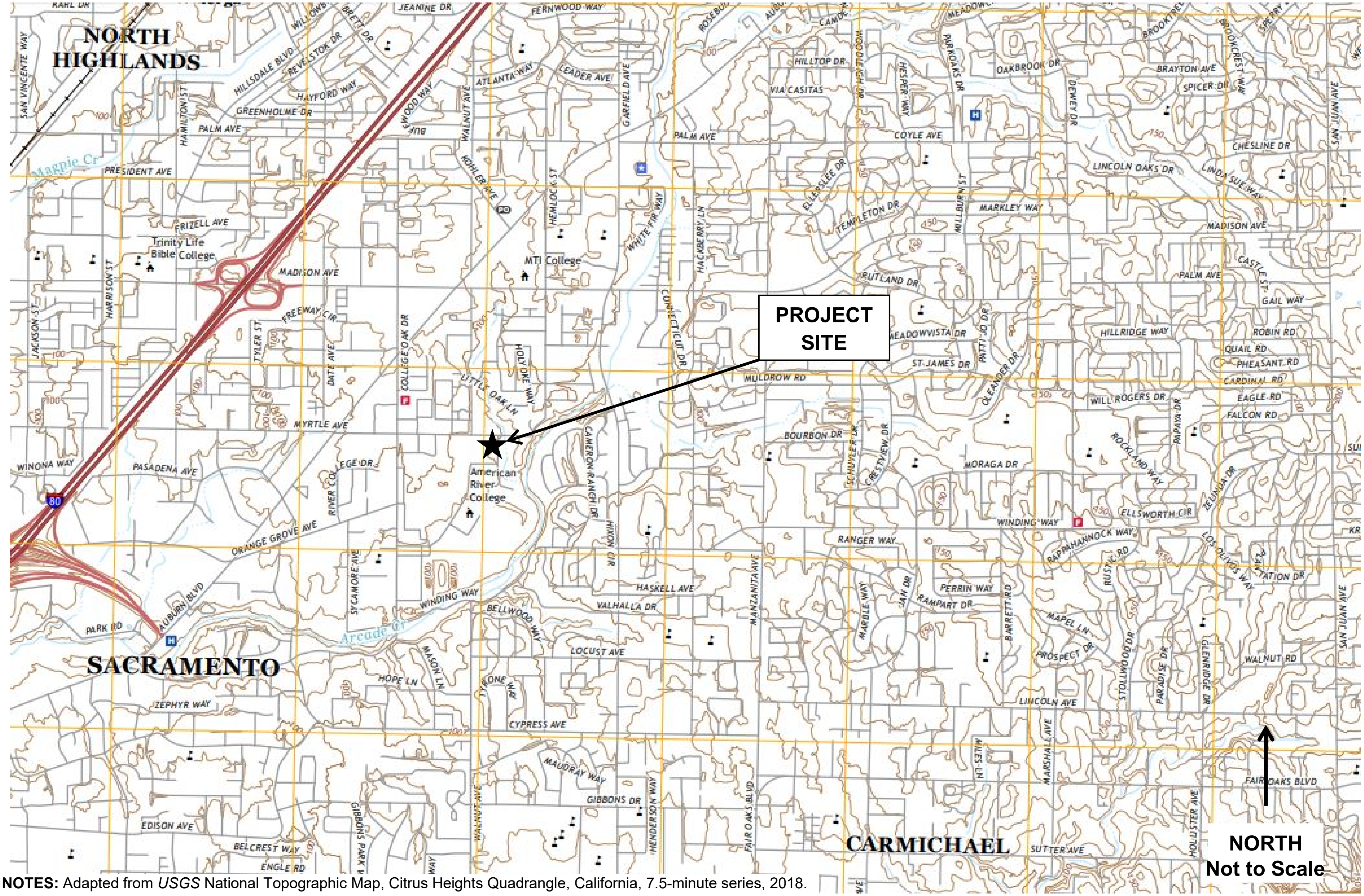
FIGURES



NOTES: Adapted from *USGS* National Topographic Map, Citrus Heights Quadrangle, California, 7.5-minute series, 2018.

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VICINITY MAP

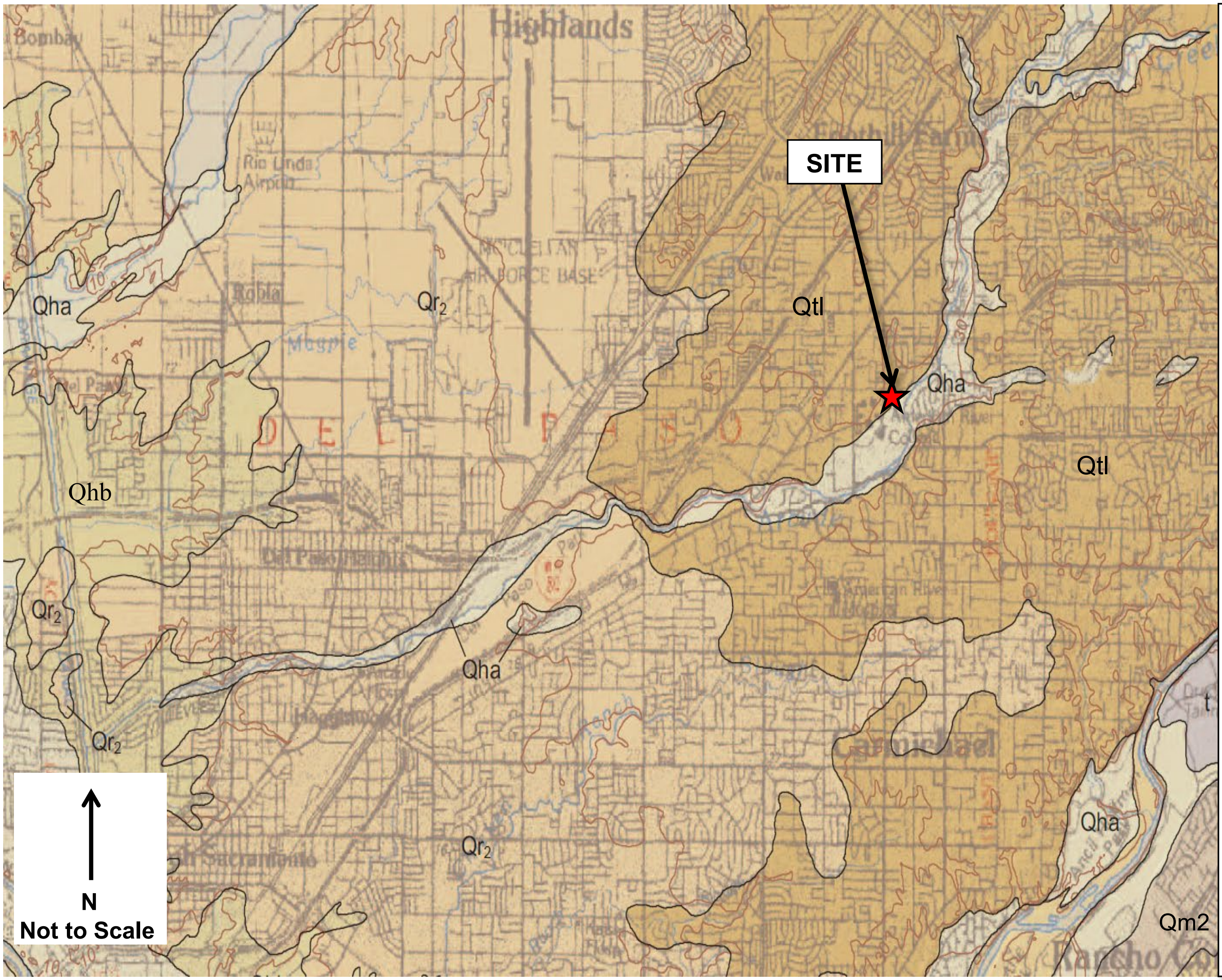
AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING

4700 College Oak Drive
Sacramento, California

FIGURE 1

Date: 01/20

MPE No. 04842-01



EXPLANATION

Surficial Deposits

t -	Dredge tailings
Qha -	Holocene Alluvium, undivided
Qhb -	Holocene basin deposits
Qa -	Alluvium

Modesto Formation

Qm -	Undivided
Qm2 -	Upper member, undivided alluvium
Qm2b –	Upper member, fine-grained
Qm1 –	Lower member, undivided alluvium
Qm1b -	Lower member, fine-grained

Riverbank Formation

Qr -	Undivided
Qr3 -	Upper unit
Qr2 -	Middle unit
Qr1 -	Lower unit

Turlock Lake Formation

Qtl -	Turlock Lake Formation

GEOLOGIC MAP SYMBOLS

- Geologic contact
- Fault
- Strike and dip of bedding

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

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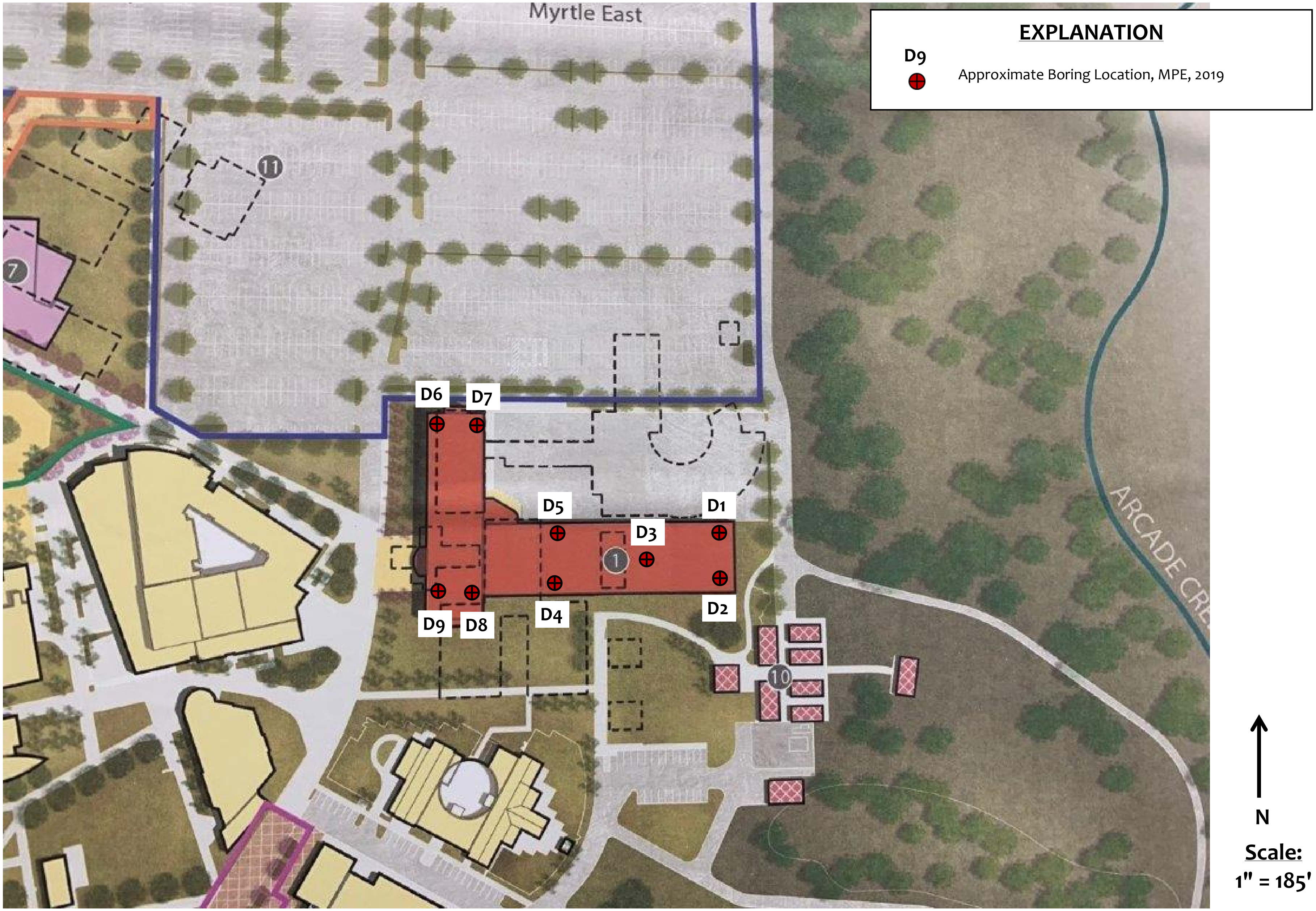
REGIONAL GEOLOGIC MAP

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE 2
Date: 01/20
MPE No. 04842-01



MID PACIFIC ENGINEERING, INC.

SITE PLAN

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE 3

Date: 01/20

MPE No. 04842-01

Project: Los Rios ARC Tech ED FPP Building Project Location: 4700 College Oak Drive, Sacramento CA MPE Number: 04842-01	LOG OF SOIL BORING D1 Sheet 1 of 2

Date(s) Drilled: 12/23/2019	Logged By: PJP	Checked By:
Drilling Method: Solid Flight Augers	Drilling Contractor: Hillside Geotechnical Drilling	Total Depth of Drill Hole: 51½ Feet
Drill Rig Type: Mobile B-24 Track	Diameter(s) of Hole, inches: 4½ inches	Approx. Surface Elevation, ft MSL:
Groundwater Depth [Elevation], feet: 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			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨	Brown, moist, silty clay (CL)						
	5	▥	Brown, orange, black mottling, slightly moist, very dense, fine sandy silt (ML)	▧	D1-2	54			
	10	▥	no mottling, moist, medium dense	▧	D1-3	26			
	15	▥	orangish brown, medium dense, no sand	▧	D1-4	20			
	20	▥	medium dense	▧	D1-5	27			
	25	▥	Orangish, brown, slightly moist, very dense, fine, sandy silt (ML) cemented	▧	D1-6	50/6"			

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D1

Sheet 2 of 2

Date(s) Drilled 12/23/2019

Logged By PJP

Checked By

Drilling Method Solid Flight Augers

Drilling Contractor Hillside Geotechnical Drilling

Total Depth of Drill Hole 51½ Feet

Drill Rig Type Mobile B-24 Track

Diameter(s) of Hole, inches 4½ inches

Approx. Surface Elevation, ft MSL

Groundwater Depth [Elevation], feet 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			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	30		Brown, moist, dense, silty, very fine to fine sand (SM)	▨	D1-7	30			
	35			▨	D1-8	44			
	40			▨	D1-9	30			-200=34.9%
			Brown, slightly moist, dense, fine to medium, poorly graded sand with silt (SP-SM)						
	45			▨	D1-10	42			-200=11.7%
			Light brown, slightly moist, medium dense, silt (ML)						
	50		moist, fine sandy silt (ML)	▨	D1-11	20			

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FIGURE 4

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D2

Sheet 1 of 1

Date(s) Drilled	12/23/2019	Logged By	PJP	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	16½ Feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	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			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, silty clay (CL)						
			Brown/ Light brown, slightly moist, very stiff, variably cemented, clayey silt (ML)	▥	D2-1	34			
	5		hard	▥	D2-2	48			
			Reddish brown, moist, medium dense, clayey fine sand (SC)						
	10			▥	D2-3	31	10.3	120	-200=14.7%
			Light Brown, moist, medium dense, very fine sandy silt (ML)						
	15			▥	D2-4	25	32.4	75	-200=69.1%
	20								
	25								

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FIGURE 5

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D3

Sheet 1 of 1

Date(s) Drilled	12/23/2019	Logged By	PJP	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	15 Feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	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			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, stiff, fine sandy clay (CL)						
				▥	D3-1	12	19.2	109	UCC 1.3 tsf
				▥	D3-2	50/6"			
	5		Dark brown/ Brown, slightly moist, hard, variably cemented, silt (ML)						
			Reddish brown, moist, medium dense, clayey fine-medium sand (SC)						
				▥	D3-3	20			
	10								
			Brown, moist, medium dense, sandy silt (ML)						
	15			▥	D3-4	23			
	20								
	25								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 6

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D4

Sheet 1 of 1

Date(s) Drilled	12/23/2019	Logged By	DR	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	15 Feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Soil Cuttings

Remarks

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨	Brown, moist, soft, silty, clay (CL)	▥	D4-1	7	16.9	104	UCC=0.9tsf PI=11% LL=27%
		▨		▥	D4-2	48			
	5	▥	Orangish brown, moist, dense, slightly clayey, fine sandy silt (ML)						
	10	▩	Orangish brown, moist, medium dense, silty fine sand (SM)	▥	D4-3	19			
		▥	Light orangish brown, moist, medium dense, very fine sandy silt (ML)	▥	D4-4	25			
	15								
	20								
	25								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 7

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D5

Sheet 1 of 1

Date(s) Drilled	12/23/2019	Logged By	DR	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	16½ Feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Soil Cuttings
Remarks					

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨	Brown, moist, stiff, silty, clay (CL)						
		▨		▥	D5-1	9	19.0	108	UCC 1.6 tsf
	5	▨		▥	D5-2	16	16.2	111	
		▥	Orangish brown, moist, medium dense, slightly clayey, fine sandy silt (ML)						
	10	▥	Light orangish brown, orange and black mottling, very moist, very stiff, clayey silt (ML)	▥	D5-3	31			
	15	▥	Light orangish brown, white mottling, moist, very dense, very fine sandy silt (ML)	▥	D5-4	50/5"			
	20								
	25								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 8

Project: American River College Technical Education Building Project Location: 4700 College Oak Drive, Sacramento CA MPE Number: 04842-01	LOG OF SOIL BORING D6 Sheet 1 of 2

Date(s) Drilled	12/24/2019	Logged By	DR	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	51½ Feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Cuttings
Remarks	SPT				

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨	Dark brown, moist ,silty clay (CL)						
		▥	Light orangish brown, black mottling, slightly moist, dense, very fine sandy silt (ML)	▨	D6-1	47			
	5		medium dense, moist	▨	D6-2	20			-200=78.4%
	10			▨	D6-3	36			
	15	▩	Light brown, white veins, moist, dense, silty fine sand (SM)	▨	D6-4	15			-200=45.7% PI=N/P
	20	▩	Brown, light brown, moist, medium dense, poorly graded fine sand with silt (SP-SM)	▨	D6-5	27			-200=8.8%
		▩	Light brown, moist, dense, silty, very fine sand (SM)						
	25			▨	D6-6	32			

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D6

Sheet 2 of 2

Date(s) Drilled 12/24/2019	Logged By DR	Checked By
Drilling Method Solid Flight Augers	Drilling Contractor Hillside Geotechnical Drilling	Total Depth of Drill Hole 51½ Feet
Drill Rig Type Mobile B-24 Track	Diameter(s) of Hole, inches 4½ Inches	Approx. Surface Elevation, ft MSL
Groundwater Depth [Elevation], feet 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
				SAMPLE DATA			TEST DATA		
			Light brown, moist, medium dense, very fine sandy silt (ML)						
	30			▨	D6-7	24			
	35		light brown, orangish brown, red veins, dense	▨	D6-8	35			
	40		Light brown, slightly moist, medium dense, silty fine sand (SM)	▨	D6-9	30			-200=38.3%
	45		Light Brown, red mottling, moist, very hard, clayey silt (ML)	▨	D6-10	64			
	50			▨	D6-11	63			

Project: American River College Technical Education Building Project Location: 4700 College Oak Drive, Sacramento CA MPE Number: 04842-01	LOG OF SOIL BORING D7 Sheet 1 of 1

Date(s) Drilled	12/24/2019	Logged By	DR	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	16½ Feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Cuttings
Remarks					

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▥	Orangish brown, black mottling, moist, hard, clayey silt (ML)						
		▥		▥	D7-1	37			
	5	▥	very stiff	▥	D7-2	19	36.2	81	
	10	▥	medium dense, fine sandy silt (ML)	▥	D7-3	32			
	15	▥	Light orangish brown, white mottling, moist, medium dense, very fine sandy silt (ML)	▥	D7-4	31			
		▩	Brown, moist, medium dense, silty fine sand (SM)						
	20								
	25								

Project: American River College Technical Education Building

Project Location: 4700 College Oak Drive, Sacramento CA

MPE Number: 04842-01

LOG OF SOIL BORING D8

Sheet 1 of 1

Date(s) Drilled	12/24/2019	Logged By	DR	Checked By	
Drilling Method	Solid Flight Augers	Drilling Contractor	Hillside Geotechnical Drilling	Total Depth of Drill Hole	15 feet
Drill Rig Type	Mobile B-24 Track	Diameter(s) of Hole, inches	4½ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Soil Cuttings
Remarks					

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨	Brown, moist, stiff, silty, clay (CL)	▥	D8-1	12	14.6	115	UCC 1.0 tsf
		▩	Brown, moist, medium dense, silty fine sand (SM)	▥	D8-2	23	11.0	125	
	5								
		▥	Brown, moist, medium dense, fine sandy silt (ML)						
				▥	D8-3	31			
	10								
				▥	D8-4	29			
	15								
	20								
	25								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 11

Project: American River College Technical Education Building Project Location: 4700 College Oak Drive, Sacramento CA MPE Number: 04842-01	LOG OF SOIL BORING D9 Sheet 1 of 1

Date(s) Drilled 12/24/2019	Logged By PJP	Checked By
Drilling Method Hand Auger	Drilling Contractor	Total Depth of Drill Hole 3½ feet
Drill Rig Type	Diameter(s) of Hole, inches 3 Inches	Approx. Surface Elevation, ft MSL
Groundwater Depth [Elevation], feet Not Encountered	Sampling Method(s)	Drill Hole Backfill Cuttings
Remarks		

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨	Brown, moist, slightly clayey, fine sandy silt (ML-FILL)						
		▨	Auger refusal on a subsurface structure (possible pipe)						
	5								
	10								
	15								
	20								
	25								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 12

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	▥	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

☐ = Hand Driven Sample

▧ =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

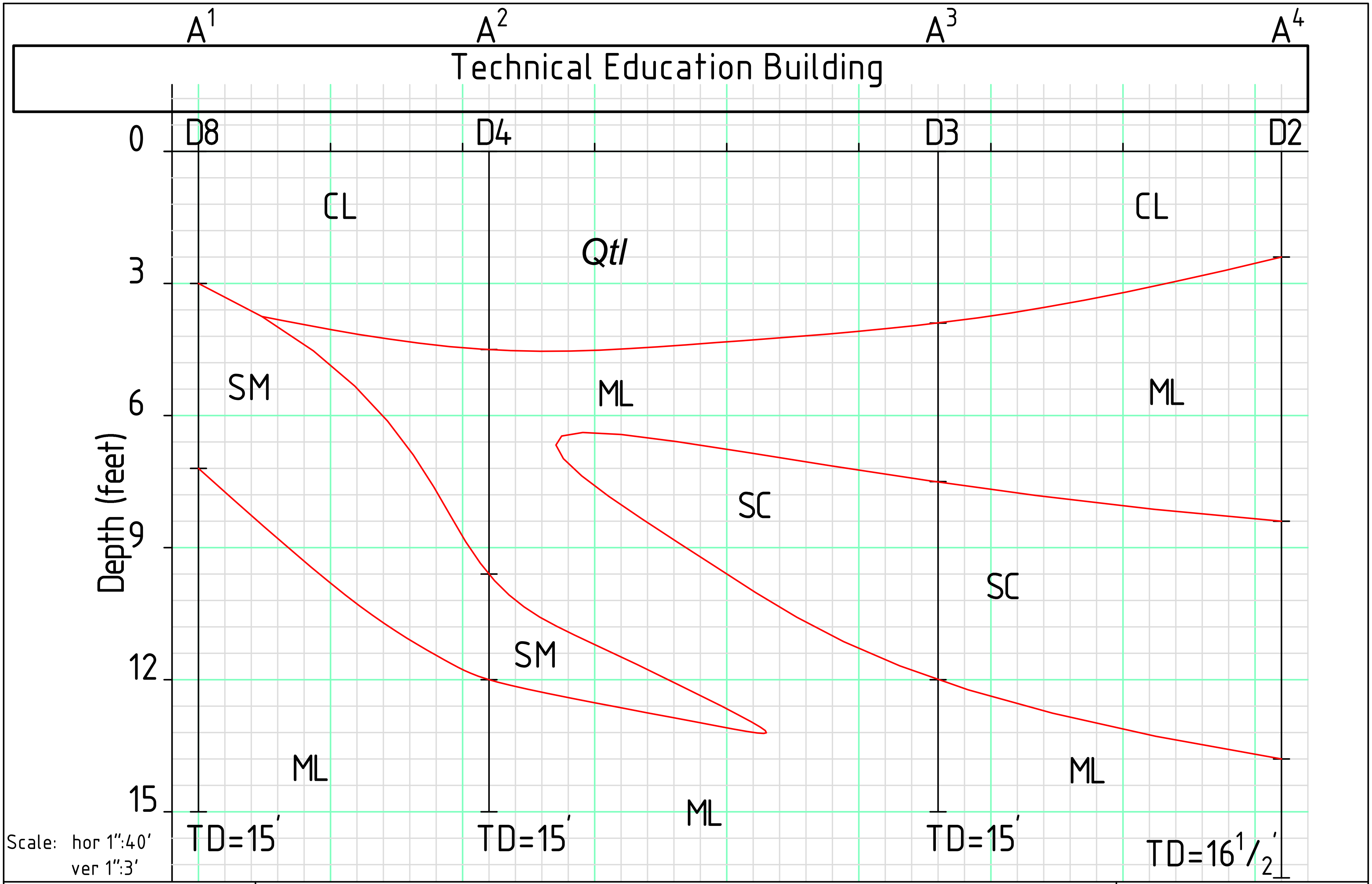
GRAIN SIZE CLASSIFICATION

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
Mid Pacific Engineering, Inc.

UNIFIED SOIL CLASSIFICATION SYSTEM
AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 COLLEGE OAKS DRIVE
SACRAMENTO, CALIFORNIA

FIGURE 13
Date: 01/20
MPE No. 04842-01



MID PACIFIC ENGINEERING, INC.

GEOLOGIC CROSS SECTION $A^1 – A^4$

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING

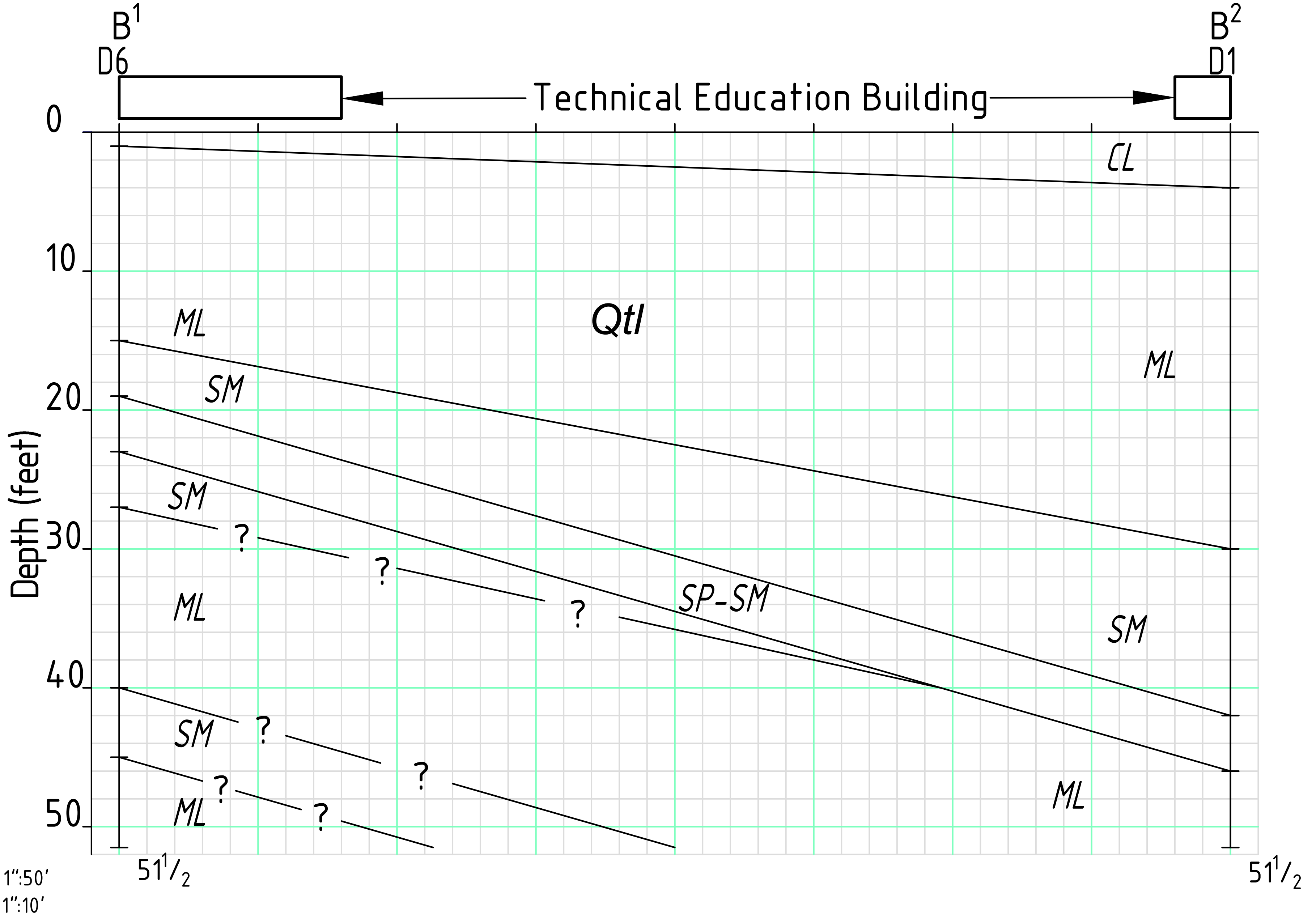
4700 College Oak Drive

Sacramento, California

FIGURE 14

Date: 01/20

MPE No. 04842-01



MID PACIFIC ENGINEERING, INC.

GEOLOGIC CROSS SECTION B^1 – B^2

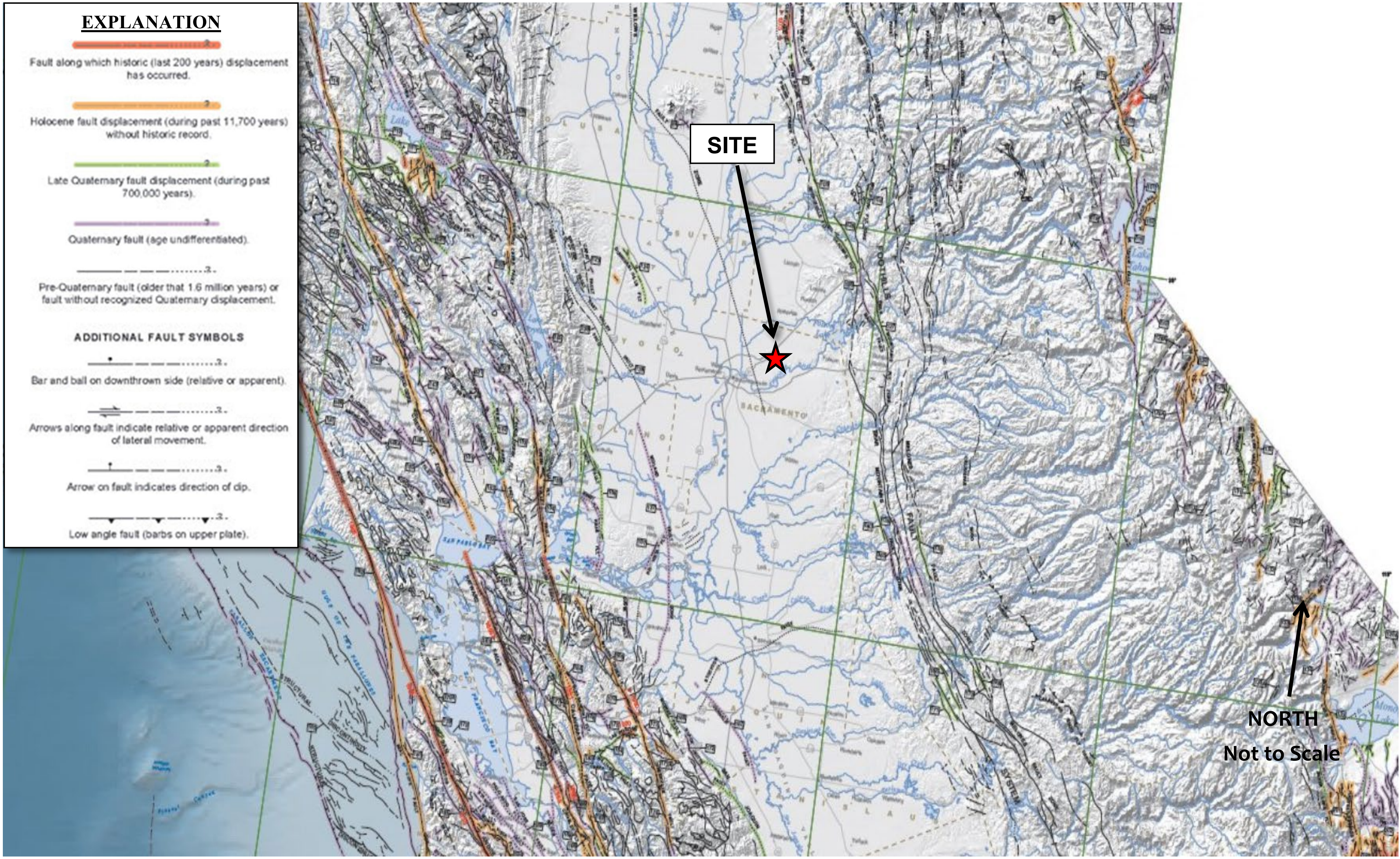
AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING

4700 College Oak Drive
Sacramento, California

FIGURE 15

Date: 01/20

MPE No. 04842-01



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

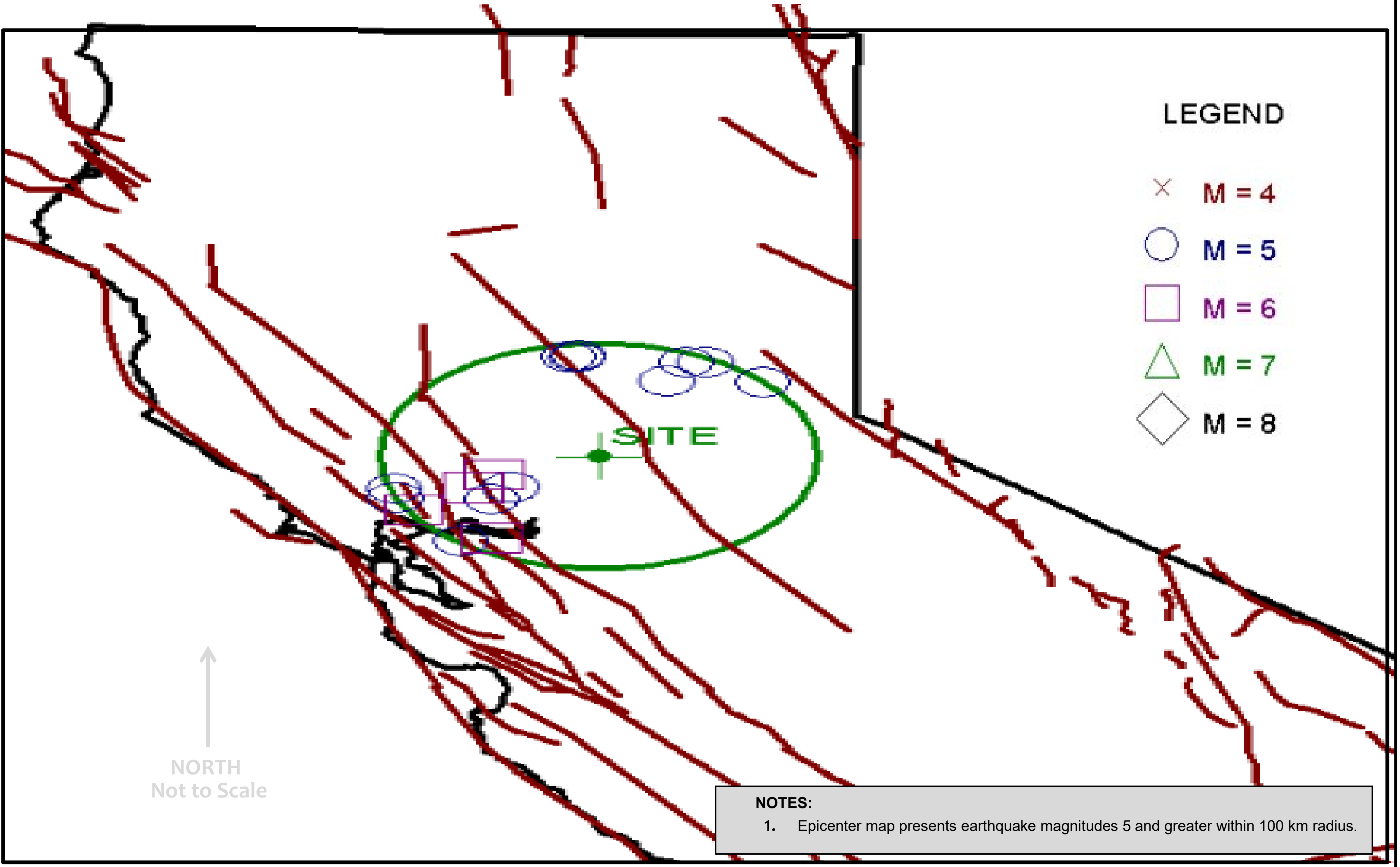
AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING

4700 College Oak Drive
Sacramento, California

FIGURE 16

Date: 01/20

MPE No. 04842-01



NOTES:

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

MPE
MID PACIFIC ENGINEERING, INC.

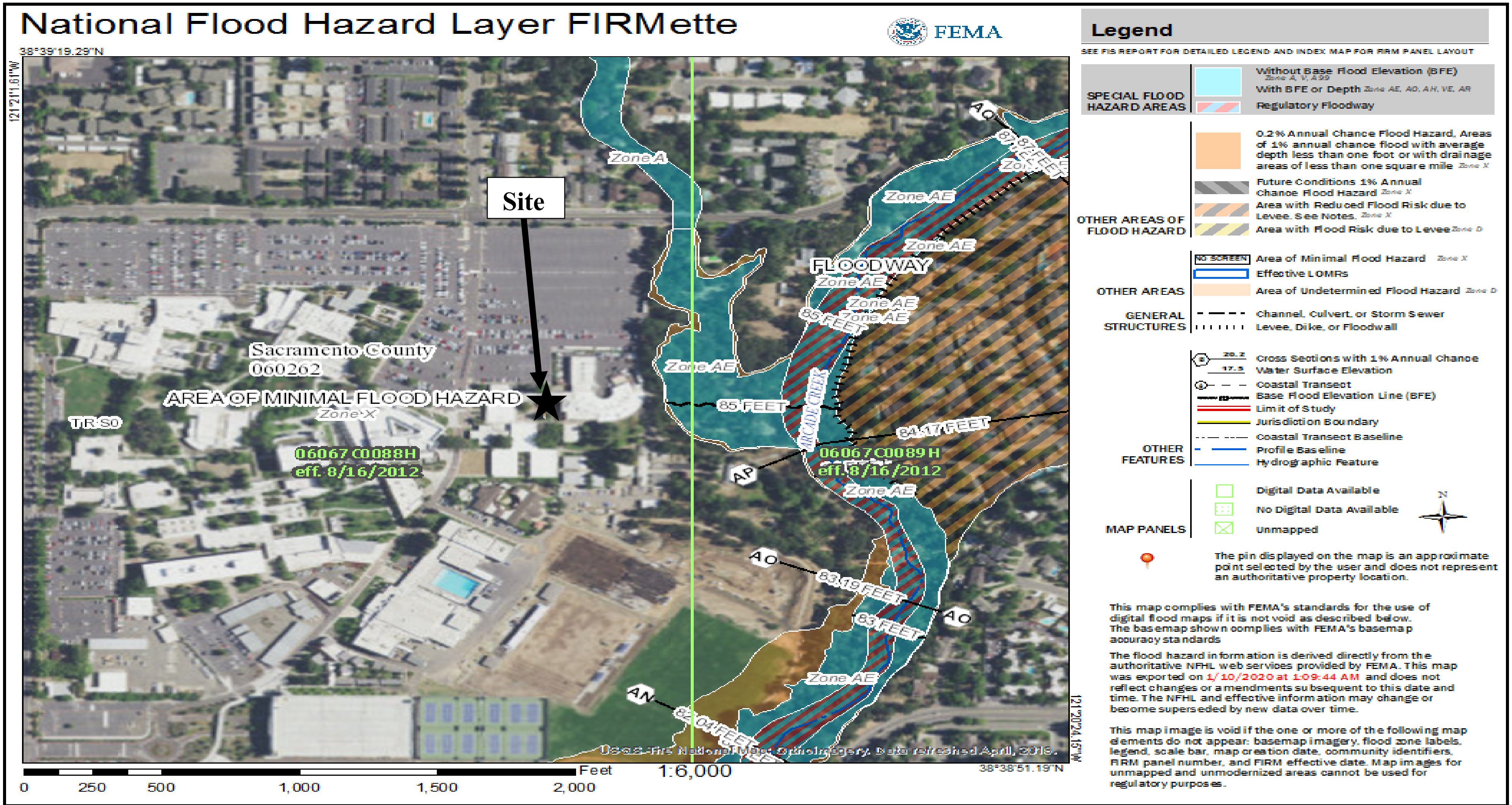
EARTHQUAKE EPICENTER MAP

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE 17

Date: 01/20

MPE No. 04842-01



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

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

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 1/10/2020 at 1:09:44 AM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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

<u>FLOOD HAZARDS</u>

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

NOT TO SCALE

Adapted from: Federal Emergency Management Agency (FEMA), Flood Insurance Rate Map (FIRM), Map Numbers 06067C0088H 06067C0089H, dated October 16, 2009

MPE
MID PACIFIC ENGINEERING, INC.

FEMA FLOOD MAP

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE 18
Date: 01/20
MPE No. 04842-01

APPENDICES

APPENDIX A

APPENDIX A

A. GENERAL INFORMATION

The performance of a Geotechnical Engineering Investigation and Geologic Hazards Report for the proposed Educational Science Building project to be constructed at the existing American River College campus located at 4700 College Oak Drive in Sacramento, California, was authorized by Dan Cox with the Los Rios Facilities Management on December 3, 2019, whose mailing address is 3753 Bradview Drive, Sacramento, California 95827; telephone (916) 826-9201.

B. FIELD EXPLORATION

On December 23 and 24, 2019, nine soil borings were drilled at the approximate locations indicated on Figure 3, utilizing a B-24 Mobile, track-mounted drill rig equipped with 4½-inch diameter, solid flight augers. The borings were drilled to maximum depths of approximately 3½ to 51½ feet below existing site grades.

At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D. Modified California sampler (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 sampler 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 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. Soil samples were taken to our laboratory for additional classification (ASTM D2488) and selection of samples for testing.

The Logs of Soil Borings, Figures 4 through 12, contain descriptions of the soils encountered in each boring. A Boring Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 13.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937) and natural moisture content (ASTM D2216), percent passing the 200 sieve (ASTM D1140), and unconfined compression strength (ASTM D2166). The results of these tests are included on the boring logs at the depth each sample was obtained.

Two bulk samples of the near-surface soils were subjected to an Expansion Index testing (ASTM D4829). The results of these tests are presented on Figures A1 and A2.

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 A3.

One sample of soil was subjected to Atterberg limits (ASTM D4318) tests. The results of this test are presented on Figure A4.

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.

EXPANSION INDEX TEST RESULTS
(ASTM D4829-03)
(UBC 18-2)

Material Description:	Dark brown, moist, silty clay (CL)
Location:	D2 (0 to 2 feet)

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
D2	11.3	24.6	102	63

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

MPE

EXPANSION INDEX TEST RESULTS
AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE A1
Date: 01/20
MPE No. 04842-01

EXPANSION INDEX TEST RESULTS
(ASTM D4829-03)
(UBC 18-2)

Material Description:	Dark brown, moist, soft, silty clay (CL)
Location:	D4 (0 to 3 feet)

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
D4	10.3	21.1	107	48

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

MPE

EXPANSION INDEX TEST RESULTS
AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE A2
Date: 01/20
MPE No. 04842-01

RESISTANCE VALUE TEST RESULTS
(California Test 301)

Material Description: Dark brown, moist, fine sandy, silty, clay (CL)

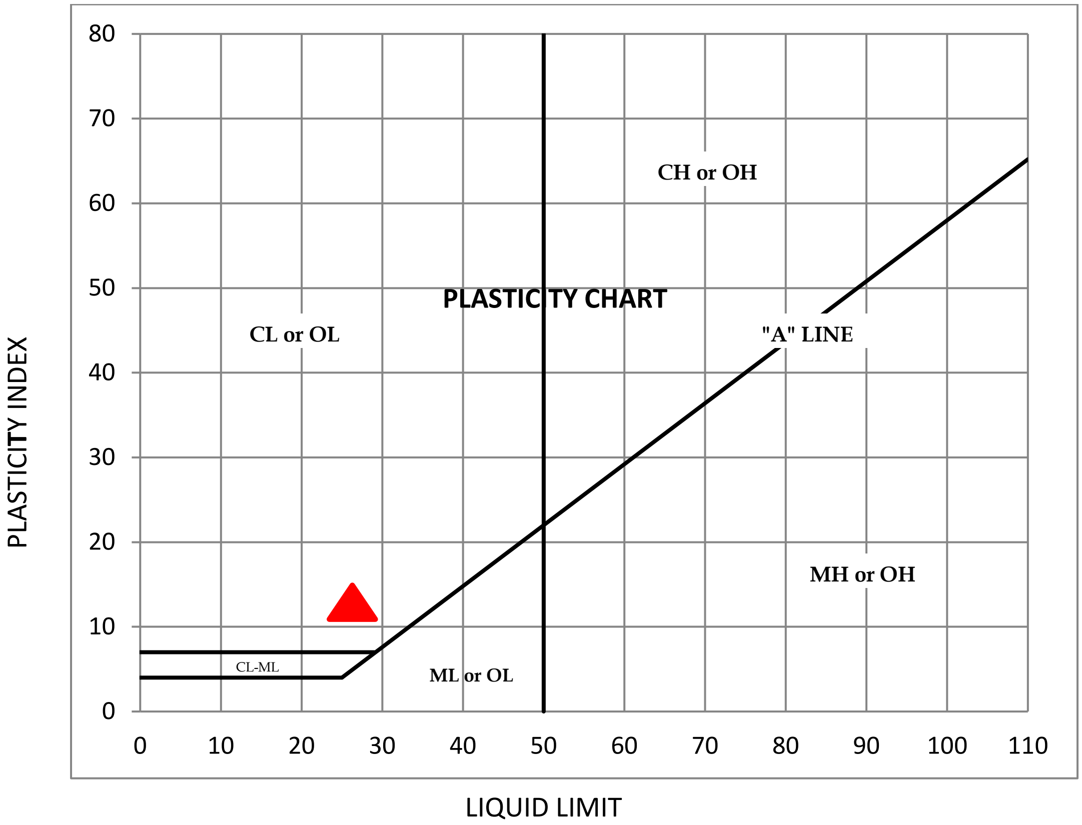
Location: Composite Sample D3(0-3’) & D6 (0-1’)

Specimen No.	Dry Unit Weight (pcf)	Moisture at Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (psi)	R-Value
R9	108.4	18.2	242	130	17
R1	110.0	17.1	657	143	18
R12	109.4	18.1	573	87	16
R10	109.4	18.2	463	48	18

Resistance-value @ 300 psi = 18

ATTERBERG LIMITS
(ASTM D4318)

Symbol	Sample & Depth (ft)	Natural Moisture Content (%)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
▲	D4 (1½–4')	27.3	27	16	11



ATTERBERG LIMITS TEST RESULTS

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING
4700 College Oak Drive
Sacramento, California

FIGURE A4
Date: 01/20
MPE No. 04842-01

APPENDIX B

APPENDIX B

GUIDE EARTHWORK SPECIFICATIONS

AMERICAN RIVER COLLEGE TECHNICAL EDUCATION BUILDING

4700 College Oak Drive
Sacramento, California
MPE No. 04842-01

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. 04842-01; 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 soils with a maximum particle size of approximately three inches (3"); free from significant quantities of rubble, rubbish and vegetation; and, having been tested and approved by the Geotechnical Engineer prior to use. The upper twelve inches (12") of the building pad and exterior flatwork subgrades shall consist of approved imported or on-site granular non-expansive soils, or Class 2 Aggregate Base. Clays soils shall not be used within the upper twelve inches (12") of the pad or exterior flatwork subgrades unless properly lime-treated.

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, existing structures, concrete slabs, foundations, asphalt concrete, utilities to be relocated or abandoned including all associated backfill, fences, trees, 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. Trees and shrubs designated to be removed shall include the entire rootball and all roots larger than one-half inch (½") in diameter. 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. All disturbed areas shall be sub-excavated in depth and lateral extent, as required by the Geotechnical Engineer, to expose firm, undisturbed native soils.

C. The upper twelve inches (12") of soil subgrades within areas of removed concrete slabs, flatwork, pavements, and trees as well as sub-excavated and disturbed areas 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.

D. 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 twelve inches (12") until the surface is free from ruts, hummocks or other uneven features which would tend to prevent uniform compaction by the selected equipment.

E. 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.

F. 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.

G. 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.

H. 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 815 or equivalent size compactor) capable of providing compaction to the full depth of soils scarification/ripping. 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.

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. Compaction shall be undertaken with a heavy, self-propelled sheepsfoot compactor (Caterpillar 815 or equivalent size 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.

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

3.5 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.

The upper twelve inches (12") of the building pad and exterior flatwork subgrades shall consist of approved imported or on-site granular non-expansive soils, or Class 2 Aggregate Base. Clays soils shall not be used within the upper twelve inches (12") of the pad or exterior flatwork subgrades unless properly lime-treated.

3.6 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.

3.7 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.

EQ FAULT OUTPUT
&
EQ SEARCH OUTPUT

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

                           DETERMINISTIC ESTIMATION OF
                     PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 04842-01
                                                     DATE: 12-30-2019

JOB NAME: Los Rio-ARC Tech ED FPP

CALCULATION NAME: NEHRP C

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

SITE COORDINATES:
   SITE LATITUDE:  38.6512
   SITE LONGITUDE:  121.3453

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
```

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                                 ---------------
                                 EQFAULT SUMMARY
                                 ---------------

                          -----------------------------
                          DETERMINISTIC SITE PARAMETERS
                          -----------------------------

-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
FOOTHILLS FAULT SYSTEM 1        |  13.5(  21.8)|   6.5    |   0.148  |  VIII
FOOTHILLS FAULT SYSTEM 2        |  24.7(  39.8)|   6.5    |   0.094  |   VII
FOOTHILLS FAULT SYSTEM 3        |  32.5(  52.3)|   6.5    |   0.077  |   VII
GREAT VALLEY 3                  |  35.5(  57.2)|   6.9    |   0.088  |   VII
GREAT VALLEY 4                  |  36.0(  57.9)|   6.6    |   0.075  |   VII
GREAT VALLEY 5                  |  38.3(  61.7)|   6.5    |   0.067  |   VI
HUNTING CREEK - BERRYESSA       |  48.0(  77.2)|   7.1    |   0.064  |   VI
CONCORD/GV (CON+GVS+GVN)        |  48.7(  78.3)|   6.7    |   0.051  |   VI
CONCORD/GV (GVN)                |  48.7(  78.3)|   6.0    |   0.036  |    V
CONCORD/GV (FLOATING)           |  48.7(  78.3)|   6.2    |   0.039  |    V
CONCORD/GV (GVS+GVN)            |  48.7(  78.3)|   6.5    |   0.046  |   VI
CONCORD/GV (CON+GVS)            |  51.0(  82.1)|   6.6    |   0.046  |   VI
CONCORD/GV (GVS)                |  51.0(  82.1)|   6.2    |   0.039  |    V
FOOTHILLS FAULT SYSTEM 4        |  53.9(  86.8)|   6.5    |   0.052  |   VI
WEST NAPA                       |  57.6(  92.7)|   6.5    |   0.040  |    V
CONCORD/GV (CON)                |  57.8(  93.1)|   6.3    |   0.035  |    V
MOUNT DIABLO (MTD)              |  59.0(  94.9)|   6.7    |   0.052  |   VI
GREENVILLE (GN)                 |  59.2(  95.2)|   6.7    |   0.043  |   VI
GREAT VALLEY 2                  |  59.4(  95.6)|   6.4    |   0.046  |   VI
*******************************************************************************
-END OF SEARCH-   19 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE FOOTHILLS FAULT SYSTEM 1         FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 13.5 MILES (21.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.1482 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: 04842-01
                                                     DATE: 12-30-2019

JOB NAME: Los Rios-ARC TECH ED FPP

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  38.6512
   SITE LONGITUDE:  121.3453

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
```

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                          -------------------------
                          EARTHQUAKE SEARCH RESULTS
                          -------------------------
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |38.4000|121.8000|04/30/1892| 0 9 0.0|  0.0| 5.50| 0.048 | VI | 30.1( 48.4)
DMG |38.5000|121.9000|04/21/1892|1743 0.0|  0.0| 6.20| 0.067 | VI | 31.7( 51.0)
DMG |38.3000|121.9000|05/19/1902|1831 0.0|  0.0| 5.50| 0.040 |  V | 38.6( 62.0)
DMG |38.4000|122.0000|04/19/1892|1050 0.0|  0.0| 6.40| 0.063 | VI | 39.4( 63.4)
T-A |39.2500|121.0000|12/01/1867| 712 0.0|  0.0| 5.00| 0.027 |  V | 45.3( 72.9)
DMG |38.0000|121.9000|05/19/1889|1110 0.0|  0.0| 6.00| 0.040 |  V | 54.1( 87.0)
USG |39.4330|121.4750|08/02/1975|2059 0.0|  5.1| 5.20| 0.026 |  V | 54.4( 87.6)
USG |39.4360|121.5230|08/01/1975|202012.0|  8.8| 5.70| 0.033 |  V | 55.0( 88.5)
USG |39.4490|121.4730|08/02/1975|202216.2|  4.1| 5.20| 0.026 |  V | 55.5( 89.3)
DMG |39.4000|120.9000|03/03/1909|12 0 0.0|  0.0| 5.00| 0.023 | IV | 56.9( 91.6)
DMG |39.4000|120.8000|06/23/1909| 724 0.0|  0.0| 5.50| 0.028 |  V | 59.4( 95.6)
GSB |38.2152|122.3123|08/24/2014|102044.1| 11.1| 6.02| 0.037 |  V | 60.3( 97.1)
DMG |37.9700|122.0500|10/24/1955| 41044.0|  0.0| 5.40| 0.026 |  V | 60.6( 97.5)
GSB |38.3790|122.4130|09/03/2000|083630.1| 10.0| 5.00| 0.021 | IV | 60.7( 97.6)
UNR |39.2450|120.4960|11/28/1980|182112.4|  1.5| 5.20| 0.024 | IV | 61.3( 98.7)
DMG |38.3000|122.4000|10/12/1891| 628 0.0|  0.0| 5.50| 0.027 |  V | 61.9( 99.7)

*******************************************************************************
-END OF SEARCH-   16 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 30.1 MILES (48.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.4
LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.067 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  0.147
  b-value=  0.291
  beta-value=  0.670

  ------------------------------------
  TABLE OF MAGNITUDES AND EXCEEDANCES:
  ------------------------------------
  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       16        |   0.07306
     4.5     |       16        |   0.07306
     5.0     |       16        |   0.07306
     5.5     |        9        |   0.04110
     6.0     |        4        |   0.01826
```

LIQSVS ANALYSES
OUTPUT

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : Technical Education Building **SPT Name: D1**

Location : Americn River College, Sacramento

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	120.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	120.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.10
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.24 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.00	8	55.00	127.00	4.00	Yes
5.00	50	78.40	122.00	6.00	Yes
10.00	26	78.40	122.00	5.00	Yes
15.00	20	78.40	122.00	5.00	Yes
20.00	27	78.40	122.00	5.00	Yes
25.00	50	78.40	122.00	5.00	Yes
30.00	30	34.90	135.00	5.00	Yes
35.00	44	34.90	135.00	5.00	Yes
40.00	30	34.90	135.00	1.50	Yes
45.00	42	11.70	135.00	5.00	Yes
50.00	20	55.00	122.00	5.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.00	10	0.01	0.04	0.24	0.13	33459.21	0.00	0.00	10.85	0.01	4.00	0.006
5.00	55	0.05	0.21	0.84	0.14	12985.93	0.00	0.00	10.85	0.00	6.00	0.002
10.00	27	0.09	0.41	0.95	0.15	8588.48	0.00	0.00	10.85	0.00	5.00	0.006
15.00	18	0.14	0.61	1.05	0.16	6739.32	0.00	0.00	10.85	0.01	5.00	0.011
20.00	24	0.18	0.82	1.31	0.17	5673.23	0.00	0.00	10.85	0.01	5.00	0.009
25.00	40	0.22	1.02	1.70	0.18	4963.54	0.00	0.00	10.85	0.00	5.00	0.005
30.00	22	0.27	1.25	1.57	0.20	4403.24	0.00	0.00	10.85	0.01	5.00	0.012
35.00	29	0.31	1.48	1.86	0.21	3985.02	0.00	0.00	10.85	0.01	5.00	0.009
40.00	18	0.34	1.70	1.75	0.22	3658.30	0.00	0.00	10.85	0.01	1.50	0.005
45.00	24	0.36	1.93	1.84	0.24	3394.46	0.00	0.00	10.85	0.01	5.00	0.018
50.00	10	0.37	2.13	1.68	0.25	3195.35	0.00	0.00	10.85	0.03	5.00	0.034

Cumulative settlemetns: 0.116

Abbreviations

τ_{av}:	Average cyclic shear stress
p:	Average stress
G_{max}:	Maximum shear modulus (tsf)
α, b:	Shear strain formula variables
γ:	Average shear strain
ε_{15}:	Volumetric strain after 15 cycles
N_c:	Number of cycles
ε_{Nc}:	Volumetric strain for number of cycles N_c (%)
Δh:	Thickness of soil layer (in)
ΔS:	Settlement of soil layer (in)

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : Technical Education Building

SPT Name: D6

Location : Americn River College, Sacramento

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	120.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	120.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.10
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.24 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.00	8	55.00	127.00	1.00	Yes
2.00	47	78.40	122.00	4.00	Yes
5.00	20	78.40	122.00	5.00	Yes
10.00	36	78.40	122.00	5.00	Yes
15.00	15	45.70	132.00	4.00	Yes
19.00	27	8.80	132.00	4.00	Yes
25.00	32	45.70	132.00	4.00	Yes
30.00	24	78.40	122.00	8.00	Yes
35.00	35	78.40	122.00	5.00	Yes
40.00	30	38.30	132.00	5.00	Yes
45.00	50	78.40	122.00	6.50	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.00	10	0.01	0.04	0.24	0.13	33459.21	0.00	0.00	10.85	0.01	1.00	0.001
2.00	59	0.02	0.08	0.55	0.13	22339.75	0.00	0.00	10.85	0.00	4.00	0.001
5.00	22	0.05	0.21	0.64	0.14	12985.93	0.00	0.00	10.85	0.00	5.00	0.005
10.00	38	0.09	0.41	1.06	0.15	8588.48	0.00	0.00	10.85	0.00	5.00	0.003
15.00	13	0.14	0.63	0.98	0.16	6631.49	0.00	0.00	10.85	0.01	4.00	0.014
19.00	24	0.18	0.81	1.18	0.17	5718.26	0.00	0.00	10.85	0.01	4.00	0.012
25.00	25	0.24	1.07	1.52	0.19	4822.82	0.00	0.00	10.85	0.01	4.00	0.008
30.00	18	0.27	1.28	1.52	0.20	4344.11	0.00	0.00	10.85	0.01	8.00	0.025
35.00	23	0.31	1.48	1.75	0.21	3974.20	0.00	0.00	10.85	0.01	5.00	0.012
40.00	18	0.34	1.70	1.75	0.22	3656.14	0.00	0.00	10.85	0.01	5.00	0.017
45.00	28	0.36	1.91	2.09	0.23	3415.87	0.00	0.00	10.85	0.01	6.50	0.012

Cumulative settlemetns: 0.110

Abbreviations

τ_{av}:	Average cyclic shear stress
p:	Average stress
G_{max}:	Maximum shear modulus (tsf)
α, b:	Shear strain formula variables
γ:	Average shear strain
ε_{15}:	Volumetric strain after 15 cycles
N_c:	Number of cycles
ε_{Nc}:	Volumetric strain for number of cycles N_c (%)
Δh:	Thickness of soil layer (in)
ΔS:	Settlement of soil layer (in)

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PETRALOGIX REPORT

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January 10, 2020

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Project No. 2019-00095

SUBJECT: Vs100 Site Class Determination
American River College – Tech Ed Project
Sacramento, California

Dear Martin,

We have completed our 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 two (2) individual refraction microtremor (REMI) and Multi-Channel Analysis of Surface Wave (MASW) survey lines. The locations of our survey lines are shown on Plate 2. Plates 3 through 6 (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 C. 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:

Refraction Microtremor (REMI)

Two (2) 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.

Data Acquisition:

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 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.

Multi-Channel Analysis of Surface Waves (MASW)

Multichannel Analysis of Surface Waves (MASW) was also performed using Surfseis Version 5.3. The active method operation was chosen to evaluate the SEG-2 field files. A frequency overtone generator was used to develop a Phase Velocity-Frequency Image. Frequency ranges were allowed to span from 5 Hz to 50 Hz, with an allowed Phase Velocity window of 10 and 6,000 feet per second (ft/sec). An automatic evaluation was performed which yielded a surface wave velocity. The risk of contamination by higher modes was considered to be low, and the overall quality of input data was excellent. From this process dispersion curves were generated for both forward and reverse geometries along the line. Offset shots were taken at 70, 40, and 10 feet from the start of each line. These individual dispersion curves were combined to create a single averaged curve for subsequent dispersion value (phase velocity vs. frequency) picking and extraction. Inversion was performed on the picked/extract values in order to create a layer model for comparison and integration with other methods to obtain a best fit shear-wave approximation for the site. The model was allowed to run through the inversion process for 4 to 12 iterations, with a final model that reached a total depth of 100+ feet. All data obtained from this processing was used to assist in developing an appropriate dispersion curve and a representative layer profile model, and for calculating the Vs100 for site class designation.

FINDINGS AND CONCLUSIONS

For Line #1 the Vs100 estimations were (REMI) 1502 ft/sec, and (MASW) 1704 ft/sec, respectively. For Line #2 the Vs100 estimations were (REMI) 1151 ft/sec, and (MASW) 1832 ft/sec, respectively. From geologic review of the area it is clear that spatial variations of soil are significant due to long-term lateral variations related to local creek/fluvial processes. In our opinion, it is likely the site has been part of the local fluvial river plane, and thereby a variety of mixed and varied soil conditions may exist throughout the site.

Therefore, variations in lateral stratigraphy of soil, gravel, and clay deposits makes sense and need to be considered/reflected in MASW and REMI site testing in order to determine appropriate Vs100 classification for the site. Based on the average values for the site (REMI Average - 1327 ft/sec, and MASW Average - 1768 ft/sec) we considered this to be Site Class C.

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,

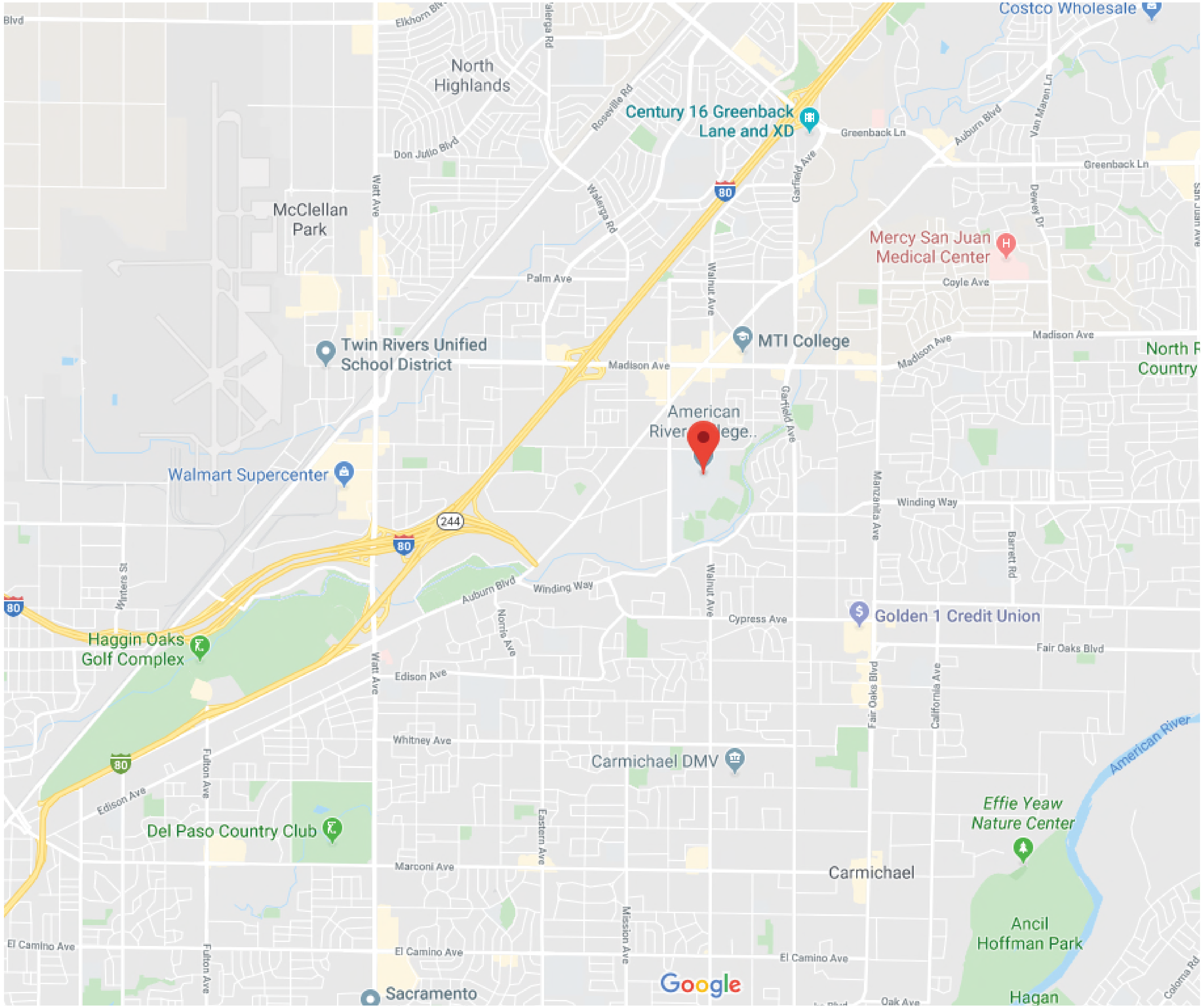
Daniel E. Kramer

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

PROFESSIONAL GEOPHYSICIST
DANIEL E.
KRAMER
Pgp 1078
STATE OF CALIFORNIA

Appendix A

ARC Tech Ed Project - Geophysical Survey
Vicinity Map



DATE: 01-10-20

JOB NUMBER: 2019-00095

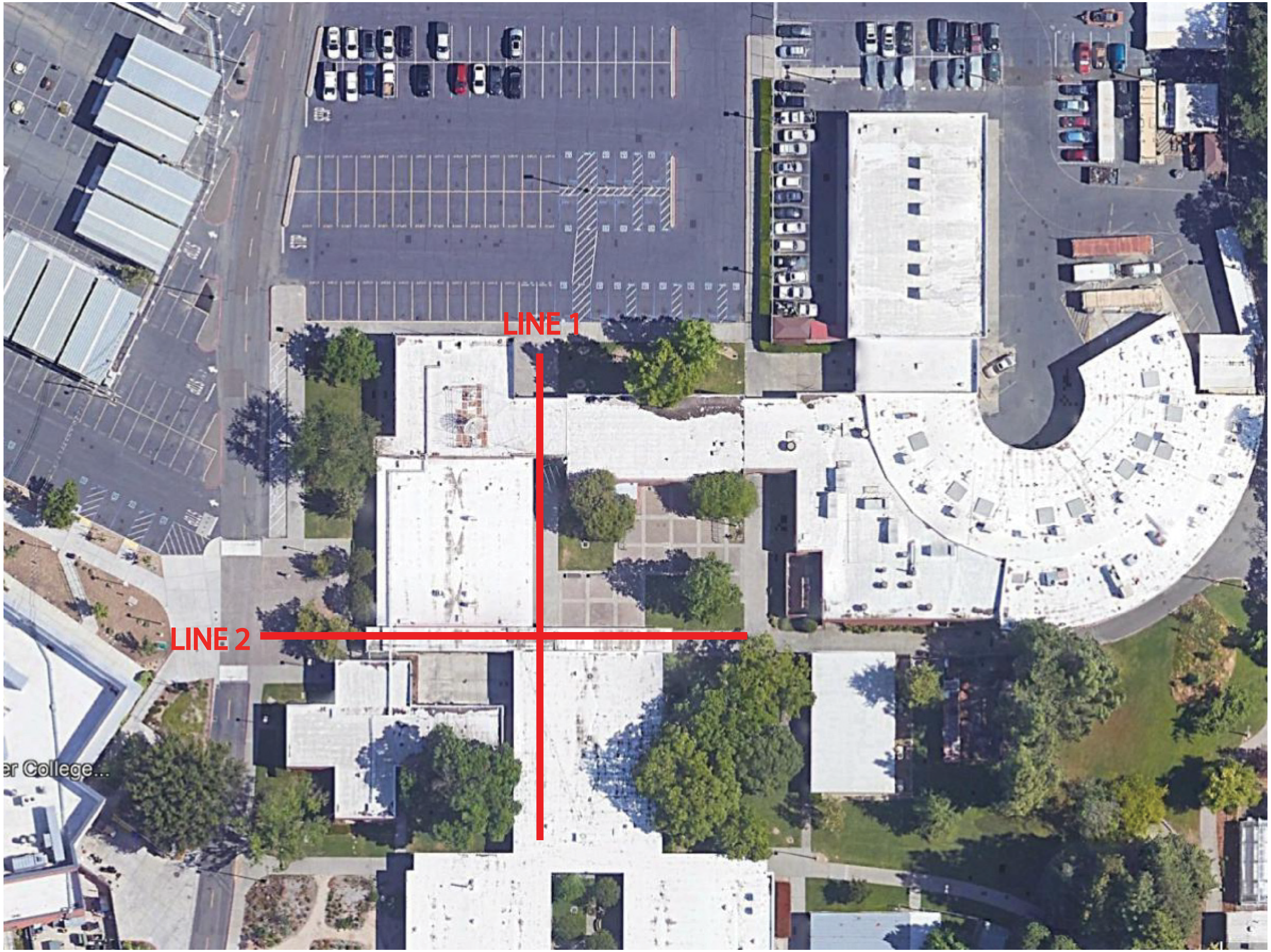
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 1

ARC Tech Ed Project - Geophysical Survey
Site Survey Map



N

LEGEND

Survey Line Locations

MASW-REMI 1D Lines 🟥

DATE: 01-10-20

JOB NUMBER: 2019-00095

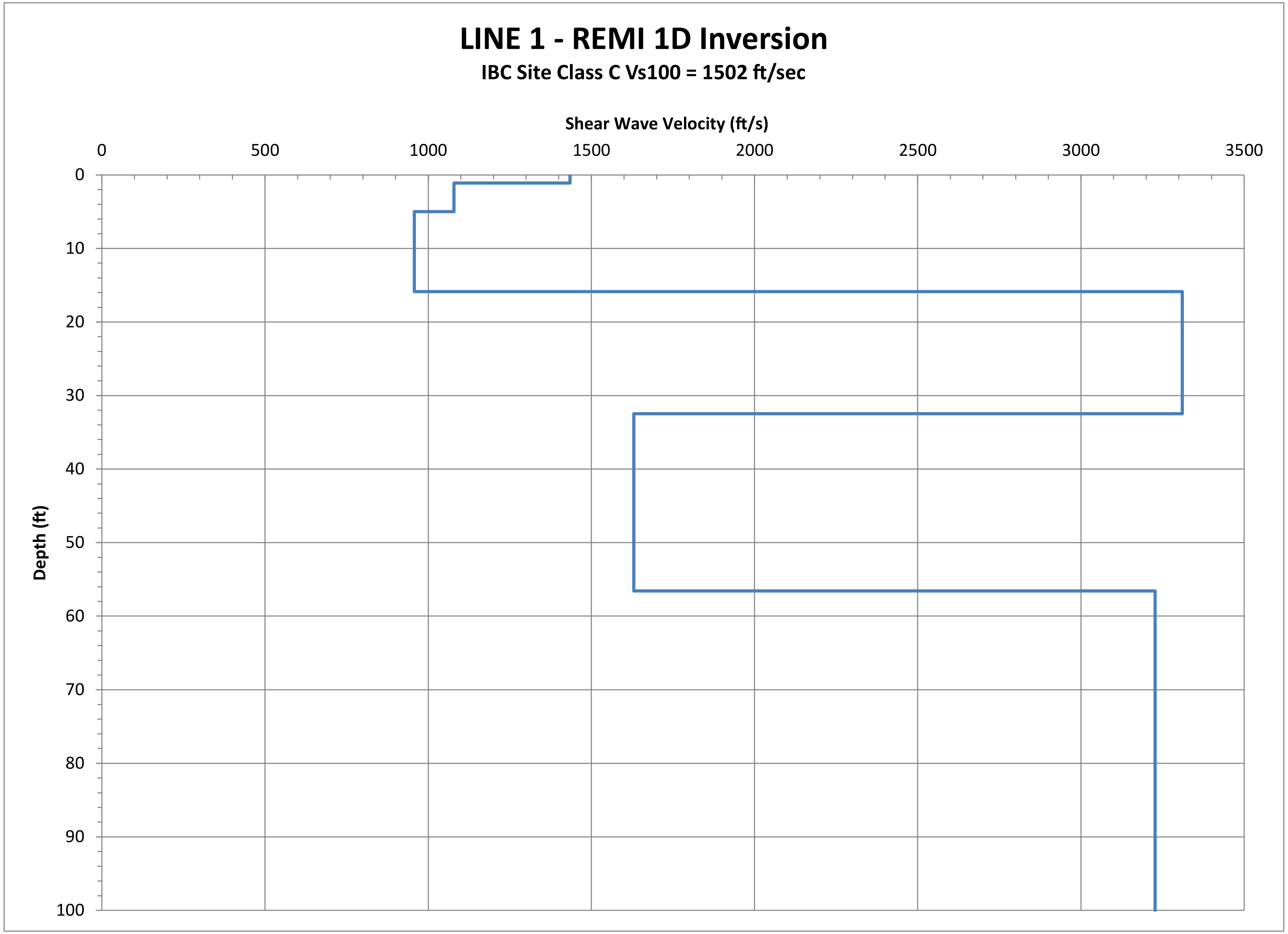
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 2

ARC Tech Ed Project - Geophysical Survey



DATE: 01-10-20

JOB NUMBER: 2019-00095

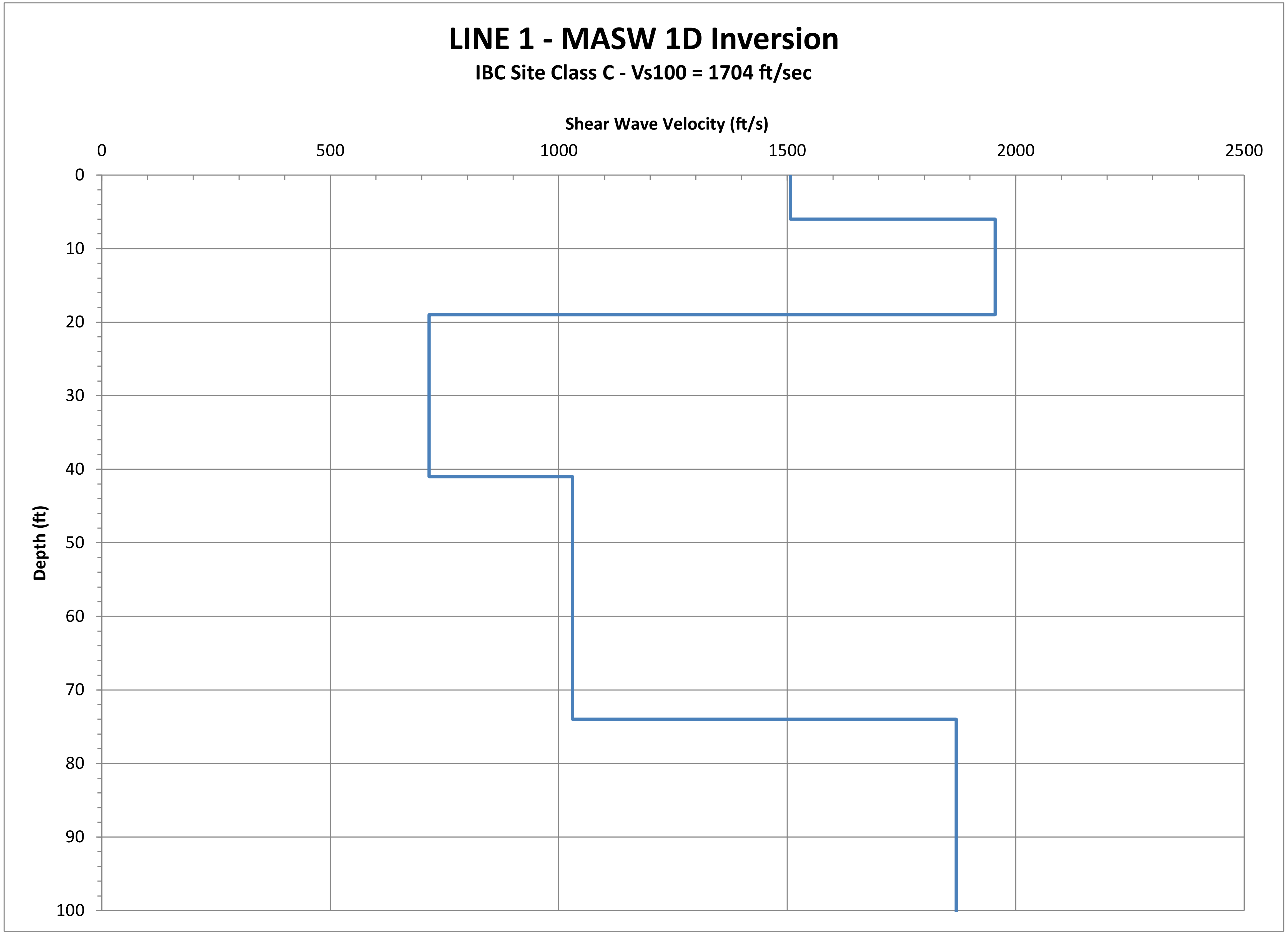
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 3

ARC Tech Ed Project - Geophysical Survey



LINE 1 - MASW 1D Inversion

IBC Site Class C - Vs100 = 1704 ft/sec

DATE: 01-10-20

JOB NUMBER: 2019-00095

SCALE: Not to Scale

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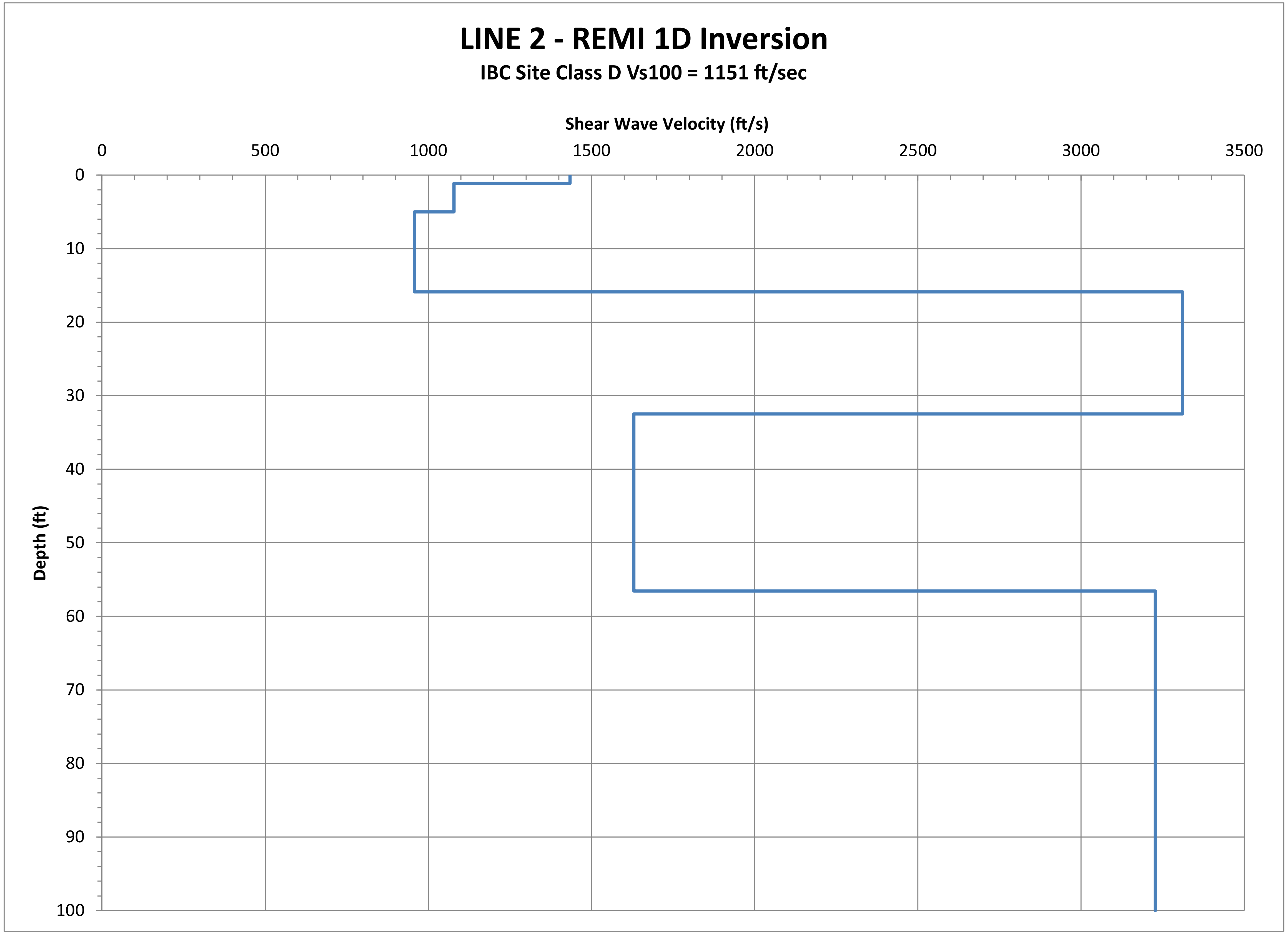
PLATE NO. 4

ARC Tech Ed Project - Geophysical Survey



DATE: 01-10-20

JOB NUMBER: 2019-00095

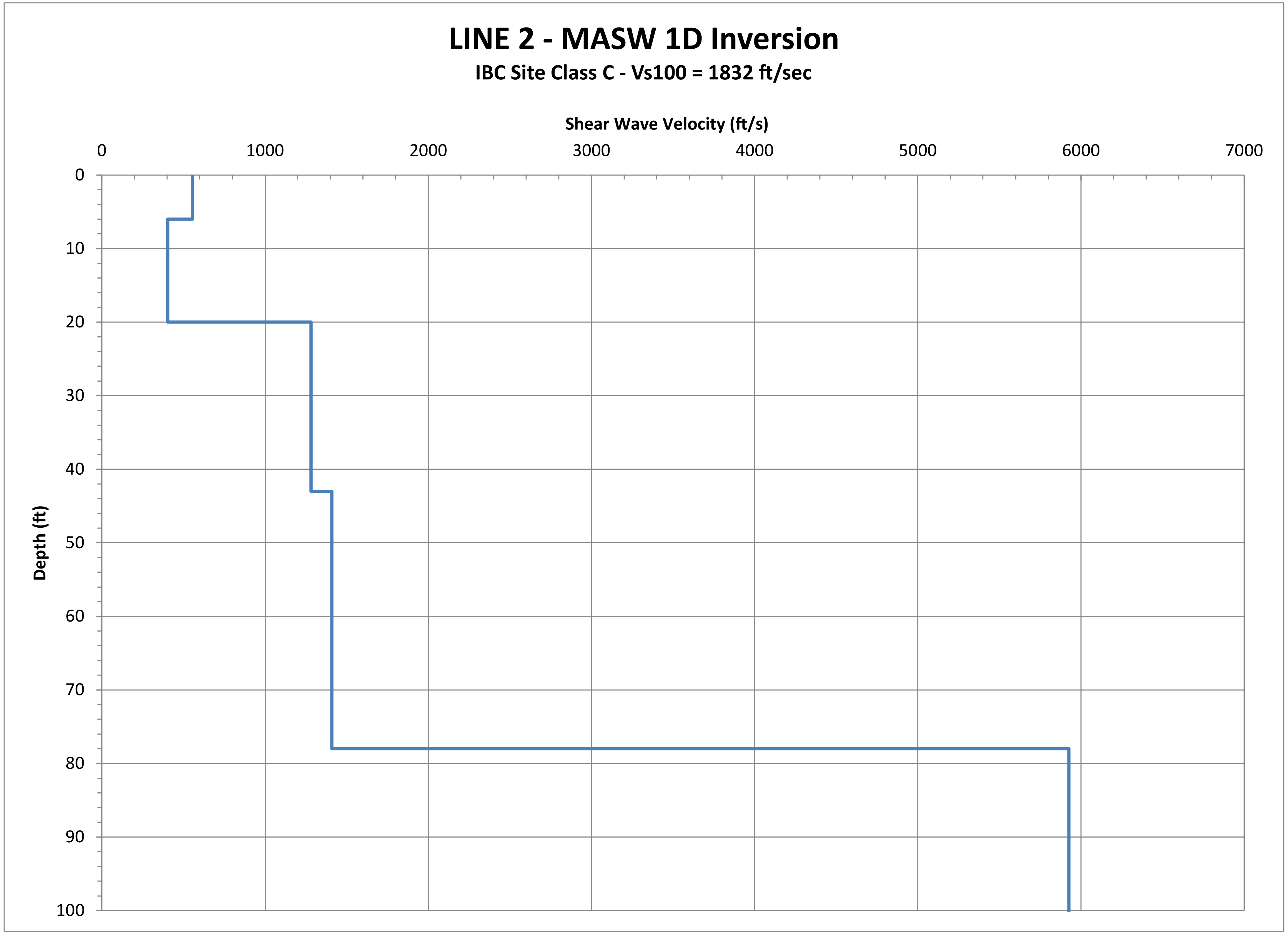
SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 5

ARC Tech Ed Project - Geophysical Survey



DATE: 01-10-20

JOB NUMBER: 2019-00095

SCALE: Not to Scale

DRAWN BY: DK

CHECKED BY: DK

PLATE NO. 6