

December 15, 2025

Addendum No. 2
To the contract documents for ELKHORN ES
For the Modular Building

NOTICE TO BIDDERS

This addendum forms a part of the contract and modifies the original bidding documents dated October 24, 2025. It is intended that all work affected by the following modifications shall conform to related provisions and general conditions of the Contract of the original drawings and specifications. Modify the following items wherever appearing in any drawings or sections of the specifications. Acknowledge receipt of Addendum No. 2 in the space provided on the Bid Form. Failure to do so may subject to disqualification.

General Items

Item No. 1 Under the basis of design 2022 California building code is called out. Is the sitework portion of the new classrooms already designed and into DSA?

Response: The site work package will be submitted to DSA on 12/18/2025.

Item No. 2 The basis of design refers to the Geotechnical Report as (Exhibit D). However, the abatement specifications are included at the end of Addendum #1 but no Geotech report. Please clarify when we can expect to receive the Geotech report for the soils.

Response: See attached geotechnical report.

Item No. 3 Are the Preconstruction Design Services of this project to be carried by the manufacturer to include; Structural, Mechanical, Electrical, Plumbing, PV systems, Fire Sprinkler, and T24 designs? If so, will the district architect (LPA), be the Architect of Record (A.O.R.) for this project?

Response: Yes LPA is the A.O.R. for this project.

Item No. 4 Will the district architect (LPA) sign and stamp the manufacturer's drawings, or will modular manufacturer be required to provide stamped drawings for DSA submittal?

Response: The modular manufacturer is responsible for stamping and signing their drawings.

Item No. 5 Please confirm that the manufacturer is responsible for PV design on the buildings.

Response: Confirmed.

Item No. 6 Will the building PV system be ground mounted? If so, is this to be included by site contractor or manufacturer?

Response: PV are to be mounted to the roof of the modular building.

Item No. 7 Does this Elkhorn classroom addition fall under WUI requirements?

Response: No.



Changes to Specifications

Item No. 1 N/A

Item No. 2 N/A

Item No. 3 N/A

Changes to Drawings

Item No. 1 N/A

Item No. 2 N/A

Item No. 3 N/A

Drawings and Documents Issued

The following drawings and documents are issued as Addendum No. 2 and are to be included in the Contract Documents.

Geo technical Report

End of Addendum No. 2

Geologic Hazards and Geotechnical Engineering Report

ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING

10505 Davis Road
Stockton, California
MPE No. 06317-03



June 6, 2025

Geologic Hazards and Geotechnical Engineering Report

ELKHORN ELEMENTARY SCHOOL

NEW CLASSROOM BUILDING AND REMODELING

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TABLE OF CONTENTS

INTRODUCTION	1
Scope of Services	1
Figures and Attachments	2
Proposed Development	3
FINDINGS	3
Site Description	3
Site History	4
GEOLOGIC SETTING	4
Regional Geology and Structure	4
Site Geology	4
Subsurface Soil Conditions	5
Groundwater	6
REGIONAL SEISMICITY	7
Faulting	7
Historic Seismicity	9
Coseismic Ground Deformation	11
Site Class	11
Seismic Code Parameters	11
Primary Seismic Hazards	13
Secondary Hazards	15
Foundation and Structural Support	20
Expansive Soils	21
Suitability of On-site Soils for Use as Fill	21
Excavation Conditions	22
Soil Corrosion Potential	22
Pavement Subgrade Qualities	23
Permanent Groundwater	24
Seasonal Water	24
Erosion and Winterization	25

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TABLE OF CONTENTS

RECOMMENDATIONS.....	25
Site Clearing.....	26
Site Preparation and Over-Excavation.....	27
Engineered Fill Construction	29
Lime-Treatment Alternative	31
Utility Trench Backfill	32
Foundation Design.....	33
Interior Floor Slab Support.....	35
Floor Slab Moisture Penetration Resistance.....	36
Drilled Cast-In-Place Concrete Piers.....	37
Exterior Flatwork	39
Site Drainage	40
Pavement Design	41
Future Services.....	43
LIMITATIONS.....	44

FIGURES

Vicinity Map	Figure 1
Regional Geologic Maps	Figures 2A and 2B
Site Investigation Map.....	Figure 3
Logs of Soil Borings	Figures 4 through 10
Unified Soil Classification System	Figure 11
Logs of CPT Soundings	Figures 12 and 13
Soil Behavior Charts.....	Figure 14
Measured Shear Wave Velocities.....	Figure 15
Engineering Geologic Cross-Section	Figures 16 through 18
Regional Fault Map	Figure 19
Earthquake Epicenter Map.....	Figure 20
FEMA Flood Map	Figure 21

APPENDIX A – General Project Information, Field and Laboratory Test Results

APPENDIX B – Guide Earthwork Specifications

APPENDIX C – Output files from the EQSEARCH program

APPENDIX D – Output of GeoSuite Liquefaction Analyses

APPENDIX E – References

Geologic Hazards and Geotechnical Engineering Report

**ELKHORN ELEMENTARY SCHOOL
NEW CLASSROOM BUILDING AND REMODELING**

10505 Davis Road
Stockton, California
MPE No. 06317-03
June 6, 2025

INTRODUCTION

We have completed a Geologic Hazards and Geotechnical Engineering Report for the proposed Elkhorn Elementary School New Classroom Building and Remodeling project to be constructed within the existing Elkhorn Elementary School campus located at 10505 Davis Road in Stockton, 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 plan entitled *Elkhorn School, Option E*, prepared by LPA and provided via e-mail on October 29, 2024;
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 of seven (7) exploratory soil borings to the approximate maximum depth of 51½ feet below existing ground surface (bgs); and,
 - advancement of two (2) Cone Penetration Test (CPT) soundings to the approximate maximum depth of 51 feet bgs and measurement of shear wave velocities in sounding.
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	Soil Behavior Charts
2A, 2B	Regional Geologic Maps	15	Measured Shear Wave Velocities
3	Site Plan	16-18	Engineering-Geological Cross-Sections
4-10	Logs of Soil Borings	19	Regional Fault Map
11	Unified Soil Classification System	20	Regional Seismicity Map
12-13	Logs of CPT Soundings	21	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 EQSEARCH program.
- Appendix D - Output of Liquefaction and Seismic Settlement Analyses.
- Appendix E - A list of references cited.

PROPOSED DEVELOPMENT

Based on our review of the available information, we understand the proposed project will include the construction of a new classroom building and a new shade structure, a reconfiguration of the existing administration/faculty building, a modernization of the existing classroom building, and rehabilitation of the existing pavements. The proposed new classroom building is anticipated to be a two-story, wood- or steel-framed structure, with a footprint of approximately 9,522 square feet, supported on the conventional foundation system, with a concrete slab-on-grade ground floor. New foundations may be constructed within the buildings proposed for the modernization and reconfiguration.

Associated development is anticipated to include construction of exterior flatwork, underground utilities, and typical 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 and fills of up to two feet in depth will be constructed to achieve design elevations.

FINDINGS

SITE DESCRIPTION

The project site is within the existing Elkhorn Elementary School campus located at 10505 Davis Road in Stockton, California. The approximate location of the project is north latitude 38.0506° and west longitude 121.3344°.

The site is generally bounded to the north by a church, beyond which lies agricultural land, to the east by Davis Road, beyond which lies single family residences, to the south and west by single family residences. On the date of our investigation, the site of the new building was a grass covered field with a baseball field in the northwest corner. Several mature trees were observed in the project vicinity. The service access road along the northern school border supported asphalt concrete; numerous cracks were observed along the access road. Evidence of underground utilities (sprinkle lines, electric lines, and gas lines) were noted throughout the site.

Topography across the site is essentially level with surface elevation of approximately +15 feet relative to mean sea level (msl), based on review of the USGS *National Topographic Map, Lodi South, California, 7.5-minute series*, dated 1968 (1980 edition). Portion of the USGS map containing the site and vicinity is included with this report as Figure 1.

SITE HISTORY

The project site history was compiled based on review of historical aerial photographs (dated 1957, 1967, 1982, and 1984) and Google Earth images (dated 1993, 1998, 2002 through 2020 and 2022 through 2025).

The school is visible in the 1957 photograph. The area proposed for the addition is a grass covered area. A structure (northeast corner of the proposed classroom building) is visible in the 1982 photograph. The area south of the structure is paved. A second building is visible in the 1993 photograph. This building has been removed sometime between 1993 and 1998. The area proposed for the addition has remained essentially unchanged since 1998.

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 Lodi 30'x60' Quadrangle, California*, dated 2009, indicates the project site is underlain by the Late Pleistocene Lower member Modesto Formation (Map Symbol: Qm1b), described as fine grained alluvium of flood basins, lower fans, and intertributary fan areas. (Figure 2a).

The United States Geological Survey (USGS) *Geologic Maps of the Sacramento - San Joaquin Delta, California*, dated 1982, indicates the project site is underlain by Holocene and(or) Upper Pleistocene alluvium of Calaveras River (Map Symbol: Qcr), described as clayey silt, silt, sandy silt, and subordinate sand and gravel. (Figure 2b).

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 Jacktone Clay (Map symbol 180), 0 to 2 percent slopes. The soil profile consists of clay to a depth of 34 inches, underlain by an indurated layer to a depth of 37 inches, over stratified sandy loam to clay loam to a depth of 46 inches, over cemented loam soils to a depth of 60 inches.

“The soils are very limited for the construction of dwellings without basements and small commercial buildings due to shrink/swell potential. The soils are very limited for the construction of local roads and streets due to shrink/swell potential and low strength. Soils are very limited to shallow excavations predominantly due to the shallow depth to saturated zone. Soils are very limited to lawns and landscaping predominantly due to the shallow depth to saturated zone and high clay content. Soils are anticipated to exhibit “high” corrosion potential to steel and “low” corrosion potential to concrete”.

SUBSURFACE SOIL CONDITIONS

Existing pavement sections encountered in the Borings D1, D2, and D3 consisted of 4 inches of asphalt concrete (AC) over 7 inches of aggregate base (AB), 1½ inches of AC over 7 inches of AB, and 2½ inches of AB over 6½ inches of AB, respectively.

Fill soils were encountered in the Boring D1. Fill soils consist of silty sands and extend to a depth of approximately 3½ feet bgs.

In general, native soils encountered in the test borings and CPT soundings predominantly consist of clays to depths of approximately 1 foot to 4 feet bgs, underlain by interbedded layers of silty sands, poorly graded sands, sandy silts and clays to depths of approximately 34 to 44 feet bgs. These strata are underlain by poorly graded sands to the maximum depth explored of 51½ feet bgs.

For soil conditions at a specific location, please refer to the Logs of Soil Borings (Figures 4 through 10) and Logs of CPT Soundings (Figures 12 and 13). Graphic illustrations of the subsurface conditions encountered in the borings and CPT soundings are presented on the geologic cross-sections (Figures 16 through 18).

Please note that subsurface conditions within the borings and CPT soundings are representative of the soil conditions at the time of exploration and at the specific location. It should be expected that soil conditions across the site can and will vary laterally and vertically from the soils encountered during our investigation.

GROUNDWATER

At the time of the drilling, on March 19, 2025, groundwater was initially encountered at a depth of approximately 30 feet bgs. Groundwater was measured at a depth of approximately 27 feet bgs prior to backfilling the borings with grout.

After the completion of the advancement of the CPT soundings on March 17, 2025, groundwater was measured in soundings at depths of approximately 27 feet bgs and 28 feet bgs.

Pore pressure dissipation tests conducted in the CPT soundings indicate depths to groundwater of approximately 25 feet bgs (CPT1) and 27 feet bgs (CPT2).

Review of the Depth to Groundwater Maps produced by the California Department of Water Resources for the period from 2011 through 2024 indicates that the depth to groundwater ranged from 30 to 40 feet below ground surface (bgs).

We reviewed groundwater data for well 02No6E09C002M located approximately ¾-mile southeast from the project site. Review of the groundwater data for the well for the period from 1963 through 1978 indicates that the depth to groundwater ranged from approximately 22 feet bgs in 1969 to approximately 54 feet bgs in 1977.

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.

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 Greenville Fault Zone, located approximately 29 miles (46½ kilometers) southwest of the project site. This fault zone is the closest fault to the site exhibiting activity in the Holocene. A Regional Fault Map (Figure 19) 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 7, located approximately 24.2 miles (38.9 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 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 Elkhorn Elementary School Site

Fault Name	Maximum Magnitude (M _w)	Distance To Site Miles (Kilometers)
Great Valley 7	6.9	24.2 (38.9)
Great Valley 5, Pittsburg Kirby Hills	6.7	28.0 (45.1)
Greenville Connected	7.0	29.3 (47.1)
Mount Diablo Thrust	6.7	34.7 (55.9)
Great Valley 4b, Gordon Valley	6.8	37.2 (59.8)
Green Valley Connected	6.8	37.3 (60.0)
Calaveras;CN	6.87	40.1 (64.6)
Calaveras;CN+CC+CS	7.03	40.1 (64.6)
Calaveras;CN+CC	7.0	40.1 (64.6)
Great Valley 8	6.8	43.6 (70.2)
Hayward-Rodgers Creek;HN+HS	7.0	48.0 (77.3)
Hayward-Rodgers Creek;HS	6.78	48.0 (77.3)
Hayward-Rodgers Creek;RC+HN+HS	7.33	48.0 (77.3)
Great Valley 4a, Trout Creek	6.6	49.1 (79.0)
Calaveras;CC	6.39	49.2 (79.1)
Calaveras;CC+CS	6.5	49.2 (79.1)
West Napa	6.7	50.3 (80.9)
Hayward-Rodgers Creek;HN	6.6	50.5 (81.2)
Hayward-Rodgers Creek;RC+HN	7.19	50.5 (81.2)
Ortigalita	7.1	53.9 (86.7)
Hunting Creek-Berryessa	7.1	54.6 (87.8)
Great Valley 3, Mysterious Ridge	7.1	58.8 (94.6)
Hayward-Rodgers Creek;RC	7.07	61.1 (60.1)

CN Calaveras North
 CC Calaveras Central
 CS Calaveras South
 RC Rodgers Creek
 HN Hayward North
 HS Hayward South

The Geodetic zone of distributed shear (C Zone) # 1 utilized in the preparation of the USGS 2008 National Seismic Hazard Maps is located approximately 8 miles north 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 Midland Fault Zone located approximately 12 miles (19.7 kilometers) west from the site. The closest mapped fault to the site is the concealed Pre-Quaternary Stockton Fault located approximately 9 miles (14.5 kilometers) south 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 the USGS Unified Hazard Tool (2014) indicate that the mode (modal) magnitude earthquake for the site is 5.5. 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 2021). 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. Historic earthquake epicenter maps showing earthquakes (magnitude 5 or greater) within the project region are presented as Figure 20.

A review of the historical earthquake data indicates that the most significant earthquake shaking (accelerations) experienced at the project site occurred during the 1869 Hayward Earthquake (site acceleration of 0.087g) and 1836 South San Francisco Bay Region earthquake (site acceleration of 0.084g). The estimated magnitude of these events was 6.8. The closest epicenter is located approximately 29.4 miles (47.4 kilometers) southwest of the site and is attributed to one of the aftershocks of the 1980 Livermore earthquakes. Series of the events of the Livermore earthquake produced site accelerations of between 0.051g and 0.074g. An examination of the tabulated EQSEARCH data suggests that the project site has experienced maximum ground shaking equivalent to Modified Mercalli Intensity VII¹ as the result of these earthquakes.

Among the most recent earthquakes, the 2007 Alum Rock (Mw=5.6) and the 2014 South Napa (Mw=6.0) events produced estimated site peak ground accelerations of 0.043g and 0.052g, 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.

Table 2 – Magnitudes and Exceedances	
TABLE OF MAGNITUDES AND EXCEEDANCES	
Earthquake Magnitude	Number of Times Exceeded
5.0	38
5.5	25
6.0	11
6.5	3

Output files from the EQSEARCH program are included in Appendix C.

¹ VII – Very Strong: Damage is negligible in buildings of good design and construction; but slight to moderate in well-built ordinary structures; damage is considerable in poorly built or badly designed structures; some chimneys are broken. Noticed by motorists.

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). The site is not located within the State designated Seismic Hazard Zone map (Unevaluated Area).

SITE CLASS

Based on the results of the shear wave velocity testing performed in the sounding CPT1, it is our opinion that the on-site soils should be designated as Site Class D in determining seismic forces for this project in accordance with Table 1613.3.2 of the 2022 CBC. The weighted average shear wave velocity is estimated at 832 ft/s.

SEISMIC CODE PARAMETERS

The 2022 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool (<https://seismicmaps.org/>). This web-based software application calculates seismic design parameters in accordance with ASCE 7-16 and 2022 CBC. The results indicate a mapped S_1 value of 0.280. Per Section 11.4.8, a site-specific ground motion study should be performed in accordance with Section 21.2 of ASCE 7-16 for Site Class D sites with S_1 value greater than or equal 0.2.

Supplement 3 to Section 11.4.8 of ASCE 7-16 includes an exception from such analysis for specific structures on Site Class D sites.

EXCEPTION: A ground motion hazard analysis is not required where the value of the parameters SM_1 determined by Eq (11.4-2) is increased by 50% for all applications of SM_1 in this Standard. The resulting value of the parameter m determined by Eq. (11.4-4) shall be used for all applications of SD_1 , in this Standard.

The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11.4.8 of ASCE 7-16) states that “In general, this exception effectively limits the requirements for site-specific hazard

analysis to very tall and or flexible structures at Site Class D sites.” Based on our understanding of the proposed structures, it is our assumption that the exception in Section 11.4.8 applies to the proposed structure. However, the structural engineer should verify the applicability of this exception.

Based on this exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 1613.2.3 of the 2022 CBC.

Table 3 –2022 CBC Seismic Design Parameters*

Latitude: 38.0506° N Longitude: 121.3344° W	ASCE 7-16 Table/Figure	2022 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2	Figure 22-1	Figure 1613.2.1(1)	S_s	0.706 g
1.0 Period MCE	Figure 22-2	Figure 1613.3.1(2)/ Figure 1613.2.1(3)	S_1	0.280 g
Soil Class	Table 20.3-1	Section 1613.2.2	Site Class	D
Site Coefficient	Table 11.4-1	Table 1613.2.3(1)	F_a	1.235
Site Coefficient	Table 11.4-2	Table 1613.2.3(2)	F_v	2.04
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-36/ Equation 16-20	S_{MS}	0.872 g
	Equation 11.4-2	Equation 16-37/ Equation 16-21	S_{M1}	0.857** g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-38/ Equation 16-22	S_{DS}	0.582 g
	Equation 11.4-4	Equation 16-39/ Equation 16-23	S_{D1}	0.571** g

* 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

** Increased by 50 percent

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

The site modified peak ground acceleration PGA_M (Equation 11.8-1, ASCE 7-16) is 0.385 g.

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 2022 editions 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 the Geodetic zone of distributed shear (C Zone 1), Greenville Fault, and Great Valley Fault System.

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 historically active Greenville dextral strike-slip fault zone borders the eastern side of Livermore Valley and is considered to be part of the larger San Andreas fault system in the central Coast Ranges. The fault zone extends from northwest of Livermore Valley along the Marsh Creek and Clayton faults towards Clayton Valley. Unruh and Sawyer suggested that slip from the Greenville fault is transferred to the Concord fault along the Mt. Diablo fold and thrust belt and that only minimal slip

continues to the Clayton fault. The fault zone extends southeastward into San Antonio Valley, offsets late Mesozoic rocks of the Franciscan Complex. Southeast of Livermore Valley the fault is located within the uplifted Diablo Range and controls the generally linear drainage course of Arroyo Mocho, Colorado, and Sweetwater Creeks. Maximum dextral displacement along the Greenville fault zone is about 8.5–9 km, based on 9 km of dextral offset of a late Mesozoic serpentinite body and about 8.5 km dextral offset of the Tesla fault. Trench investigations along the Marsh Creek-Greenville section document latest Pleistocene and Holocene displacement. Partial late Quaternary slip rates of 0.1–0.7 mm/yr have been reported by Wright and others and Sweeney. Unruh and Sawyer suggested that the late Quaternary slip rate might be as high as 3 mm/yr on the basis of structural modeling. Sawyer and Unruh calculated a Holocene dextral slip rate of 4.1 ± 1.8 mm/yr.

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.

Surface Fault Rupture

No known faults with surface expression 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 causative faults contributing to the estimated site PGA are Geodetic zone of distributed shear ($M=5.78 - 6.13$ events), Greenville Fault ($M=7.15$ event), Mount Diablo Thrust ($M=7.06 - 7.29$ events), and the Great Valley Fault System ($M=6.83 - 7.28$ events). The calculated mode magnitude is 5.5.

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 in the current Seismic Hazard Zone for liquefaction as designated by the State of California (Unevaluated Area), which delineates areas of historical occurrence of liquefaction or local geological, geotechnical and groundwater conditions indicating a potential for permanent ground displacement.

According to the *Envision Stockton, 2040 General Plan Update and Utility Master Plan Supplements, Draft EIR*, dated June 2018:

“Although the EIR Study Area has not been mapped by the CGS or USGS for potential liquefaction hazards, the potential for seismically-triggered liquefaction could be a concern where silty sediments are prevalent and the groundwater table is shallow. However, based on reviews of local geotechnical investigations, including an investigation in support of a new courthouse in Downtown Stockton, much of the shallow sediments beneath the EIR Study Area are dominated by clays and clay-rich deposits. These soil types are less susceptible to liquefaction behavior. Therefore, the likelihood of substantial adverse effects to the EIR Study Area due to seismically-

triggered liquefaction is considered low and the impact from implementation of the proposed General Plan and UMPS would be less than significant.”

According to the San Joaquin Council of Governments, 2018 Regional Transportation Plan/ Sustainable Communities Strategy, Draft Programmatic Environmental Impact Report:

“The California Department of Conservation has not identified any liquefaction hazard zones within San Joaquin County; however, various areas of the county may be subject to liquefaction during seismic events due to high groundwater levels. Liquefaction is found near the Manteca-Lathrop area, the area just west of Woodbridge (including a small portion of the town site), and the Delta region. The east and northeast portions of the county are less susceptible because groundwater is deep.”

We performed our liquefaction analysis for the CPT soundings using the software program GeoSuite (Version 4.1.1.12). The analysis was performed using the Robertson (2009) methodology. Input earthquake ground motion for the liquefaction analysis was 0.385 g (PGAM, 2022 CBC Section 1803.5.11 for Site Classification D); Magnitude Earthquake (MW) of 5.5; in-situ depths to groundwater at the time of field exploration of 25 feet bgs and 27 feet bgs, and a depth to groundwater of 22 feet bgs during seismic event.

Liquefaction (FS against liquefaction less than 1.3) may occur in thin, sand layers encountered at depths below 25 feet bgs. The following table presents depths and thicknesses of liquefiable layers predicted for each CPT sounding.

Table 4 – Depths and Thicknesses of Liquefiable Layers	
CPT Sounding	Depths and Thicknesses of Liquefiable Layers, feet bgs
CPT 1	29 - 29¾
	30½ - 31½
CPT 2	38½ - 39
	43½ - 44

Considering the depths to liquefiable soils (deeper than 25 feet bgs) and the fact that liquefiable soils are thin layers and overlaid by non-liquefiable clays, loss of bearing

capacity of the foundations and surface manifestation of the liquefaction are not anticipated. Output files of liquefaction analysis are presented in Appendix D.

Cyclic Softening of Clay and Clay-like Soils

Cyclic softening of clay soils is 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. Cyclic softening of clay like behavior of soils was modeled in the computer program, GeoSuite (Version 4.1.1.12), used in our analyses. Based on these results, we do not anticipate cyclic softening of clays and clay-like soils beneath the site.

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 the anticipated grading (essentially level site topography), it is our opinion that the potential for lateral spreading at the site is low.

Seismically Induced Settlement

We performed estimation of the seismic settlement of the soils beneath the site for the CPT soundings using the software program GeoSuite (Version 4.1.1.12). The analysis was performed using the Robertson (2009) methodology. Input earthquake ground motion for the liquefaction analysis was 0.385 g (PGAM, 2022 CBC Section 1803.5.11 for Site Classification D); Magnitude Earthquake (MW) of 5.5; in-situ depths to groundwater at the time of field exploration of 25 feet bgs and 27 feet bgs, and a depth to groundwater of 22 feet bgs during seismic event. The seismic settlement of 0.14 inch was predicted for each CPT sounding

Conservatively, it is our opinion that total and differential seismic settlements of ½-inch and ¼-inch in 40 linear feet, respectively, should be anticipated for the design. Output files of seismic settlement analysis are presented in 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 extraction. In 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).

Review of the *Measurements of Vertical Ground Surface Displacement between January of 2015 and January of 2025*, maintain by the California Department of Water Resources indicates that the total vertical surface settlement was 0.154 foot.

Considering the subsurface soil conditions and the historical and current depths to the groundwater, hydrocollapse of soils beneath the site is not anticipated.

Landslides and Slope Stability

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

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), Map Number 06077C0315F, published by FEMA, with an effective date of October 16, 2009, the proposed site improvements lie within Zone X, Areas with reduced flood risk due to levee (southern portion of the site) and within Zone X, Areas with 0.2 percent annual chance flood hazard (northern portion of the site). It is our opinion that the site has a low risk of flooding (Figure 21).

Review of the *Dam Breach Inundation Map Web Publisher*, maintained by Department of Water Resources and *Envision Stockton, 2040 General Plan Update and Utility Master Plan Supplements, Draft EIR, dated June 2018*, indicates that the site is located in the Camanche 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 92 miles (148 kilometers) southeast of Clear Lake Volcanic Area, the closest volcanic area indicated. The closest known area of the Quaternary volcanic eruption (Sutter Buttes) is 80 miles (135 kilometers) north of the site. Based on the above information, it is our opinion that the 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. San Joaquin 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.

Review of the Special Report 247 *California Indoor Radon Potential*, produced by CGS and dated 2022, indicates the site is located in the *Preliminary moderate to high Radon Potential Area*.

CONCLUSIONS

FOUNDATION AND STRUCTURAL SUPPORT

The site contains existing underground sprinkler lines and trees; therefore, proper clearing and removal of existing and former improvements and proper backfilling of excavations is very important to provide adequate and uniform structural support. Demolition of existing improvements, 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.

Also, to provide adequate and uniform support for the proposed foundations it is recommended to provide a minimum of 12 inches of recompacted engineered fill below the foundations for the new classroom building.

It should be noted the soils exposed immediately beneath and adjacent to existing landscaped 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 engineered fill, properly placed and compacted in accordance with the recommendations of this report, will be capable of supporting the proposed structure and associated improvements.

Provided the 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

Laboratory testing of the on-site soils indicates they possess a medium to high expansion potential when tested in accordance with the ASTM D4829 test method (see Figures A1 and A2). In our opinion these soils, when present within the upper portion of the building pad, are capable of exerting significant 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. In floor slab and exterior flatwork areas, on past projects, replacement with imported non-expansive soils or aggregates, or lime treatment of expansive soils has produced significant reductions in expansive soil movements, but some floor slab (both interior and exterior) movement can still occur. Specific recommendations to reduce the effects of expansive soils are presented in this report.

SUITABILITY OF ON-SITE SOILS FOR USE AS FILL

In our opinion, the on-site soils encountered in our soil borings are considered suitable for use in engineered fill construction, provided they are free of debris and organics and are at a suitable moisture content to achieve the desired degree of compaction. On-site clays should not be used within 12 inches from the finished building pad or exterior flatwork subgrades, unless treated with lime to lower their expansion potential. Special recommendations are contained in ENGINEERED FILL CONSTRUCTION section of this report regarding the use of expansive clays in engineered fill.

EXCAVATION CONDITIONS

Based on our field investigation, the on-site soils should be readily excavatable with conventional earthmoving and trenching equipment typically used in the area.

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

Two 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 Table 6.

Table 6 – Soil Corrosivity Testing				
Sample Identification	CA DOT Test #643 Modified (Sm. Cell)		CA DOT 417	CA DOT 422
	pH	Minimum Resistivity	Chloride	Sulfate
D6 (0-2')	7.54	1,720 Ω-cm	16.7 ppm	27.3 ppm
D7 (0-1')	7.41	1,070 Ω-cm	32.5 ppm	27.0 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 3.2, March 2021, 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 2022 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 our experience in the vicinity of the project site, the on-site clay soils are poor quality materials for the support of asphalt concrete pavements. Therefore, we have

selected an R-Value of 5 for the design of pavements constructed on untreated pavement subgrades.

Based on our experience, chemical treatment of the near-surface clay soils may result in a substantial improvement to the support characteristics of the soil subgrade and reduce the required thickness of the base materials by increasing the R-value. Based on the laboratory test results of a lime treated sample of the anticipated pavement subgrades soils (see Figure A3), an R-value of 50 was utilized for the design of pavements constructed on a chemically treated subgrade.

Chemical treatment also can be used to reduce the moisture content of near-saturated soils to facilitate grading operations, and can reduce the expansion potential of clay.

PERMANENT GROUNDWATER

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

SEASONAL WATER

The near-surface soils also may be in a near-saturated condition during and for a significant time following the rainy season due to rainwater being unable to penetrate through the underlying clay and/or 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, silty sands, and sandy silts. In our opinion, the undisturbed soils may be susceptible to erosion by surface run-off that occurs during intense rainfall. As a minimum, erosion control measures including placement of straw bale sediment barriers or construction of silt filter fences in areas where surface run-off may be concentrated would be prudent. The project civil engineer should develop a site-specific erosion and sediment control plan based upon their site grading and drainage plan and the anticipated construction schedule.

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

RECOMMENDATIONS

The project is in a preliminary stage of design; 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 maximum excavations and fills of up to two 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 underground utilities scheduled for removal, vegetation, trees, remnants from former construction, and any deleterious materials to expose firm and stable soil conditions as identified by our on-site representative. Trees and dense brush designated for removal should include the rootball and all exposed roots greater than ½-inch in diameter.

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 the proposed building pad should be completely removed and/or rerouted as necessary. Utilities located outside the building area should be properly abandoned (i.e., fully grouted provided the abandoned utility is situated at least 2½ feet below the final subgrade level to reduce the potential for localized “hard spots”).

Remaining areas should be stripped of surface vegetation; strippings may be stockpiled for later use or disposed of off-site. If used on-site, strippings may be placed in landscaped areas, provided they are kept at least five feet from the building pads and areas or flatwork

or pavements, moisture conditioned and compacted. Strippings should not be used in landscaped berms that will support either walls, concrete flatwork, or any other improvements that are susceptible to damage from settlement.

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 soil prior to backfilling with properly placed and compacted engineered fill to restore the areas back to the required grades.

It is essential that our representative be present during clearing operations to verify adequate removal of existing and former improvements 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) and affected areas of improvements 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.

Following site clearing operations, the entire building pad should be over-excavated to a depth equal to the bottoms of the proposed foundations, or 2 feet below existing site grades, whichever is deeper. Where practical, the zone of over-excavation should extend 5 feet beyond the perimeter lines of the proposed building. 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 two percent above the optimum moisture content and compacted to at least 90 percent of the ASTM D1557 maximum dry density. Subgrades must be firm and stable, prior to placing engineered fill. Excavations then should be backfilled with engineered fill as recommended in this report.

Areas to receive exterior flatwork and pavements whether left at-grade, to receive fill or achieved by excavation, should be ripped and cross-ripped to a minimum depth of 12 inches, moisture conditioned to at least two percent above the optimum moisture content and compacted to at least 90 percent of the ASTM D1557 maximum dry density.

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 unstable 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 two percent above the optimum moisture content for clay soils and at least optimum moisture content for granular soils 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.

Clays should not be used within the upper 12 inches of building pad or exterior flatwork fills, unless they are lime-treated to lower their expansion potential.

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.

Building pad construction using non-expansive imported soils and/or lime-treated soils should extend at least five feet beyond the outside edge of building foundations and should also extend at least two feet beyond adjacent flatwork areas.

The upper 12 inches of final untreated building pad subgrades (import or aggregates) should be brought to at least the optimum moisture content and uniformly compacted to not less than 95 percent of the maximum dry density, regardless of whether final grade is achieved by excavation, filling or left at existing grade.

The upper 12 inches of final exterior flatwork subgrades (import or aggregates) should be scarified, brought to at least the optimum moisture content, and uniformly compacted to not less than 90 percent of the maximum dry density, as determined by ASTM D1557, regardless of whether final grade is completed by excavation, filling, or left at-grade.

The upper six inches of untreated pavement subgrades and exterior slab subgrades supporting vehicle loadings should be scarified, moisture conditioned to at least two percent above the optimum moisture content, processed, and uniformly compacted to at least 95 percent of the maximum dry density, regardless of whether final grade is completed by excavation, filling, or left at existing grade.

Refer to the LIME-TREATMENT ALTERNATIVE section of this report for treatment and compaction of treated grades.

Final pavement subgrade preparation and compaction should be performed just prior to placement of aggregate base, after construction of underground utilities is complete. The completed pavement subgrades must be proof-rolled and stable under construction traffic prior to placement of aggregate base.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2:1) and should be vegetated as soon as practical following grading to minimize erosion. 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. Slopes should be over-built and cutback to design grades and inclinations.

Subgrade soils for support of buildings and pavements shall be protected from disturbance or desiccation until covered by capillary break material or aggregate base. Disturbed or desiccated subgrade soils shall require moisture conditioning, scarification and recompaction, as required by the Geotechnical Engineer, depending on the level of desiccation or disturbance.

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 compliance with our recommendations and job specifications as well as complete removal of exposed undocumented fills and/or unstable soil deposits, to observe the earthwork construction, and perform compaction testing.

LIME-TREATMENT ALTERNATIVE

It will be important that the subgrade soils be observed and evaluated after site clearing to determine the most appropriate treatment options based on the exposed soil conditions.

The following are preliminary recommendations for clay soil subgrades. Revised recommendations will be needed if sands and silts are exposed at pavement subgrade elevations. Additional laboratory testing will be needed to determine the most appropriate product (or products) for the exposed soil conditions.

If lime-treatment is selected for pavement subgrades and/or building pad/flatwork subgrades, site preparation should be performed in accordance with the following recommendations. Following the site preparation, as recommended above, the upper 12 inches of the building pad and exterior flatwork grades and the upper 12 inches of the pavement subgrades should be treated with at least four percent high-calcium or dolomitic quicklime, as measured by dry unit weight of the untreated soil.

A large self-propelled rotary mixer should be used for mixing and remixing. Lime should be thoroughly mixed and remixed, as necessary, to a minimum depth of 12 inches at a minimum spread rate of at least 4½ pounds of lime per square foot based on a 12-inch mixing depth. This spread rate is provided for preliminary estimation purposes only as the actual amount of product can only be determined at the time of construction based upon the prevailing

site, soil and moisture conditions. The contractor should include an add/deduct unit price for lime to account for variations in the quantities of product used.

It is emphasized that higher spread rates and/or deeper mixing depths with proportionately higher spread rates will be needed for areas exposing the wet and/or unstable soils.

Initial mixing of lime should be followed by remixing the next day. Additional remix passes should be performed to provide a uniform soil-lime mixture. Lime stabilized soils should be compacted to at least 92 percent of the ASTM D1557 maximum dry density at a moisture content of at least two percent above the optimum moisture content for building pad and flatwork areas, and 95 percent within pavement areas. The moisture content of the treated soils should be maintained in the soil until the treated soil is covered by aggregate base or slabs. Compaction operations should be undertaken with a heavy, self-propelled, compactor and should be performed in the presence of our representative who will evaluate the performance of the subgrade under compactive load. No equipment or vehicle traffic should be allowed on the lime-treated materials during the first three days after treatment and compaction are completed.

If the lime-treatment alternate is selected, additional laboratory testing may be warranted to further define the amount of lime required to produce the desired results.

A contractor experienced in such work should perform lime-treatment as specified in Chapter 24 of the *Caltrans Standard 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. 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.

If subgrades are lime-treated, then the upper 12 inches of trench backfill should consist of properly compacted Class 2 aggregate base.

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 1½-inches and ¾-inch in 40 linear feet, respectively, should be anticipated for the design of the proposed foundations.

Provided the building pads are over-excavated and re-compacted as recommended, the proposed structures may be supported upon continuous and/or isolated spread foundations extending at least 24 inches into the prepared building pad, or at least 24 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 7 below for the

design in accordance with 2019/2022 CBC 1605A.1.1, assuming the structure would be designed for a system overstrength factor (Ω_0) of 3. For foundations designed using ASD, the factor of safety for soil bearing pressure shall not be less than the overstrength factor.

Table 7 – Allowable Bearing Pressures

Load Condition	Ultimate Bearing Pressure (psf)	Minimum Factor of Safety	Allowable Bearing Pressure (psf)
Dead plus Live Loads	10,000	4	2,500
Total Loads (Including Wind or Seismic)	10,000	3	3,300

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 a minimum of 12 inches of non-expansive granular soils, Class 2 aggregate base, or lime-treated soils over 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 shop areas and 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.

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 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.

DRILLED CAST-IN-PLACE CONCRETE PIERS

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

In general, we recommend the proposed piers consist of drilled, straight-shafted holes, filled with concrete, and reinforced with steel to resist and transfer lateral and axial loads. Further, we recommend the pier excavations should have a minimum diameter of 24 inches and a minimum depth of 5 feet below lowest adjacent soil final grade. The Structural Engineer should determine the actual final pier depth and diameter based on the required capacities.

Drilled pier foundations may be sized for a maximum allowable soil pressure of 2,000 pounds per square foot (psf) for Allowable Stress Design such as using the Basic Load Combinations listed in Sections 1605.1 of 2022 CBC. This value can be increased by one-third to consider the short-term effects of seismic or wind forces if they are not factored in the load combinations such as the Alternative Basic Load Combinations listed in Sections 1605.2 of 2022 CBC for dead plus live load. The weight of the foundation concrete extending below adjacent soil grade may be disregarded in sizing computations.

Alternatively, cast-in-place concrete drilled piers should be designed to resist downward loads using an allowable unit skin friction of 150 psf. Total downward capacities derived from the parameters provided above may be increased by 1/3 for total load conditions to include short-term loading due to wind or seismic forces.

We recommend the uppermost 1 foot of the embedded portion of the foundation be neglected when evaluating the skin friction component of the axial capacities due to the expansion potential of the on-site clay soils.

Sizing of piers to resist lateral loads can be evaluated using Section 1807.3.2 of the 2022 California Building Code (CBC). A value of 150 pounds per cubic foot (pcf) for lateral bearing as defined in Table 1806.2 of the CBC may be used for the coefficients S_1 and S_3 for the nonconstrained and constrained conditions, respectively. Per Section 1806.1 of the 2022

CBC, an increase of $1/3$ is permitted when using the alternate load combinations in Section 1605.2 that include wind or earthquake loads. The upper 18 inches of embedment should be neglected for the non-constrained condition unless exterior flatwork or pavements completely surrounds the top of the pier and extends at least three feet beyond the outside edge of the pier.

Actual depth and diameter of piers should be determined by the Structural Engineer based on actual design loading conditions and requirements. Larger and deeper piers than the minimum recommended may be needed based on actual design loading requirements and capacities.

Drilled Pier Construction Considerations

Relatively cohesionless soils were encountered during our field investigation. In our opinion, these soils may be susceptible to caving, requiring advancing casing, drilling fluid, and/or other methods to advance the drilled excavations for the proposed piers and maintain hole stability. The contractor should be prepared to fully case the drilled pier holes, as needed.

Casing

If casing is used, we recommend it be removed from the excavation as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete at all times during casing withdrawal and concrete placement. Further, continuous vibration or other approved methods should be used during casing withdrawal to reduce the potential for void-space formation within the concrete. Abandoning the casing in-place should not be allowed.

Drilling Fluids

If drilling fluids are used to facilitate construction of the proposed drilled pier, we recommend steel reinforcement and concrete be placed immediately upon completion of the pier excavation to reduce the quantity of suspended soil particles which may settle to the bottom of the hole. Further, we recommend all pier construction operations which utilize drilling fluids be in accordance with procedures outlined in the Federal Highway Administration publication titled: *Drilled Shafts: Construction Procedures and Design Methods*.

Bottom Preparation

All debris, loose or disturbed soil, and any water (if applicable) should be removed from the pier excavation just prior to placing reinforcing steel and/or concrete. A representative from Mid Pacific Engineering should observe the bottom of the pier and its excavation to verify that subsurface conditions are consistent with those encountered during our field investigation and to verify competent support conditions are exposed.

Steel and Concrete Placement

Reinforcement and concrete should be placed in the pier excavations immediately upon completion of the pier excavation to minimize the chances of sidewall caving into the excavations. If water is present during concrete placement, or if drilling fluids are used to advance the pier, concrete should be pumped or otherwise discharged to the bottom of the hole via a hose or tremie pipe. The end of the hose or tremie pipe must remain below the top surface of any water, drilling fluids, and the in-place concrete at all times. In addition, concrete used for pier construction should be consolidated using vibratory methods over the entire length and width of the pier. If water and/or drilling fluids are present, concrete within the upper portion of the pier should be consolidated to the extent possible upon removal of these fluids.

It is considered essential that our representative be present during pier drilling to verify adequate depth of penetration. Concrete reinforcing steel should not be placed in any pier excavation until approved by our representative.

EXTERIOR FLATWORK

Due to expansive clays at the site, exterior flatwork should be supported on a minimum of 12 inches of non-expansive granular soils, Class 2 aggregate base, or lime-treated soils moisture conditioned and compacted as recommended. 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.*

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 buildings, 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

The following pavement sections have been calculated based on anticipated traffic indices (TI's), results of R-value testing, and the procedures contained within Chapters 600 to 670 of the *California Highway Design Manual*, Sixth Edition. An R-Value of 5 was used for the design on-site pavements constructed on native, untreated subgrade soils. An R-value of 50 was used for the design of clayey subgrades amended with at least 4 percent high calcium or dolomitic quicklime as measured by dry weight of soil to be treated. The project civil engineer should determine the appropriate traffic index based on anticipated traffic conditions. We can provide additional section thicknesses for other TI's, as needed.

Table 8 - Pavement Design Alternatives				
Traffic Index (TI)	Untreated Subgrades R-value = 5		Lime-Treated Subgrades (a) R-value = 50	
	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2½*	10	2½*	4
5.0	2½	11	2½	4
	3*	10	3*	4
6.0	2½	15	2½	6
	3	14	3	5
	3½*	13	3½*	4
7.0	3	18	3	7
	4*	16	4*	5

* = Asphalt concrete thickness includes the Caltrans Safety Factor.

(a) = Lime-treated subgrade should be at least 12 inches thick and possess a minimum R-value of 50 when testing in accordance with CT 301.

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 City of Stockton 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 untreated pavement subgrades should be uniformly moisture conditioned to at least the optimum moisture content and compacted to at least 95 percent relative compaction. Lime-treated subgrades should be compacted as recommended in the Lime-Treatment Alternative section. 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.

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 and CPT 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 the 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.

Vasily V. Parfenov
Vasily V. Parfenov
Senior Geologist
CEG No. 2355



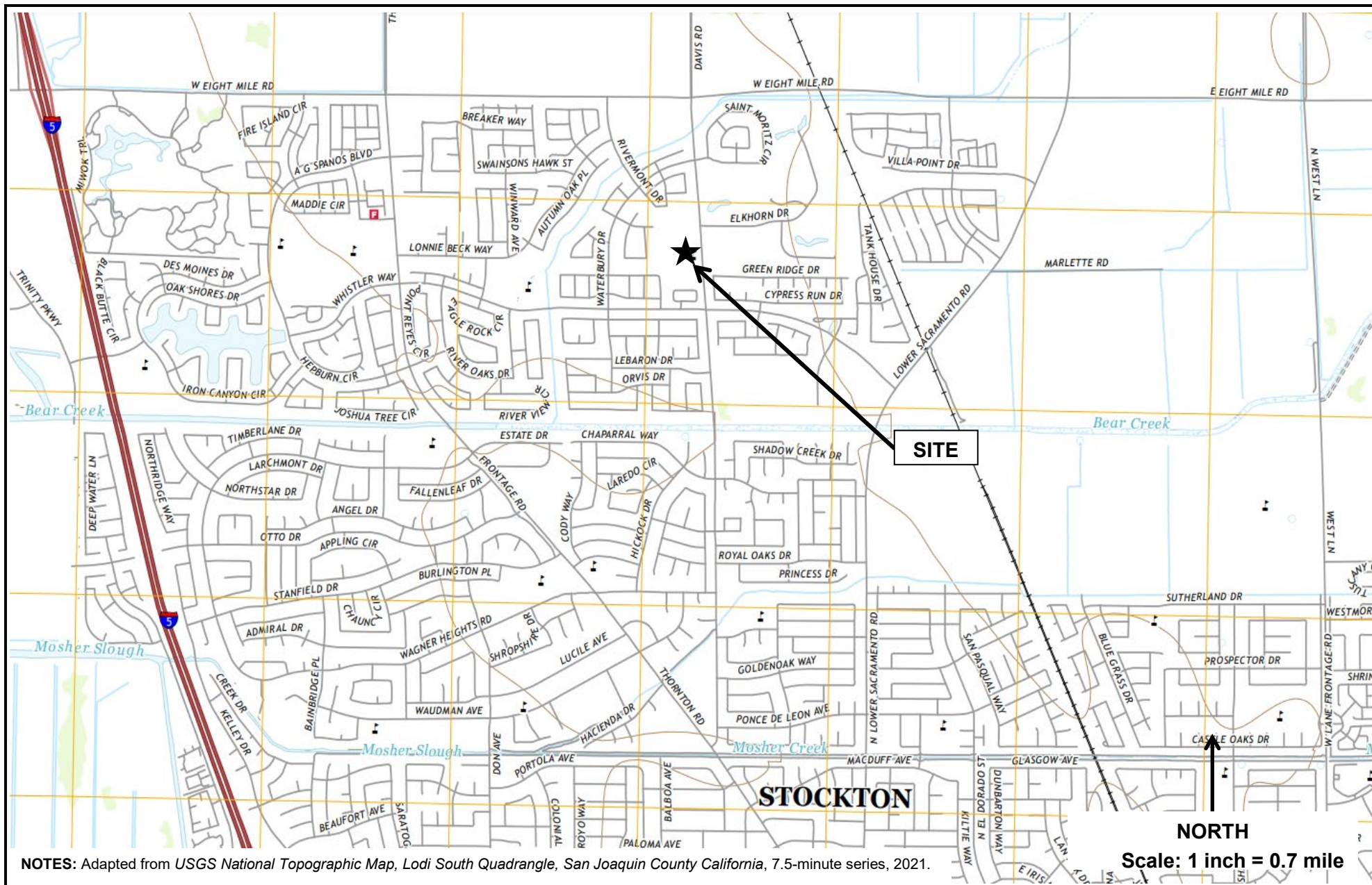
Daniel C. Smith
Daniel C. Smith
Principal Engineer
GE No. 2530



Martin S. Osier
Martin S. Osier
Senior Project Engineer
PE No. 89860

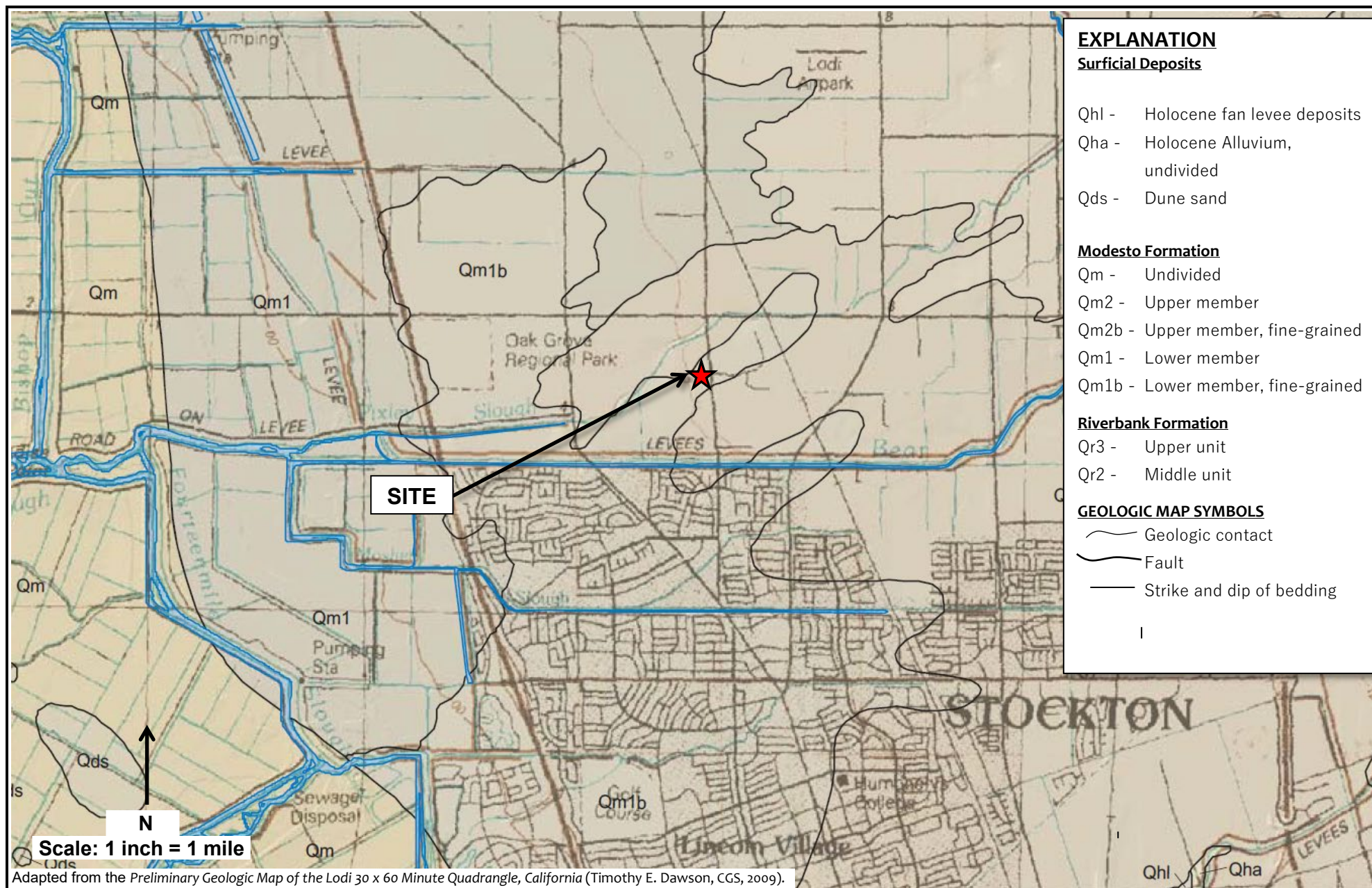


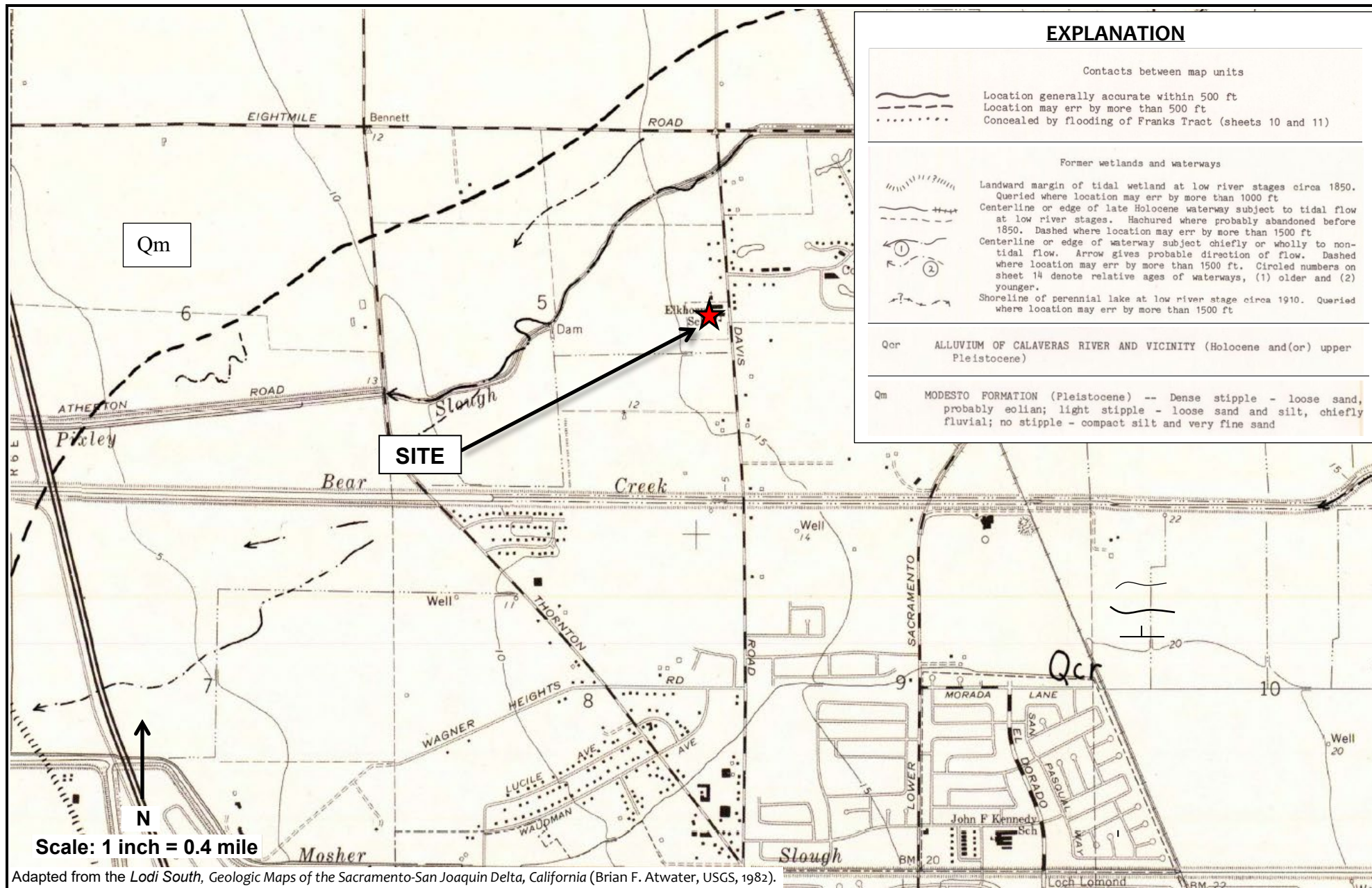
FIGURES



VICINITY MAP
ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
 10505 Davis Road
 Stockton, California

FIGURE 1
 Date: 06/25
 MPE No. 06317-03







NOTES: Adapted from *Elkhorn School Option E* prepared for Lodi USD, undated



SITE INVESETIGATION MAP
ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
 10505 Davis Road
 Stockton, California

FIGURE 3
 Date: 06/25
 MPE No. 06317-03

Project: Elkhorn ES New Classroom Building and Remodeling		LOG OF SOIL BORING D2	
Project Location: 10505 Davis Road, Stockton, California		Sheet 1 of 1	
MPE Number: 06317-03			
Date(s) Drilled	3/19/2025	Logged By	AW
Checked By	VVP		
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling
Total Depth of Drill Hole	10 Feet		
Drill Rig Type	CME 55	Diameter(s) of Hole	4 inches
Approx. Surface Elevation			
Groundwater Depth below ground surface	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop
Drill Hole Backfill	Neat Cement		
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
			1½ inches AC over 7 inches AB						
			Gray-Brown, moist, medium stiff, lean clay (CL)		D2-1	3 3 4			
			Olive-Brown, very stiff		D2-2	6 8 11			
5									
					D2-3	5 8 9			
10									
15									
20									
25									

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FIGURE 5

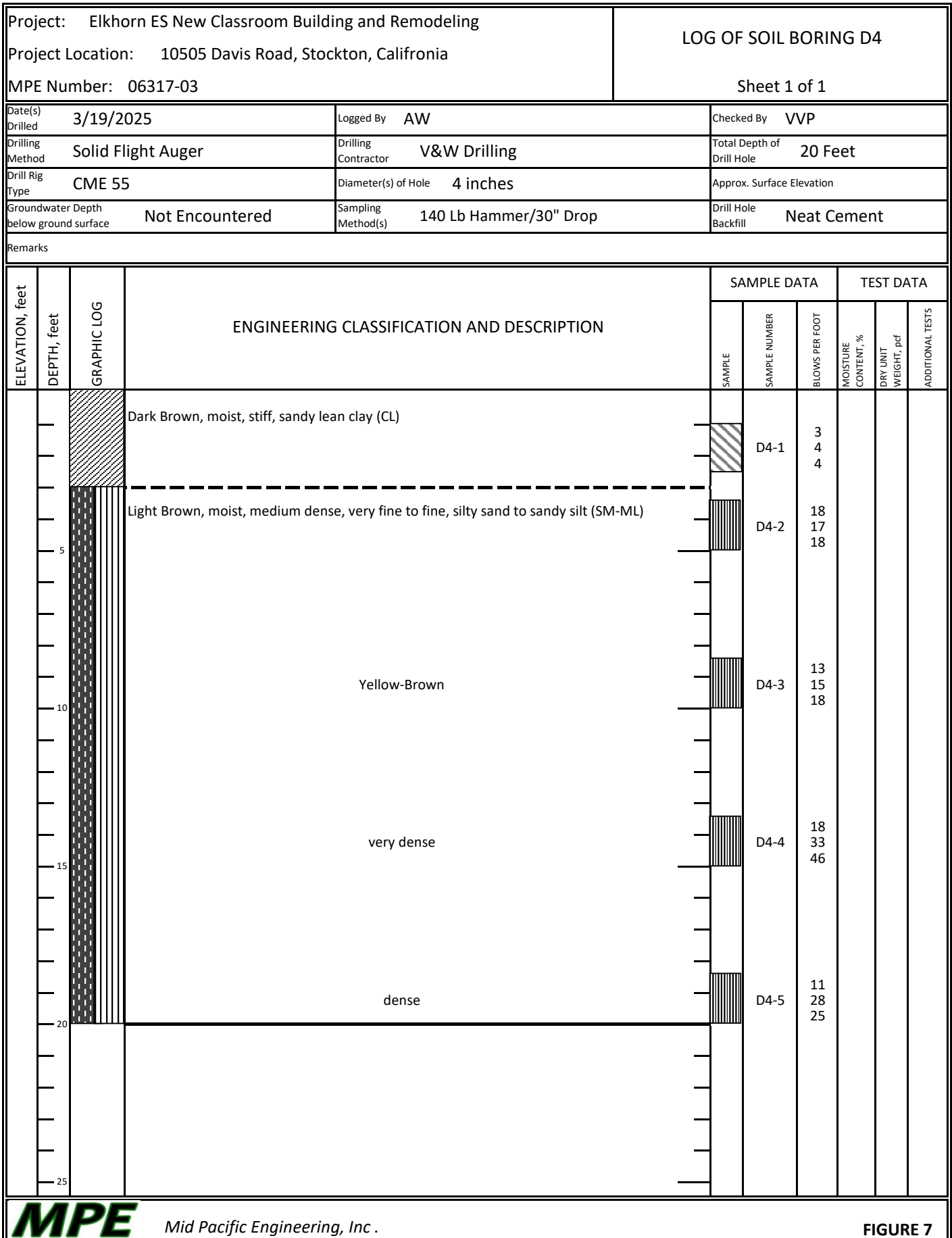
Project: Elkhorn ES New Classroom Building and Remodeling				LOG OF SOIL BORING D3			
Project Location: 10505 Davis Road, Stockton, California				Sheet 1 of 1			
MPE Number: 06317-03							
Date(s) Drilled 3/19/2025		Logged By AW		Checked By VVP			
Drilling Method Solid Flight Auger		Drilling Contractor V&W Drilling		Total Depth of Drill Hole 10 Feet			
Drill Rig Type CME 55		Diameter(s) of Hole 4 inches		Approx. Surface Elevation			
Groundwater Depth below ground surface Not Encountered		Sampling Method(s) 140 Lb Hammer/30" Drop		Drill Hole Backfill Neat Cement			
Remarks							

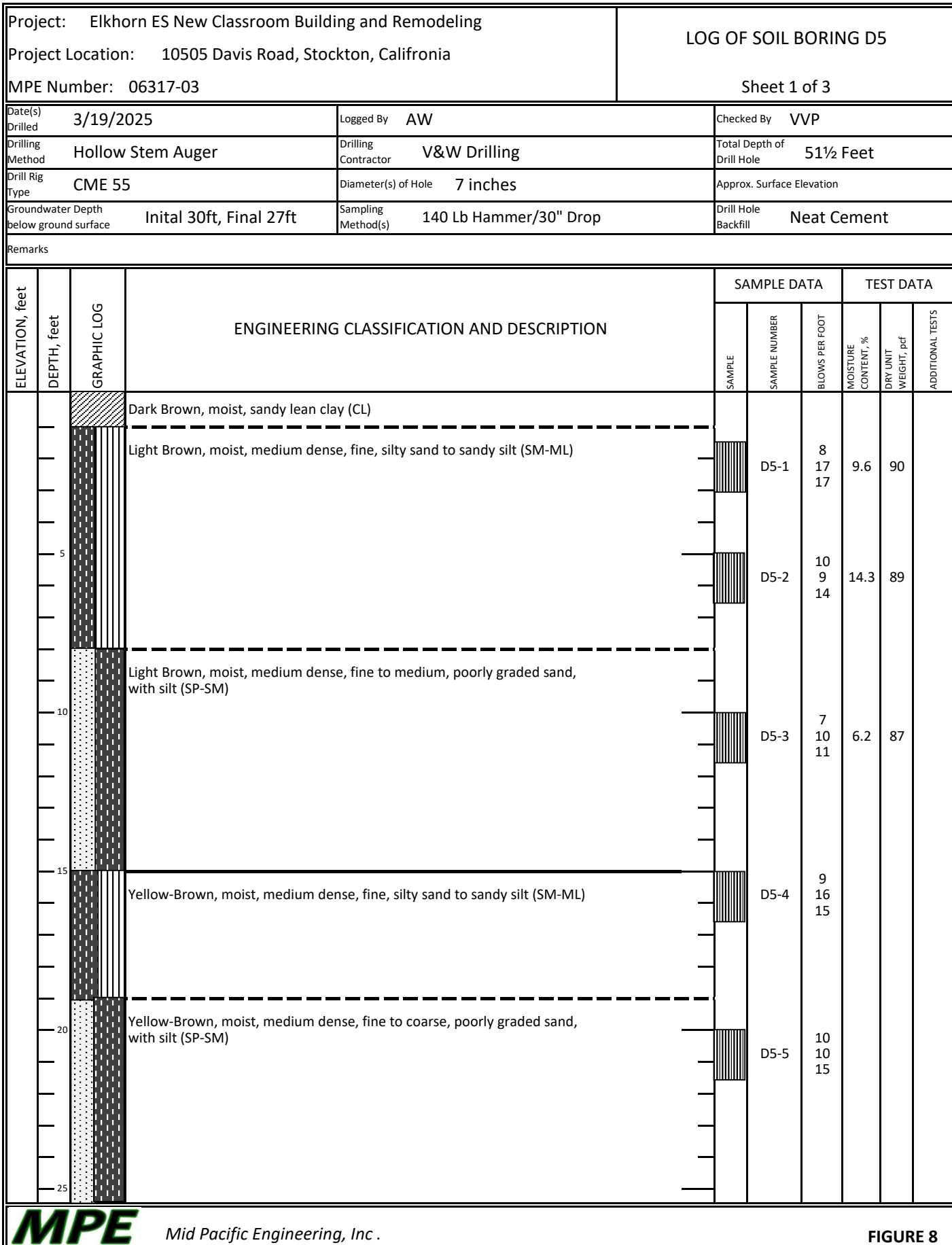
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			2½ inches AC over 6½ inches AB						
			Gray-Brown, moist, medium stiff, lean clay (CL)		D3-1	3 3 4			
			Olive-Brown, moist, very stiff		D3-2	4 11 8			
			medium stiff		D3-3	2 3 4			

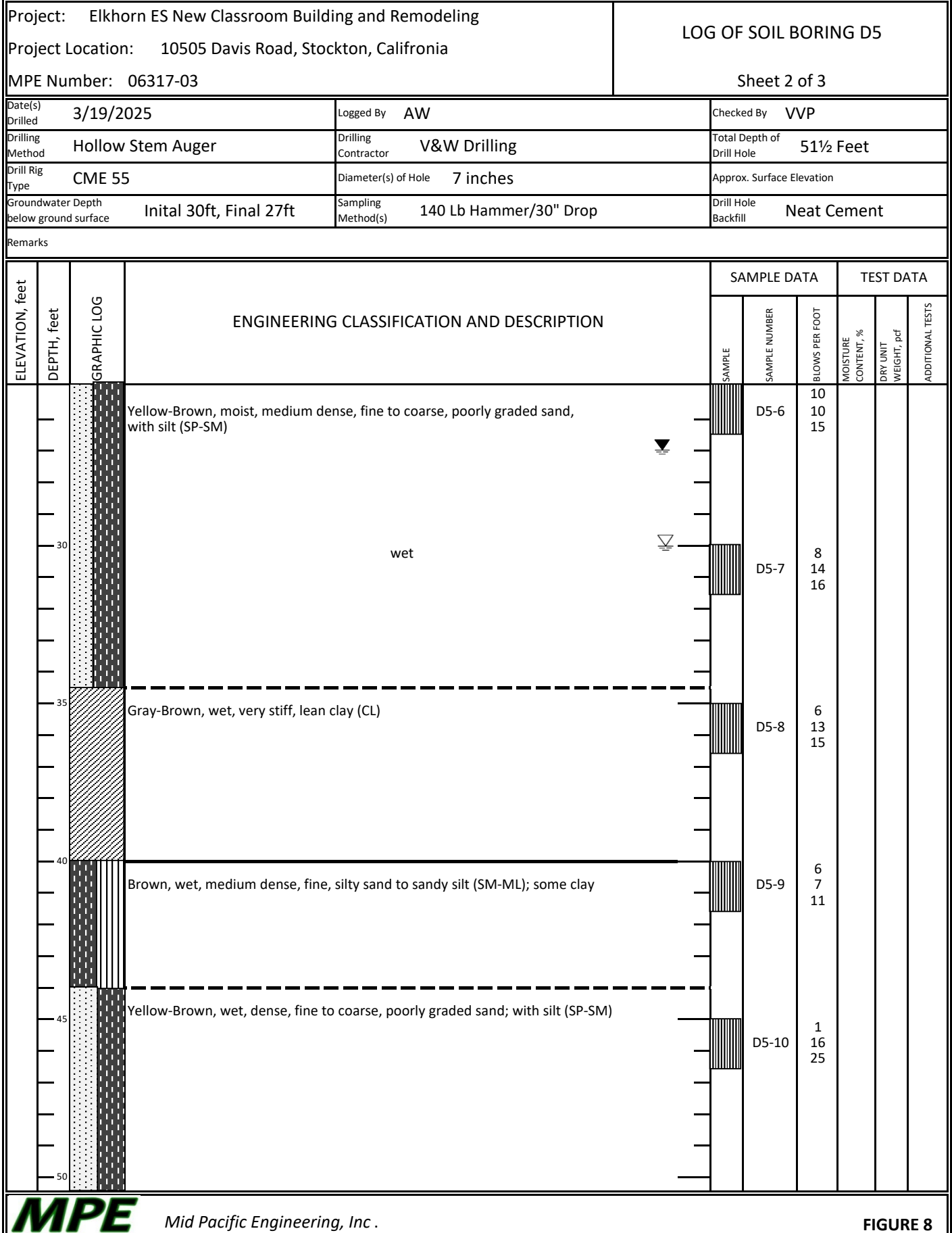


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FIGURE 5







Project: Elkhorn ES New Classroom Building and Remodeling			LOG OF SOIL BORING D5		
Project Location: 10505 Davis Road, Stockton, California			Sheet 3 of 3		
MPE Number: 06317-03					
Date(s) Drilled 3/19/2025		Logged By AW		Checked By VVP	
Drilling Method Hollow Stem Auger		Drilling Contractor V&W Drilling		Total Depth of Drill Hole 51½ Feet	
Drill Rig Type CME 55		Diameter(s) of Hole 7 inches		Approx. Surface Elevation	
Groundwater Depth below ground surface Initial 30ft, Final 27ft		Sampling Method(s) 140 Lb Hammer/30" Drop		Drill Hole Backfill Neat Cement	
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
			Yellow-Brown, wet, very dense, fine to coarse, poorly graded sand; with silt (SP-SM)		D5-11	6 40 50/5"			
55									
60									
65									
70									
75									

Mid Pacific Engineering, Inc .

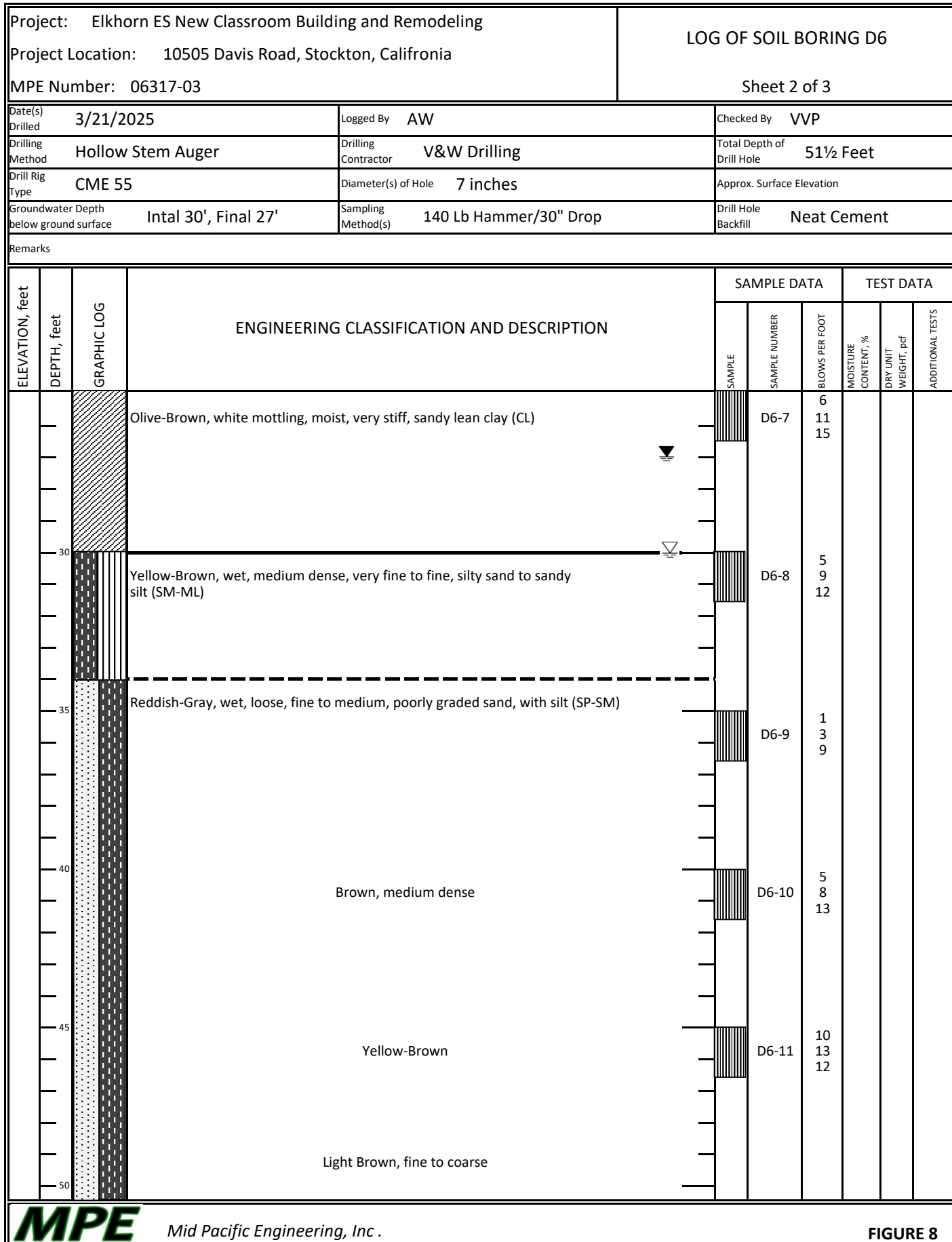
FIGURE 8

Project: Elkhorn ES New Classroom Building and Remodeling Project Location: 10505 Davis Road, Stockton, California MPE Number: 06317-03	LOG OF SOIL BORING D6 Sheet 1 of 3
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Date(s) Drilled: 3/21/2025	Logged By: AW	Checked By: VVP
Drilling Method: Hollow Stem Auger	Drilling Contractor: V&W Drilling	Total Depth of Drill Hole: 51½ Feet
Drill Rig Type: CME 55	Diameter(s) of Hole: 7 inches	Approx. Surface Elevation:
Groundwater Depth below ground surface: Intal 30', Final 27'	Sampling Method(s): 140 Lb Hammer/30" Drop	Drill Hole Backfill: Neat Cement

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, stiff, sandy lean clay (CL)		D6-1	2 6 8	14.3	100	UCC = 1.070 tsf
			Olive, moist, medium dense, very fine to fine, silty sand to sandy silt (SM-ML)		D6-2	6 18 18	8.9	84	
5			Light Brown, Red mottling		D6-3	7 12 12	24.1	86	
			Yellow-Brown, moist, medium dense, fine to medium, poorly graded sand, with silt (SP-SM)						
10			Reddish-Gray and Yellow-Brown		D6-4	4 6 8			
			Brown, moist, dense, very fine to fine, silty sand to sandy silt (SM-ML)		D6-5	9 16 22			
			Yellow-Brown, moist, medium dense, fine to medium, poorly graded sand, with silt (SP-SM)						
20			Yellow-Brown, moist, medium dense, fine to medium, poorly graded sand, with silt (SP-SM)		D6-6	10 16 18			
			Olive-Brown, white mottling, moist, very stiff, sandy lean clay (CL)						
25			Olive-Brown, white mottling, moist, very stiff, sandy lean clay (CL)						



Project: Elkhorn ES New Classroom Building and Remodeling			LOG OF SOIL BORING D6 Sheet 3 of 3		
Project Location: 10505 Davis Road, Stockton, California					
MPE Number: 06317-03					
Date(s) Drilled: 3/21/2025		Logged By: AW		Checked By: VVP	
Drilling Method: Hollow Stem Auger		Drilling Contractor: V&W Drilling		Total Depth of Drill Hole: 51½ Feet	
Drill Rig Type: CME 55		Diameter(s) of Hole: 7 inches		Approx. Surface Elevation:	
Groundwater Depth below ground surface: Intal 30', Final 27'		Sampling Method(s): 140 Lb Hammer/30" Drop		Drill Hole Backfill: Neat Cement	
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
			Light Brown, wet, loose, fine to coarse, poorly graded sand, with silt (SP-SM)		D6-12	12 16 25			
55									
60									
65									
70									
75									

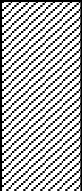
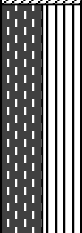
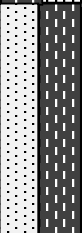
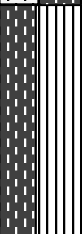
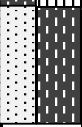

Mid Pacific Engineering, Inc .

FIGURE 8

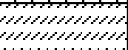
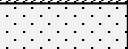
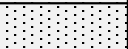
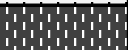
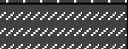
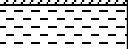
Project: Elkhorn ES New Classroom Building and Remodeling Project Location: 10505 Davis Road, Stockton, California MPE Number: 06317-03	LOG OF SOIL BORING D7 Sheet 1 of 1
---	--

Date(s) Drilled: 3/21/2025	Logged By: AW	Checked By: VVP
Drilling Method: Solid Flight Auger	Drilling Contractor: V&W Drilling	Total Depth of Drill Hole: 21½ Feet
Drill Rig Type: CME 55	Diameter(s) of Hole: 4 inches	Approx. Surface Elevation:
Groundwater Depth below ground surface: Not Encountered	Sampling Method(s): 140 Lb Hammer/30" Drop	Drill Hole Backfill: Neat Cement

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, very stiff, lean clay (CL)		D7-1	5 14 13	23.5	98	UCC = 1.503 tsf
	5		Light Brown, moist, medium dense, fine, silty sand to sandy silt (SM-ML)		D7-2	7 10 14	29.0	91	
	10		Light Brown, moist, loose, fine to medium, poorly graded sand, with silt (SP-SM)		D7-3	5 6 5	24.9	83	
	15		Light Brown, Brown, moist, dense, fine, silty sand to sandy silt (SM-ML)		D7-4	17 21 24			
	20		Light Brown, moist, medium dense, fine to medium, poorly graded sand, with silt (SP-SM)		D7-5	17 16 20			
	25								

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, gravelly 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	76.2 to 4.76
	3" to 3/4"	76.2 to 19.1
	3/4" to No. 4	19.1 to 4.76
SAND coarse (c) Medium (m) fine (f)	No. 4 to No. 200	4.76 to 0.074
	4 to No. 10 No. 10	4.76 to 2.00
	to No. 40 No. 40 to No. 200	2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074



Mid Pacific Engineering Inc

Project
Job Number
Hole Number
EST GW Depth During Test

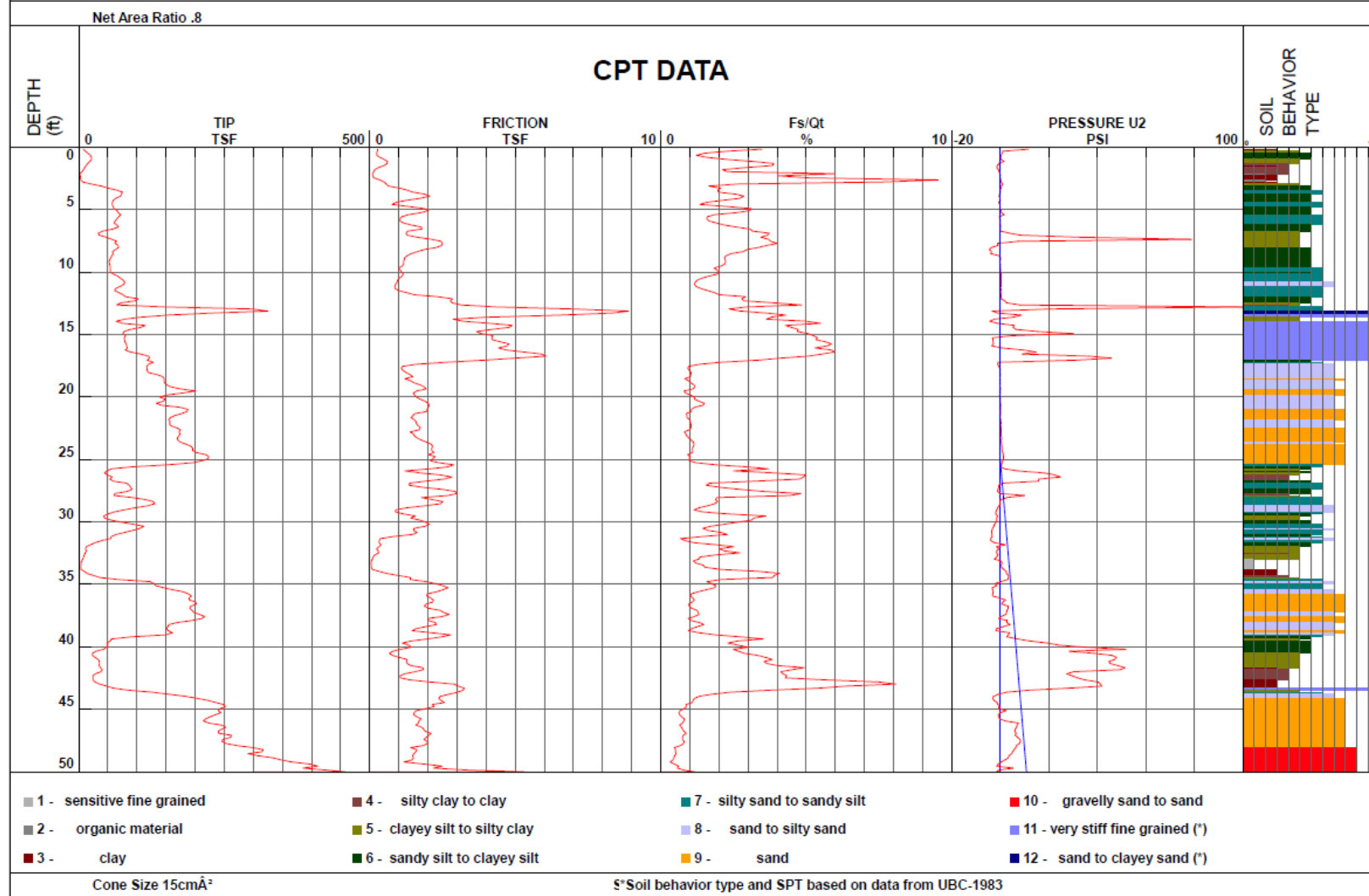
Elkhorn ES
TBD
CPT-01

Operator
Cone Number
Date and Time

JM-IY
DDG1596
3/17/2025 8:17:20 AM

Filename
GPS
Maximum Depth

SDF(711).cpt
50.52 ft



CPT SOUNDING LOG 1

ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California

FIGURE 12

Date: 06/25

MPE No. 06317-03



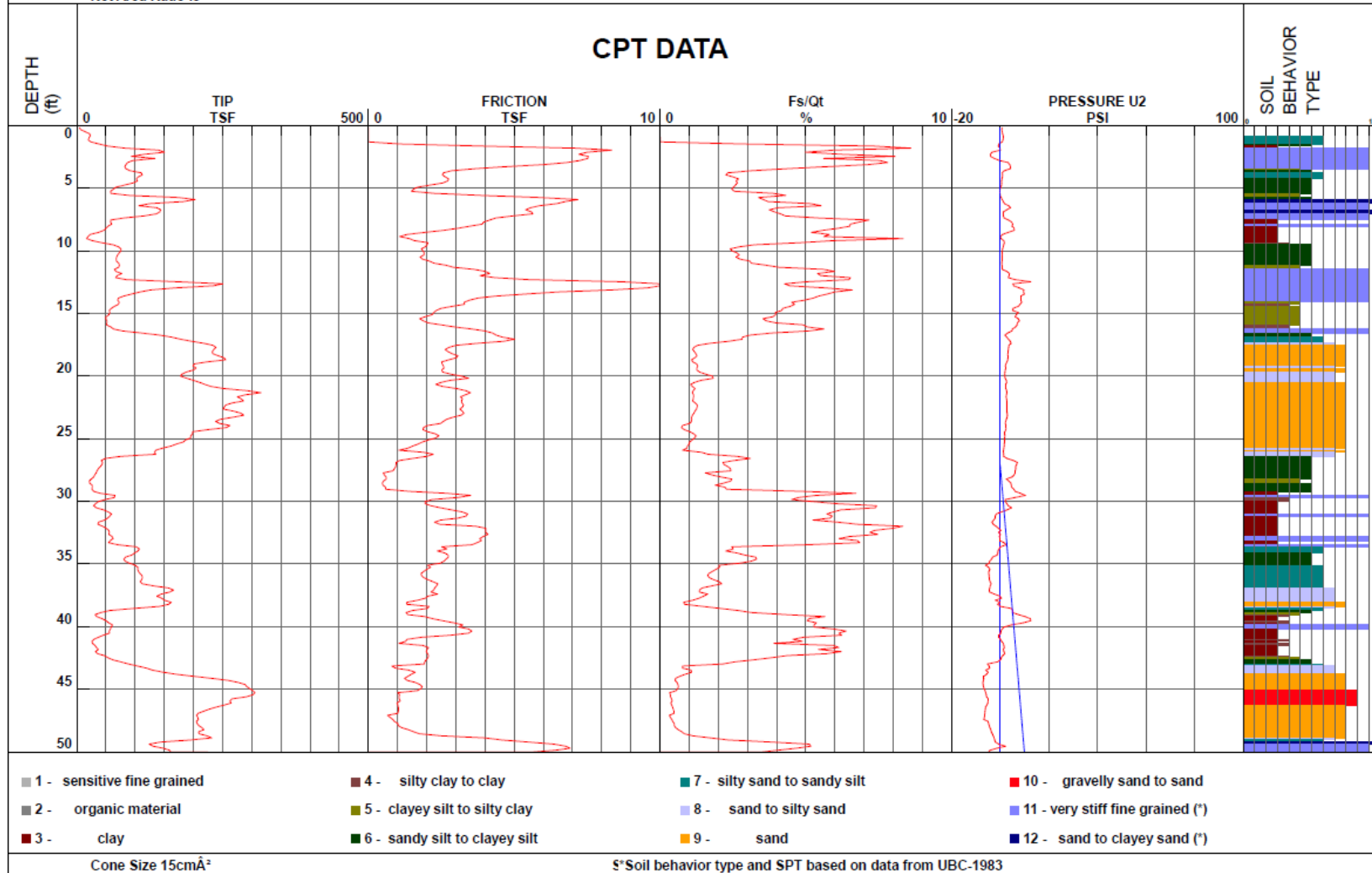
Mid Pacific Engineering Inc

Project Elkhorn ES
Job Number TBD
Hole Number CPT-02
EST GW Depth During Test

Operator JM-IY
Cone Number DDG1596
Date and Time 3/17/2025 9:53:07 AM
27.10 ft

Filename SDF(712).cpt
GPS
Maximum Depth 50.85 ft

Net Area Ratio .8



CPT SOUNDING LOG 2

ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California

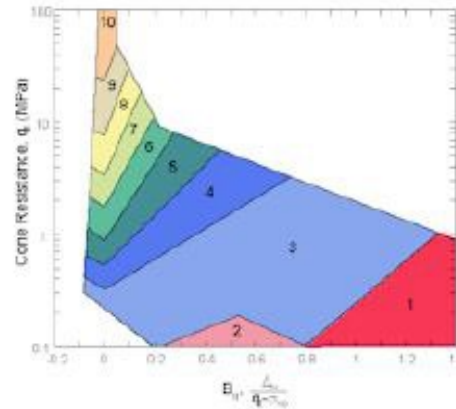
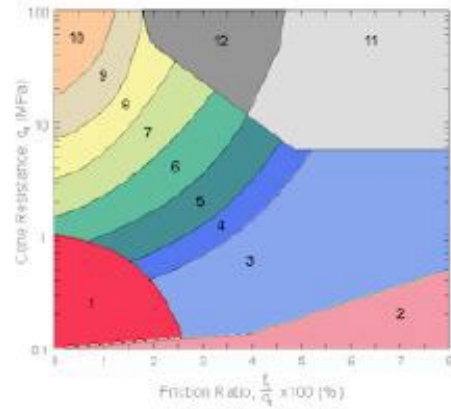
FIGURE 13

Date: 06/25

MPE No. 06317-03

CPT Soil Behavior Type Legend

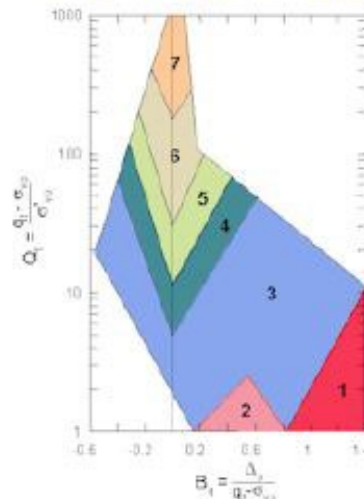
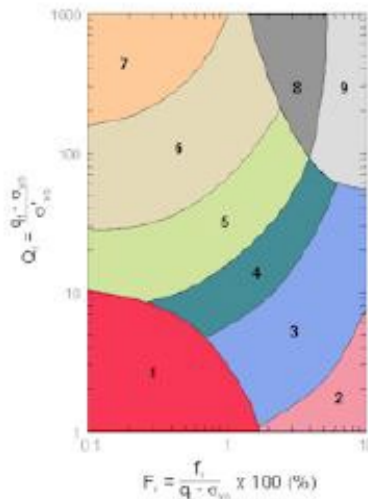
Robertson et al. 1986



Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Material
3	Clay
4	Silty Clay to Clay
5	Clayey Silt to Silty Clay
6	Sandy Silt to Clayey Silt
7	Silty Sand to Sandy Silt
8	Sand to Silty Sand
9	Sand
10	Gravelly Sand to Sand
11	Very Stiff Fine Grained*
12	Sand to Clayey Sand*

*Overconsolidated or Cemented

Robertson et al. 1990



Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Soils-Peats
3	Clays; Clay to Silty Clay
4	Silt Mixtures; Clayey Silt to Silty Clay
5	Sand Mixtures; Silty Sand to Sandy Silt
6	Sands; Clean Sands to Silty Sands
7	Gravelly Sand to Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine Grained*

*Overconsolidated or Cemented

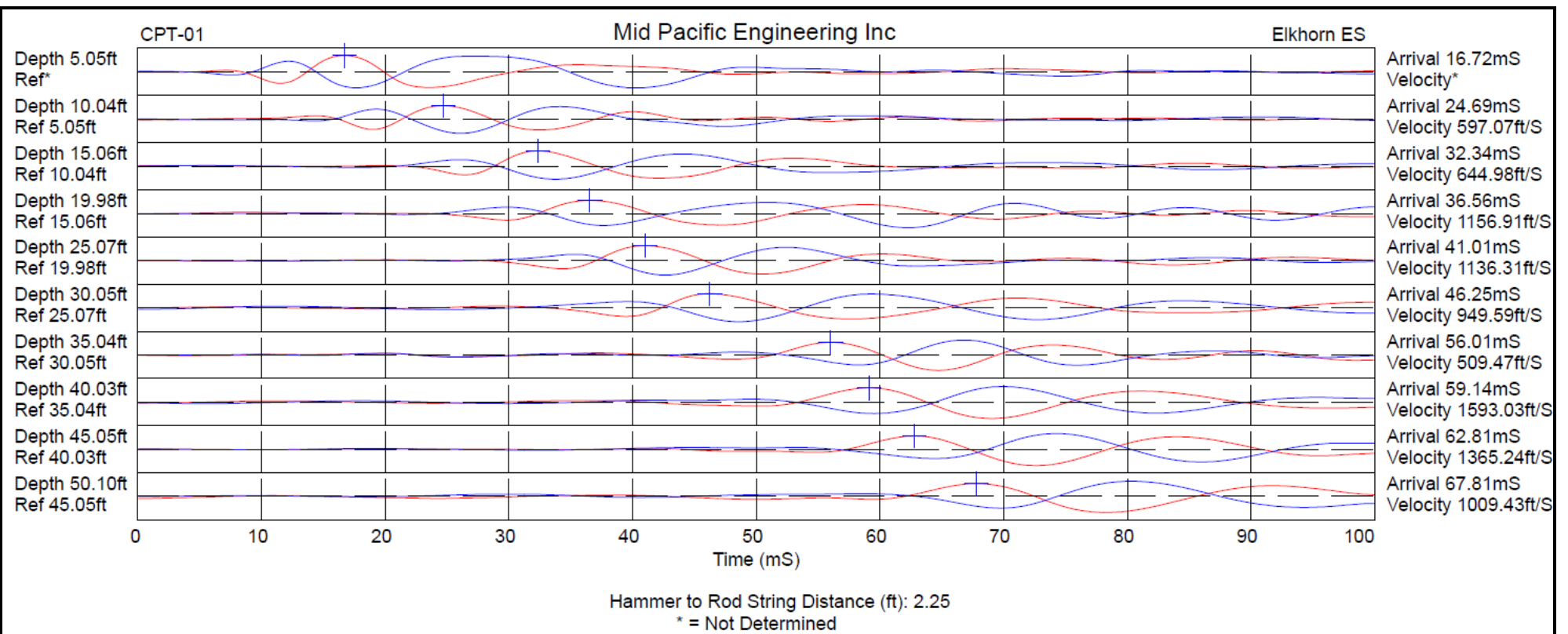
SOIL BEHAVIOR CHART

ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California

FIGURE 14

Date: 06/25

MPE No. 06317-03

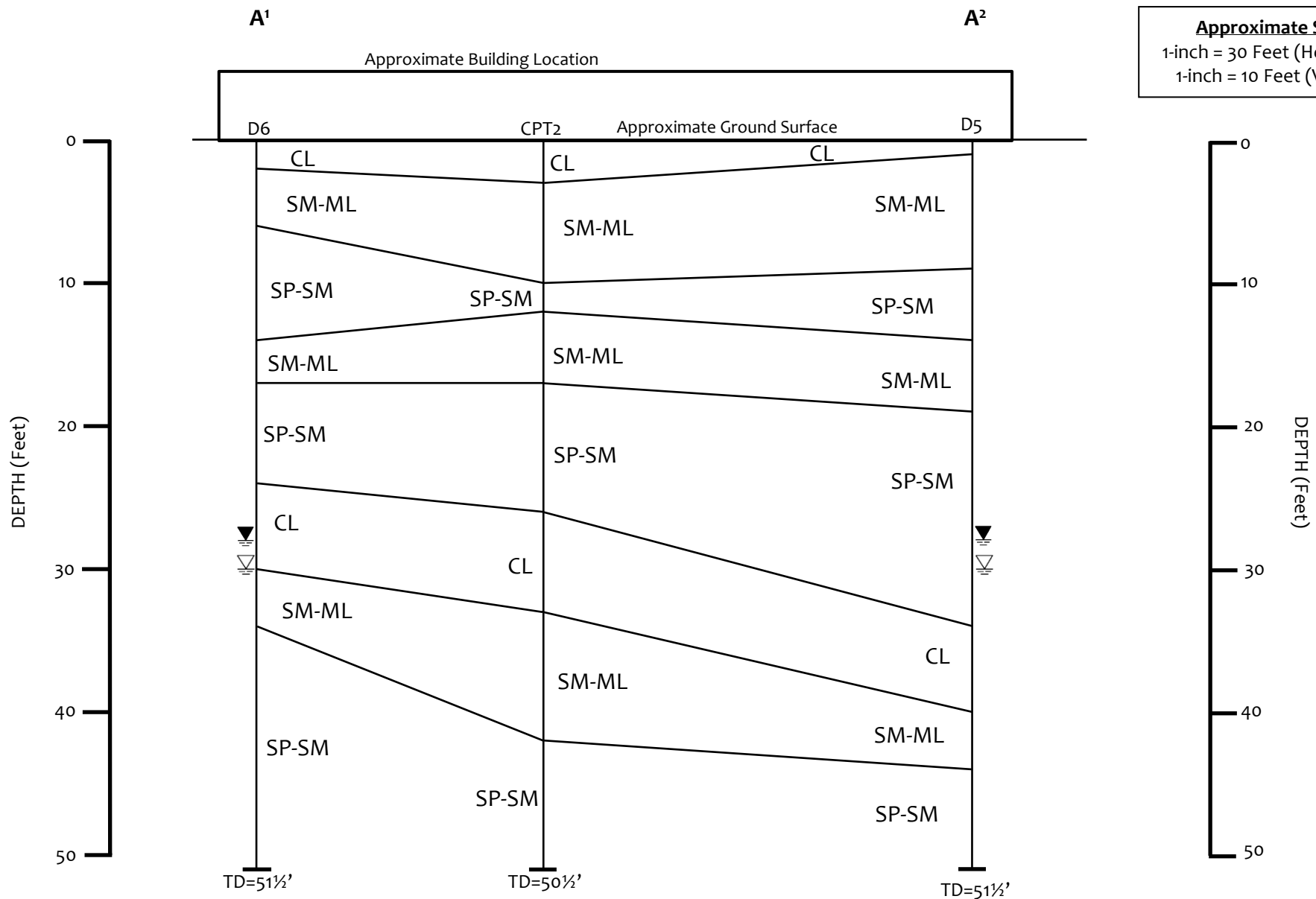


MEASURED SHEAR WAVE VELOCITIES CPT SOUNDING 1
ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
 10505 Davis Road
 Stockton, California

FIGURE 15

Date: 06/25

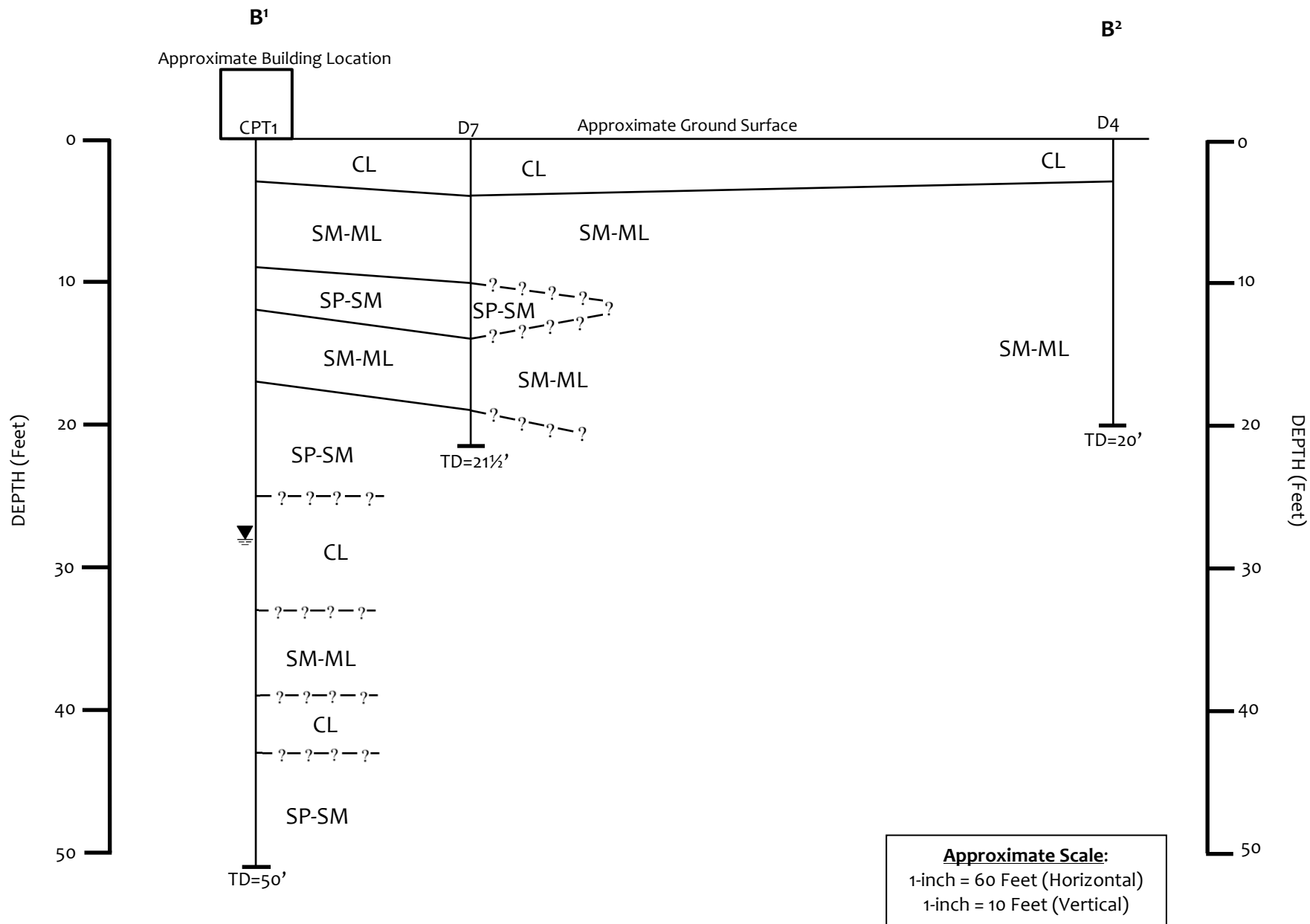
MPE No. 06317-03



ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California

Date: 06/25

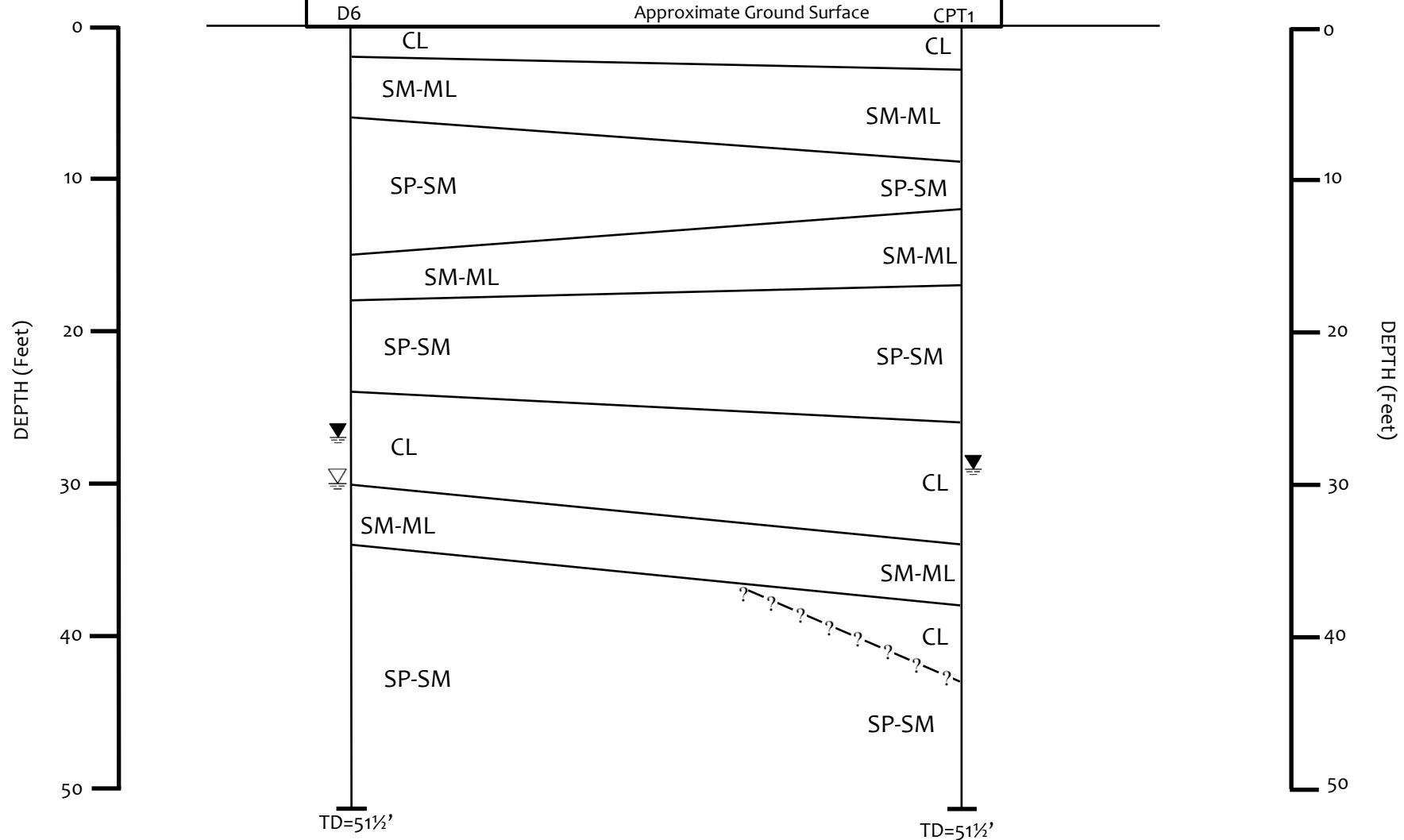
MPE No. 06317-03

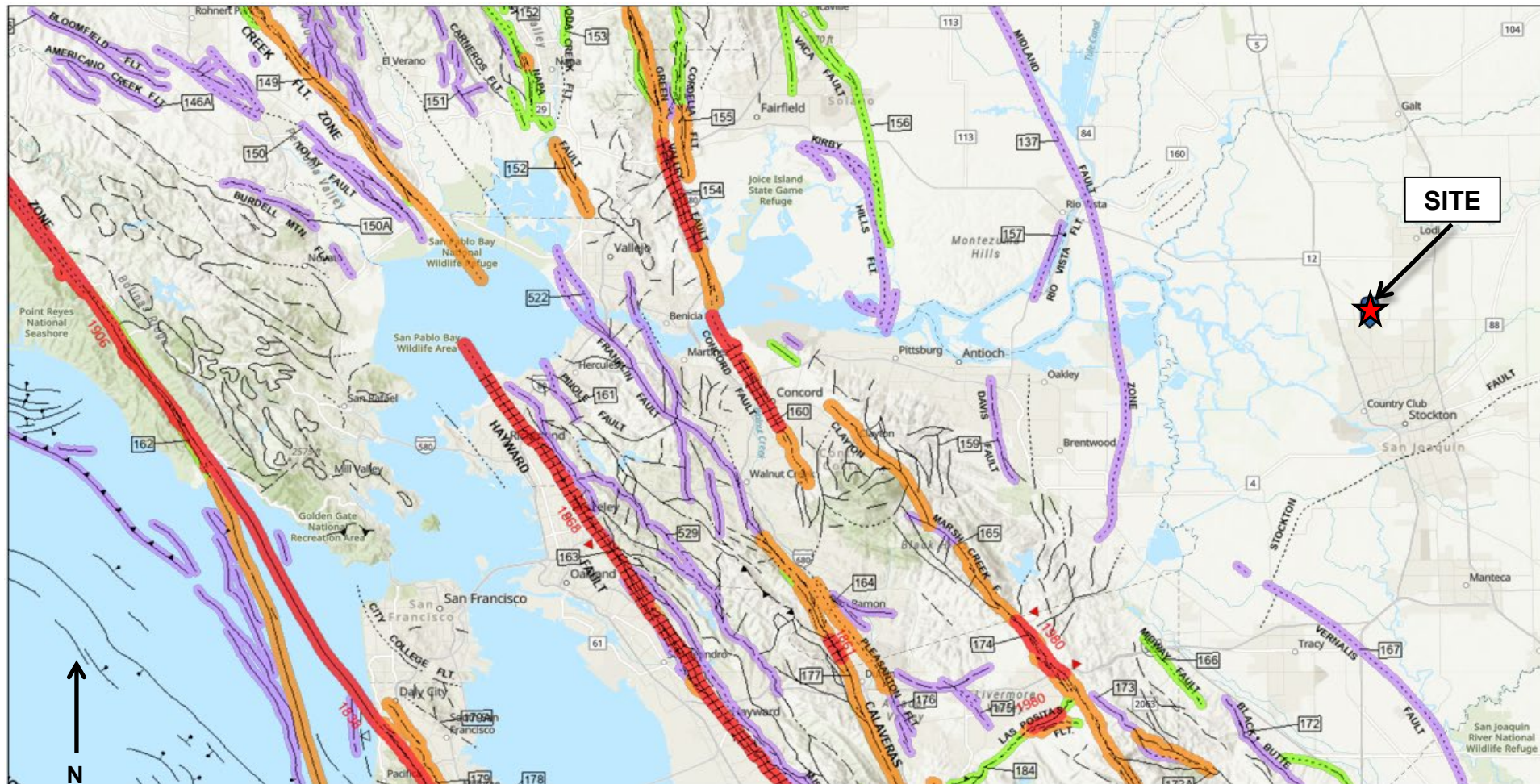


C¹C²

Approximate Building Location

Approximate Scale:
 1-inch = 30 Feet (Horizontal)
 1-inch = 10 Feet (Vertical)





4/10/2025

1:500,000

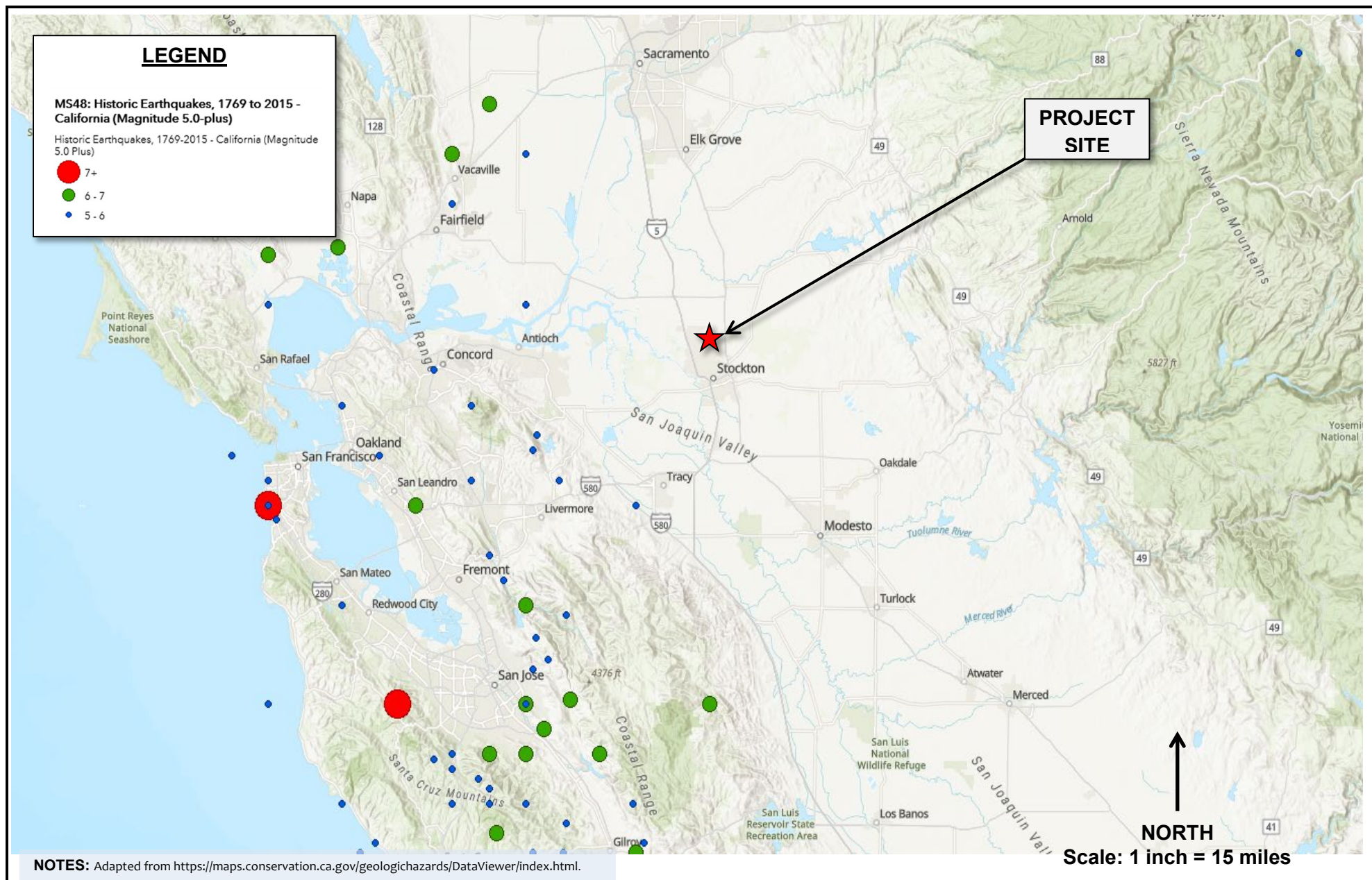


Adapted from the Fault Activity Map of California 2010. California Geological Survey. Compilation and Interpretation by C.W. Jennings and W.A. Bryant.



REGIONAL FAULT MAP
ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
 10505 Davis Road
 Stockton, California

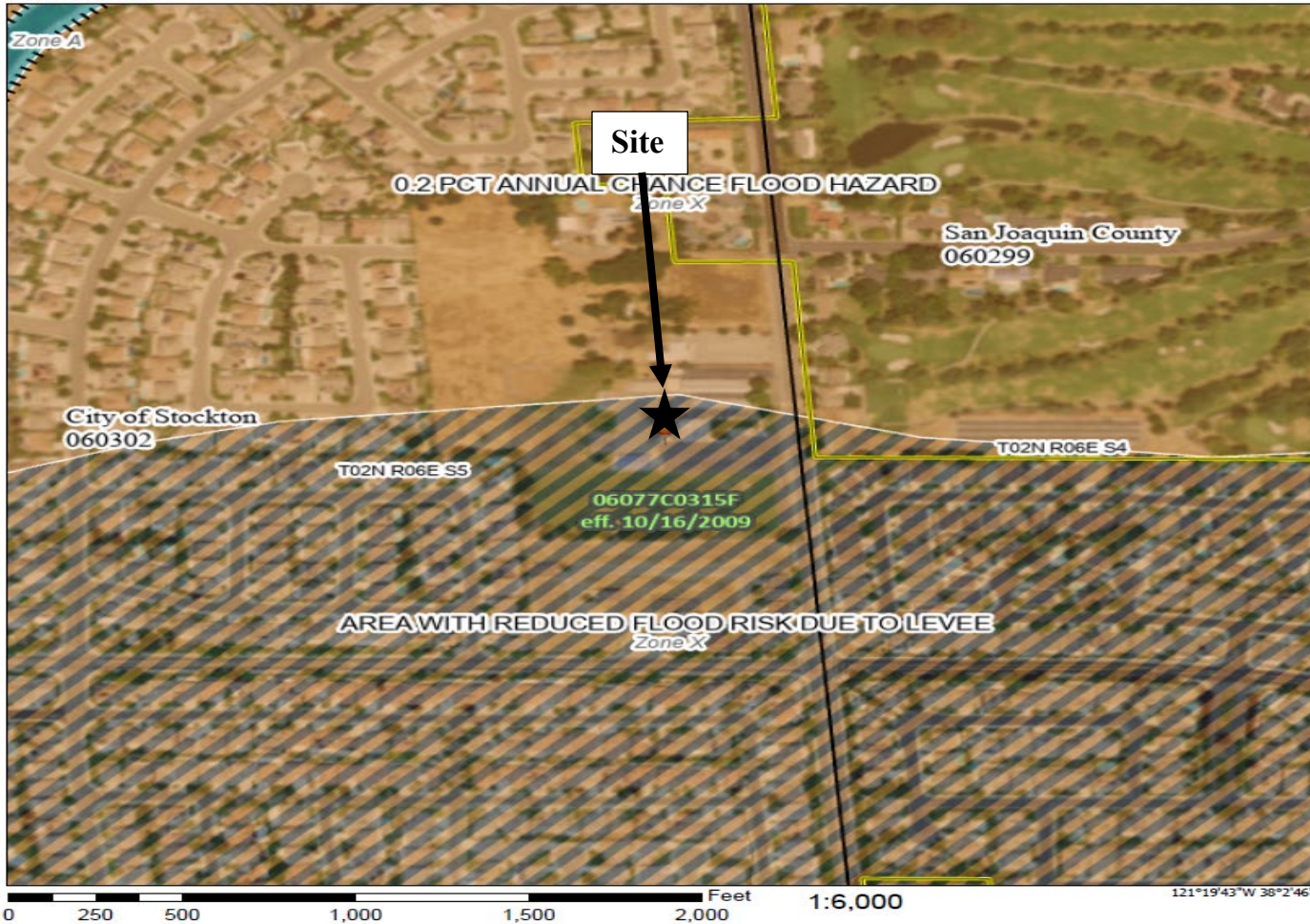
FIGURE 19
 Date: 06/25
 MPE No. 06713-03



National Flood Hazard Layer FIRMette



121°20'20"W 38°3'15"N



Legend

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

SPECIAL FLOOD HAZARD AREAS		Without Base Flood Elevation (BFE) Zone A, V, A99
		With BFE or Depth Zone AE, AO, AH, VE, AR
		Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Area of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile Zone A
		Future Conditions 1% Annual Chance Flood Hazard Zone X
		Area with Reduced Flood Risk due to Levee, See Notes, Zone X
		Area with Flood Risk due to Levee Zone D
OTHER AREAS		NO SCREEN Area of Minimal Flood Hazard Zone X
		Effective LOMRs
GENERAL STRUCTURES		Area of Undetermined Flood Hazard Zone
		Channel, Culvert, or Storm Sewer
OTHER FEATURES		Levee, Dike, or Floodwall
		Cross Sections with 1% Annual Chance Water Surface Elevation
MAP PANELS		Coastal Transect
		Base Flood Elevation Line (BFE)
		Limit of Study
		Jurisdiction Boundary
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
		Digital Data Available
		No Digital Data Available
		Unmapped



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 3/22/2023 at 5:58 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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

FLOOD HAZARDS

ZONE X – Area with Reduced Flood Risk due to the Levee

ZONE X – Area with 0.2% Annual Chance Flood Hazard

Adapted from: Federal Emergency Management Agency (FEMA), Flood Insurance Rate Map (FIRM), Map Number 06077C0315F, dated October 16, 2009



FEMA FLOOD MAP
ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING
 10505 Davis Road
 Stockton, California

FIGURE 21

Date: 06/25

MPE No. 06317-03

APPENDICES

APPENDIX A

APPENDIX A

A. GENERAL INFORMATION

The performance of a Geologic Hazards and Geotechnical Engineering Report for the proposed Elkhorn Elementary School New Classroom Building and Remodeling project to be constructed within the existing Elkhorn Elementary School campus located at 10505 Davis Road in Stockton, California, was authorized by Mr. Daniel Kramer on March 48, 2025. The authorization was for an investigation as described in our proposal letter of February 27, 2025, sent to Petralogix Engineering, Inc., whose address is 26675 Bruella Road, Galt, California 95632.

B. FIELD EXPLORATION

Two (2) cone penetration test (CPT) soundings (ASTM D5778) were completed on March 17, 2025, at the approximate locations indicated on Figure 3. The CPT soundings were advanced to the maximum depth of approximately 51 feet below ground surface (bgs).

The Logs of CPT Soundings, Figures 12 and 13, contain descriptions of the soils encountered in the soundings. Soil Behavior Charts explaining the symbols used on the CPT logs are contained on Figure 14. Measured in the CPT soundings Shear Wave Velocities are contained on Figure 15.

On March 19 and 21, 2025, seven (7) soil borings were drilled at the approximate locations indicated on Figure 3, utilizing a CME-55 HT, track-mounted drill rig equipped with 5-inch O.D. solid flight augers or 7-inch O.D. hollow stem augers to the maximum depth of 51½ feet bgs.

At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D. Modified California sampler (ASTM D3550), 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 were retained in 2-inch diameter by 6-inch long, thin-walled brass tubes contained within the sampler. Immediately after recovery, the field engineer visually classified the soil in the tubes. The ends of the tubes were sealed 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 10, 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 11.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216), and unconfined compressive strength (ASTM 2166). 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 the tests are presented on Figures A1 and A2.

One bulk sample of the anticipated pavement subgrade soils, mixed with 4 percent of lime by dry unit weight, 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 the 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, lean clay (CL)
Location: D7 (0 to 1 feet)

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
D7	12.1	26.7	89.3	94

CLASSIFICATION OF EXPANSIVE SOIL

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



EXPANSION INDEX TEST RESULTS
ELKHORN ELEMENTARY SCHOOL NEW
CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California

FIGURE A1
Date: 06/25
MPE No. 06317-03

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

Material Description: Dark Brown, sandy lean clay (CL)
Location: D6 (0 to 2 feet)

Sample Number	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
D6	11.4	22.9	107	62

CLASSIFICATION OF EXPANSIVE SOIL

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

<i>MPE</i>	<u>EXPANSION INDEX TEST RESULTS</u> ELKHORN ELEMENTARY SCHOOL NEW CLASSROOM BUILDING AND REMODELING 10505 Davis Road Stockton, California	<u>FIGURE A2</u> Date: 06/25 MPE No. 06317-03
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RESISTANCE VALUE TEST RESULTS
(California Test 301)

Material Description: Gray-Brown, lean clay (CL) treated with 4% of lime by dry weight

Location: D3 (1 to 3 feet)

Specimen No.	Dry Unit Weight (pcf)	Moisture at Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
1	93.6	23.9	97	0	28
2	101.4	21.2	202	0	64
3	100.4	18.3	374	0	86

Resistance-value @ 300 psi = 77

MPE

RESISTANCE VALUE TEST RESULTS
ELKHORN ELEMENTARY SCHOOL NEW
CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California

FIGURE A3

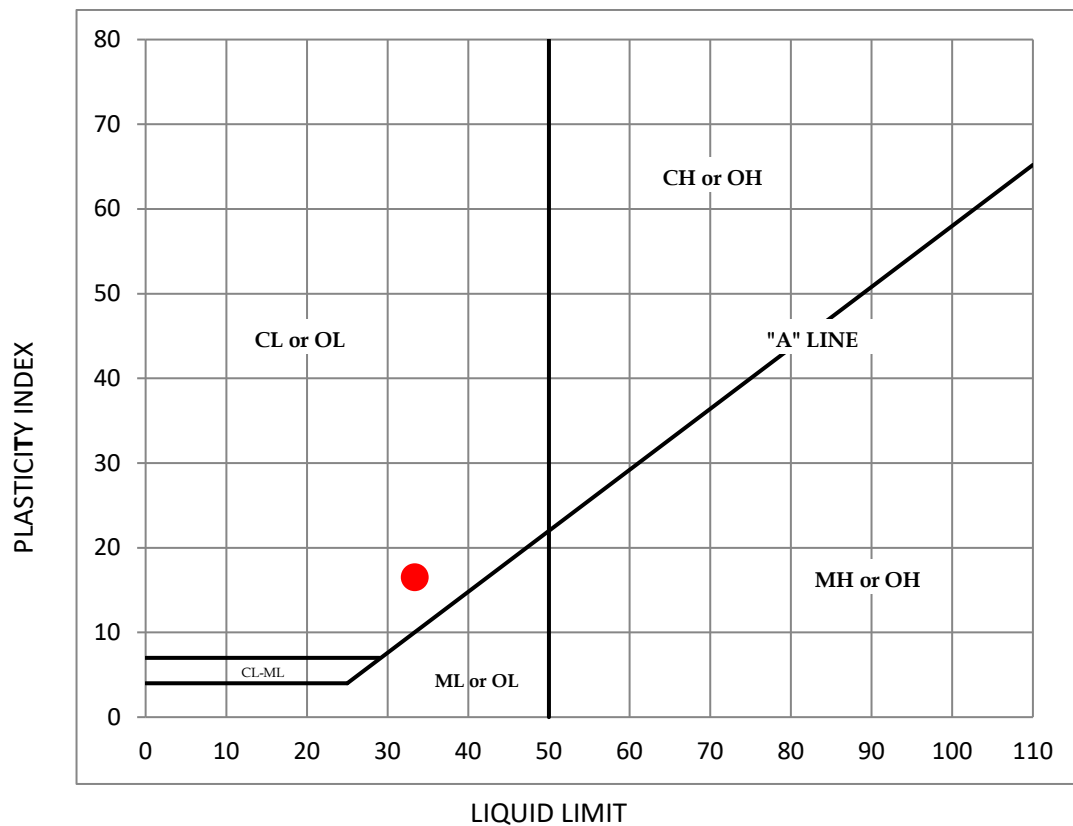
Date: 06/25

MPE No. 06317-03

ATTERBERG LIMITS (ASTM D4318)

Symbol	Sample & Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
●	D6 (0' – 2')	33	16	17

PLASTICITY CHART



MPE

LABORATORY TEST RESULTS
ELKHORN ELEMENTARY SCHOOL NEW
CLASSROOM BUILDING AND REMODELING
 10505 Davis Road
 Stockton, California

FIGURE A4

Date: 06/25
 MPE No. 06317-03

APPENDIX B

APPENDIX B
GUIDE EARTHWORK SPECIFICATIONS
ELKHORN ELEMENTARY SCHOOL
NEW CLASSROOM BUILDING AND REMODELING
10505 Davis Road
Stockton, California
MPE No. 06317-03

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. 06317-03; dated June 6, 2025) has been prepared for this site by Mid Pacific Engineering, Inc., Geotechnical Engineers. A copy is available for review at the offices of Mid Pacific Engineering, Inc., 840 Embarcadero Drive, Suite 20, West Sacramento, California 95605, and 8910 East Victor Road, Lodi, California 95240.
- 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.
- 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. Materials to be lime stabilized shall be on-site clayey soils free from significant quantities of rubble, rubbish and vegetation and shall have been tested and approved by the Geotechnical Engineer.
- D. Lime shall be high-calcium or dolomitic quicklime conforming to Caltrans Section 24 and the definitions in ASTM Designation C977, as applicable. The burden of proof as to quality and suitability of all products used for stabilization shall be upon the Contractor and Supplier and they shall furnish test data and all information necessary as required by the Geotechnical Engineer. Lime from more than one source may be used on the same project but different limes shall not be used or mixed. The lime shall be protected from moisture until used and shall be sufficiently dry to flow freely when handled.
- E. Water for use in subgrade stabilization shall be clean and potable and shall be added during mixing, remixing and compaction operations, and during the curing period to keep the cured material moist until covered.
- F. 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, utilities to be relocated or abandoned including all associated backfill, demolition debris, rubbish, rubble and other unsuitable materials. Subsurface utilities to be relocated or abandoned shall be removed from within and to at least five feet beyond the perimeter of the proposed structural areas; remaining piping beyond the structure that is not removed shall be plugged. Trees and shrubs designated to be removed shall include the entire rootball and all roots larger than one-half inch ($\frac{1}{2}$ ") 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 undocumented fill soils 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 utilities 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 and debris shall be removed from the subgrades. Hand picking of exposed rubble and debris shall be performed by the Contractor to adequately clear the grades.
- D. The entire building pad shall be over-excavated to a depth equal to the bottom of the proposed foundations, or two feet (2') below existing site grades, whichever is deeper.
- E. 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.

- F. 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.
- G. 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.
- H. 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.
- I. 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. Loose, soft, and/or 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.

3.4 LIME-STABILIZED SUBGRADE CONSTRUCTION

- A. Lime-stabilized materials and construction shall be in general conformance with Caltrans Section 24 and the following requirements. In the event of conflict between project requirements, the more stringent requirement shall govern, or as determined by the Geotechnical Engineer. Contract prices and payments shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals for doing all the work involved in constructing the lime-treated subgrade, complete in place, as shown on the

plans, specified in these specifications and the special provisions, and as directed by the Geotechnical Engineer.

- B. Initial mixing of lime should be followed by remixing the next day. Additional remix passes should be performed to provide a uniform soil-lime mixture.
- C. Compaction operations should be undertaken with a heavy, self-propelled, compactor and should be performed under the presence of our representative who will evaluate the performance of the subgrade under compactive load.
- D. The material to be treated shall be placed at a moisture content at least two percent (2%) over optimum moisture and uniformly compacted to at least ninety-two percent (92%) of maximum dry density, as defined by the ASTM D1557 Compaction Test for building pads and flatwork subgrades and ninety-five percent (95%) for pavement subgrade areas.
- E. Lime treated subgrades shall be properly cured. No equipment or vehicle traffic should be allowed on the lime treated materials during the first three days after treatment and compaction are completed.

3.5 FINAL SUBGRADE PREPARATION

- A. The upper twelve inches (12") of final building pads and final exterior flatwork subgrades shall consist of imported non-expansive, granular soils, aggregate base, or lime-treated on-site clayey soils.
- B. The upper twelve inches (12") of final building pad and exterior flatwork subgrades (imported non-expansive, granular soils, aggregates, or treated clayey soils) should be brought to a uniform moisture content of at least the optimum moisture content for imports and aggregates, and two percent (2%) above the optimum moisture content for lime treated clayey soils, and shall be uniformly compacted to not less than least ninety percent (90%) relative compaction (imported non-expansive, granular soils, aggregates) or not less

than least ninety-two percent (92%) relative compaction (lime-treated clayey soils).

- C. The upper twelve inches (12") of lime-treated pavement subgrades should be uniformly compacted to not less than ninety-two percent (92%) of the maximum dry density, as determined by ASTM D1557, for building pads and flatwork subgrades and ninety-five percent (95%) for pavement subgrade areas.
- D. For untreated pavement subgrades and exterior slab subgrades supporting vehicle loadings, the upper six inches (6") of final subgrades supporting pavement sections shall be brought to a uniform moisture content of at least two percent above the optimum moisture content and shall be uniformly compacted to at least ninety-five percent (95%) relative compaction, regardless of whether final subgrade elevations are attained by filling, excavation, or are left at existing grades.
- E. Pavement subgrades shall be proof-rolled in the presence of the Geotechnical Engineer prior to placement of aggregate base and shall be stable under construction equipment traffic.

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 City of Vacaville Standards, latest editions. The upper six inches (6") in pavement areas, the minimum compaction should be ninety-five (95%) percent of ASTM D1557. The upper 12 inches (12") of trench backfill in lime-treated structural areas (i.e. building pads, exterior flatwork, pavements) or to the depth of lime-treatment should consist of ninety-five percent (95%) compacted material Class 2 aggregate base.

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.

APPENDIX C

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

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 06317-03

DATE: 04-20-2025

JOB NAME: Elkhorn ES School New Classroom Building

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:

MINIMUM MAGNITUDE: 5.00

MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:

SITE LATITUDE: 38.0506

SITE LONGITUDE: 121.3344

SEARCH DATES:

START DATE: 1800

END DATE: 2021

SEARCH RADIUS:

62.0 mi

99.8 km

ATTENUATION RELATION: 3) Boore et al. (1997) Horiz. - NEHRP D (250)

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: 1.50 km Campbell SSR: Campbell SHR:

COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 0.0

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
BRK	37.7600	121.7300	01/27/1980	23336.0	0.0	5.40	0.061	VI	29.4 (47.4)
BRK	37.8100	121.7900	01/24/1980	19 1 2.0	0.0	5.10	0.051	VI	29.9 (48.0)
BRK	37.8300	121.8100	01/24/1980	19 0 9.0	0.0	5.80	0.074	VII	30.0 (48.3)
DMG	38.0000	121.9000	05/19/1889	1110 0.0	0.0	6.00	0.080	VII	31.0 (49.8)
DMG	38.4000	121.8000	04/30/1892	0 9 0.0	0.0	5.50	0.056	VI	34.9 (56.2)
DMG	38.3000	121.9000	05/19/1902	1831 0.0	0.0	5.50	0.056	VI	35.2 (56.6)
DMG	37.5000	121.3000	07/15/1866	630 0.0	0.0	5.80	0.061	VI	38.1 (61.2)
DMG	37.9700	122.0500	10/24/1955	41044.0	0.0	5.40	0.049	VI	39.3 (63.3)
DMG	37.6000	121.8000	06/11/1903	1312 0.0	0.0	5.50	0.050	VI	40.2 (64.6)
DMG	37.8000	122.0000	07/04/1861	011 0.0	0.0	5.60	0.053	VI	40.2 (64.6)
DMG	38.4000	122.0000	04/19/1892	1050 0.0	0.0	6.40	0.076	VII	43.4 (69.9)
DMG	37.7000	122.0000	03/05/1864	1649 0.0	0.0	5.70	0.053	VI	43.6 (70.2)
DMG	38.5000	121.9000	04/21/1892	1743 0.0	0.0	6.20	0.068	VI	43.6 (70.2)
GSB	37.4830	121.6900	03/31/1986	115540.0	8.0	5.70	0.052	VI	43.7 (70.4)
DMG	37.4000	121.4000	04/10/1881	10 0 0.0	0.0	5.90	0.057	VI	45.1 (72.5)
DMG	37.7000	122.1000	10/21/1868	1553 0.0	0.0	6.80	0.087	VII	48.2 (77.6)
GSB	37.4340	121.7740	10/31/2007	030454.8	10.0	5.50	0.043	VI	48.9 (78.6)
DMG	37.5000	121.9000	11/26/1858	835 0.0	0.0	6.10	0.059	VI	49.0 (78.8)
DMG	37.8000	122.2000	07/31/1889	1247 0.0	0.0	5.20	0.036	V	50.2 (80.8)
DMG	37.8000	122.2000	06/10/1836	1530 0.0	0.0	6.80	0.084	VII	50.2 (80.8)
MGI	37.8500	122.2500	09/10/1935	2355 0.0	0.0	5.00	0.032	V	51.7 (83.3)
GSB	37.3850	121.7720	06/13/1988	014536.8	7.0	5.40	0.039	V	51.8 (83.4)
DMG	37.6000	122.1000	05/21/1864	2 1 0.0	0.0	5.30	0.037	V	52.1 (83.8)
DMG	37.3700	121.7800	09/05/1955	2 118.0	0.0	5.50	0.041	V	52.9 (85.2)
DMG	37.9000	122.3000	04/02/1870	1948 0.0	0.0	5.30	0.036	V	53.6 (86.2)
GSB	37.3200	121.6980	04/24/1984	211519.0	8.0	6.20	0.058	VI	54.2 (87.2)
GSB	38.2152	122.3123	08/24/2014	102044.1	11.1	6.02	0.052	VI	54.3 (87.4)
DMG	37.3000	121.8000	08/03/1903	649 0.0	0.0	5.50	0.038	V	57.7 (92.9)
DMG	37.3000	121.8000	01/02/1891	20 0 0.0	0.0	5.50	0.038	V	57.7 (92.9)
DMG	38.2000	122.4000	03/31/1898	743 0.0	0.0	6.20	0.054	VI	58.8 (94.6)
T-A	38.0000	120.2500	04/11/1872	12 0 0.0	0.0	5.00	0.029	V	59.1 (95.1)
DMG	37.2000	121.5000	07/06/1899	2010 0.0	0.0	5.80	0.044	VI	59.4 (95.6)
DMG	37.2500	121.7500	07/01/1911	22 0 0.0	0.0	6.60	0.066	VI	59.8 (96.2)
MGI	37.3000	121.9000	05/28/1927	1739 0.0	0.0	5.00	0.028	V	60.3 (97.1)
DMG	37.3000	121.9000	10/08/1865	2046 0.0	0.0	6.30	0.056	VI	60.3 (97.1)
DMG	38.3000	122.4000	10/12/1891	628 0.0	0.0	5.50	0.037	V	60.3 (97.1)
MGI	37.8000	122.4000	10/05/1859	2016 0.0	0.0	5.00	0.028	V	60.6 (97.5)
MGI	37.8000	122.4000	05/15/1851	1610 0.0	0.0	5.00	0.028	V	60.6 (97.5)

-END OF SEARCH- 38 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2021

LENGTH OF SEARCH TIME: 222 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 29.4 MILES (47.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.8

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.087 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

a-value= 0.334

b-value= 0.248

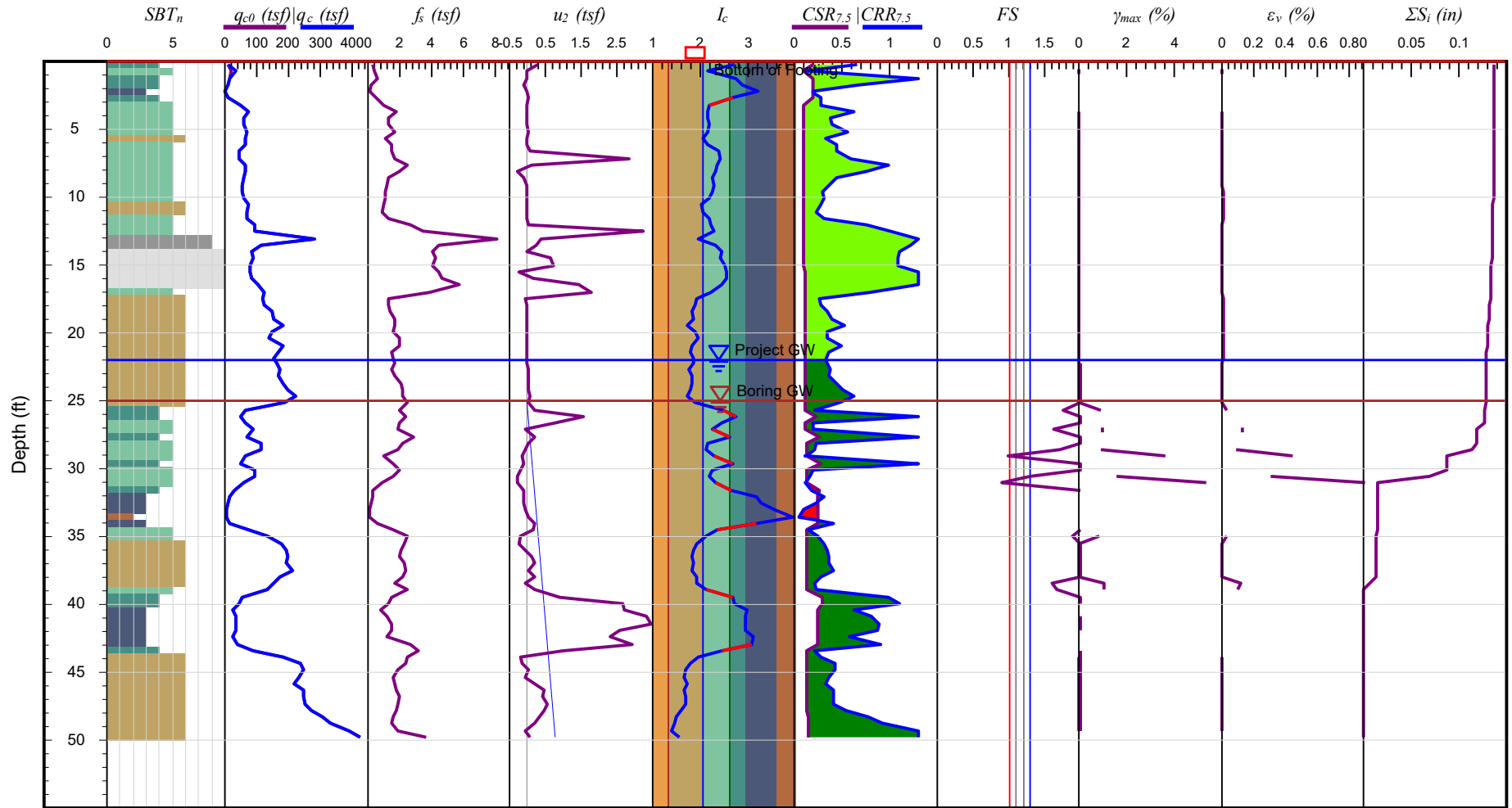
beta-value= 0.572

 TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	38	0.17117
4.5	38	0.17117
5.0	38	0.17117
5.5	26	0.11712
6.0	11	0.04955
6.5	3	0.01351

APPENDIX D

R:\06317-03 through 06317-03 - Elkhorn ES GPRV - GER & Correspondence>Data & Analysis\Geosuite\GeoSuite_06317-03_CPT1.cav

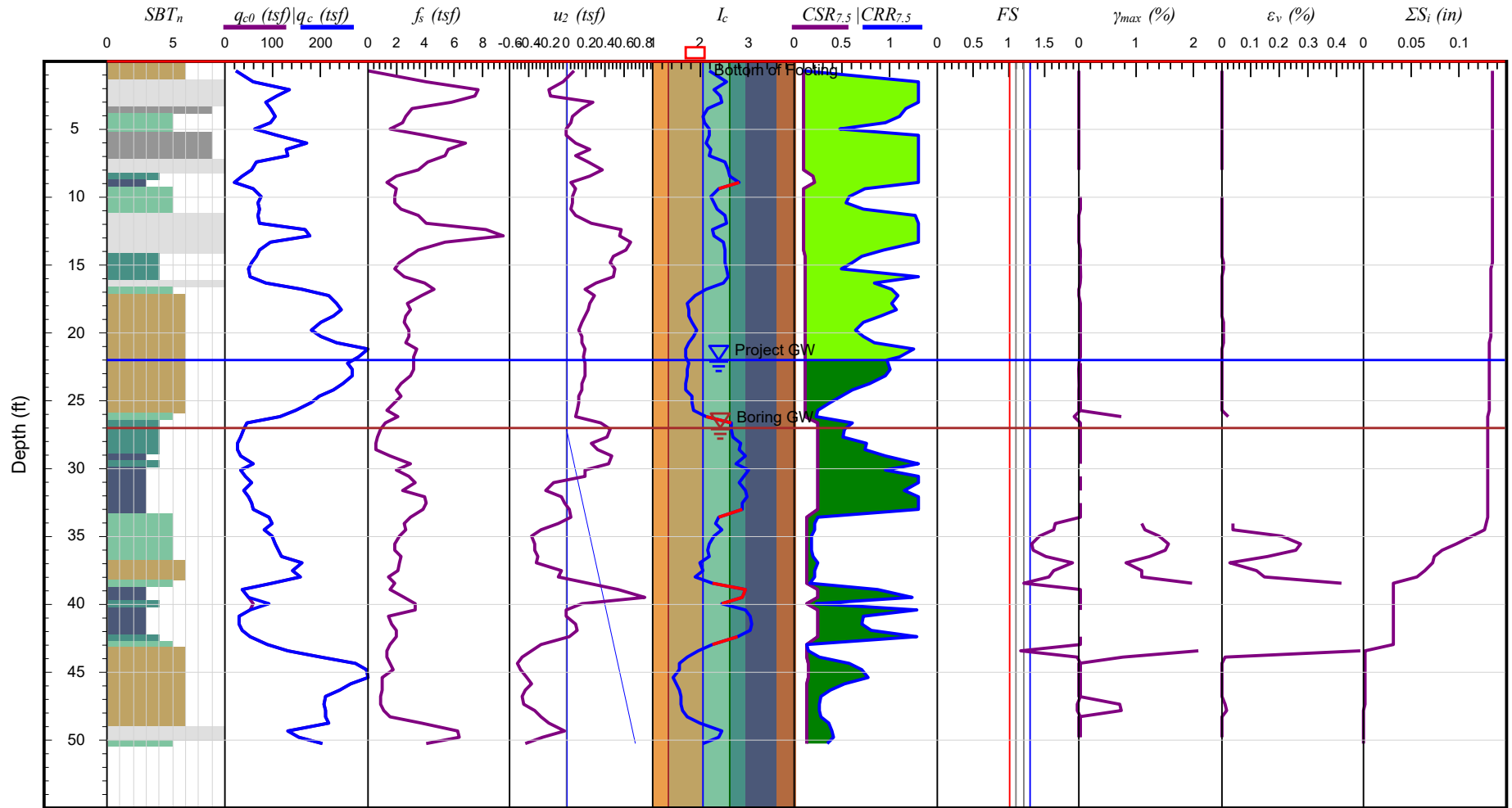


GeoSuite©+GeoLogs©+GeoPile©

Seismic Settlement Potential - CPT Data

Project:	Elkhorn Elementary School				
Location:	10505 Davis Road, Stockton, CA 95209				
Project No.:	06317-03	CPT No.:	CPT-1	Figure:	

R:\06317-03 through 06317-03 - Elkhorn ES GPRV - GER & Correspondence>Data & Analysis\GeoSuite\GeoSuite_06317-03_CPT2.cav



- Sensitive fine grained
- Organic soils - peats
- Clay to silty clay
- Silty clay to clayey silt
- Sandy silt to silty sand
- Silty sand to clean sand
- Dense sand to gravelly sand
- Clayey sand to very stiff sand

■ Very stiff fine grained *

* Overconsolidated or cemented

Silt Correction:
UCLA method

Earthquake & Groundwater Information:
Magnitude = 5.5
Max. Acceleration = 0.385 g
Project GW = 22 ft
Max. Seismic Settlement = 0.14 in
Settl. @ Bottom of Footing = 0.14 in

Liquefaction: Robertson (2009)
Settl.: [dry] Yi (2022)[CPT]; [sat] Robertson (2009)
Lateral spreading: Zhang et al (2004)
M correction:
sigma_v correction: Idriss & Boulanger (2008)
Stress reduction: Blake (1996)

GeoSuite©+GeoLogs©+GeoPile©

Seismic Settlement Potential - CPT Data

Project:	Elkhorn Elementary School				
Location:	10505 Davis Road, Stockton, CA 95209				
Project No.:	06317-03	CPT No.:	CPT-2	Figure:	

APPENDIX E

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