Geotechnical Engineering and Geologic Hazards Report

AMERICAN RIVER COLLEGE CORPORATION YARD

WKA No. 12456.01P

October 3, 2019

Prepared for:

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Facility Management
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INTRODUCTION

We have completed a geotechnical engineering investigation and geologic hazards study for the planned corporation yard to be constructed in the southwest portion of the American River College campus, located at 4700 College Oak Drive in Sacramento, California (see Figure 1). The purposes of our study have been to explore the existing site, soil and groundwater conditions at the site and to provide geologic hazards and geotechnical engineering conclusions and recommendations regarding the design and construction of the planned corporation yard. This report represents the results of our study.

Scope of Services

Our scope of services for this study has included the following tasks:

1. Site reconnaissance;
2. Review of topographic, geologic, and fault maps, historical aerial photographs, and available groundwater information relevant to the site;
3. Review of seismic activity within 100 kilometers (62 miles) of the site;
4. Subsurface exploration, including the drilling and sampling six borings to depths ranging from about 10 to 51½ feet below existing site grades;
5. Collection of representative bulk samples consisting of near-surface soils;
6. Laboratory testing of selected soil samples;
7. Engineering analyses, and;
8. Preparation this report.

Figures and Attachments

The following figures are included with this report:

Table 1: Figures

Figure	Title	Figure	Title
1	Vicinity Map	6 – 11	Boring Logs
2	Site Plan	12	Unified Soil Classification System
3	Topographic Map	13	Fault Map
4	Geologic Map	14	Epicenter Map
5	Geologic Cross Section	15	FEMA Flood Map

Appended to this report are:

- Appendix A, which contains a list of references cited in this study;
- Appendix B, which contains descriptions of the field exploration and laboratory testing programs, and the results of laboratory tests;
- Appendix C, which contains *Guide Earthwork Specifications* that may be used in the preparation of contract documents.

Proposed Development

Our review of a *Site Plan*, dated May 28, 2019, prepared by GRA Architecture (project architect), indicates the project will consist of complete demolition, removal, and clearing of an existing parking lot located within the planned footprint of a corporation yard building and associated improvement areas. Following site clearing operations, new development will include construction of a single-story corporation yard building. Based on information provided by Mr. Michael Buschow of GRA Architecture, the estimated total footprint of the planned building is approximately 24,000-square feet. Ms. Julie Moore of the Los Rios Community College District Facilities department informed us the building will be steel-framed with concrete masonry unit walls and an interior concrete slab-on-grade floor. Structural loads for the building are anticipated to be relatively light to moderately heavy based on this type of construction. Associated improvements will include construction of underground utilities, landscaping, below-grade loading docks, trash enclosures, covered storage areas, exterior flatwork, and new asphalt concrete pavements.

A grading plan was not available for review at the time this report was prepared; however, based on the existing site topography, our understanding of the planned development and existing subsurface soil conditions, we anticipate cuts and fills on the order of one to three feet will be required for development of the site, with the exception of excavations required for construction of underground utilities and below-grade loading docks. Excavations ranging from about three to 10 feet deep are anticipated to establish final subgrade levels within those areas.

FINDINGS

Site Description

The site is located in the southwest portion of the American River College campus (see Figure 2). The site is identified as a portion of Sacramento County Assessor's Parcel Number 240-0070-006. The site is bounded to the north and east by asphalt concrete pavements; to the south by Winding Way; and, to the west by College Oak Drive.

At the time our field explorations were performed, August 22, 2019, the site supported a asphalt concrete pavements associated with an existing parking lot and landscaped areas, including several mature trees and a sparse growth of grasses. The central portion of the site supported rectangular-shaped chain-link fencing. The area enclosed by the fencing was used as a contractor parking/staging area during this time for other work being performed in a different area of the college campus.

The site generally consists of flat topography with a surface elevation of about +80 feet (North American Vertical Datum of 1988 [NAVD 88]), based on topographic information shown on the United States Geological Survey (USGS) 7.5-Minute Series topographic map of the Citrus Height, California Quadrangle, dated 1992. A portion of the USGS topographic map showing the site is presented as Figure 3.

Historical Aerial Photograph Review

We reviewed historical aerial photographs of the site available from the Historic Aerials website (HA, 2019) and Google Earth software (GE, 2019). Available photographs were taken in the years 1947, 1957, 1964, 1966, 1993, 1998, 2002 through 2016, and 2018. A summary of our photograph review is presented below.

- The photographs taken in 1947, 1957 and 1964 show the site to be vacant of structures and covered with vegetation. Several mature trees were observed within the site.
- The photographs taken in 1966 and 1993 show asphalt concrete pavements in the north portion of the site. The south portion of the site remains vacant of structures and covered with vegetation.
- The remaining photographs show asphalt concrete pavements throughout the entire site, as observed during the time our field explorations were performed.

Site Geology

The project site is located within the Great Valley geomorphic province of California. The Great Valley of California is generally considered to be an elongated sedimentary trough, approximately 450 miles long and 50 miles wide. Rock units within the Great Valley geomorphic province consist of Mesozoic to Cenozoic marine and non-marine sedimentary rocks. These sediments have been folded into an asymmetric syncline, the axis of which lies immediately east of the interior Coast Ranges. The sedimentary units on the east side of the Great Valley are minimally deformed and are deposited on basement rocks of the Sierra Nevada geomorphic province. The sedimentary rocks on the west side of the Great Valley are deformed at dip at moderate angles to the east (Norris and Webb, 1990).

Surface elevations within the Great Valley generally range from several feet below mean sea level to more than 1,000 feet above sea level. The major topographical feature in the Great Valley is the Sutter Buttes (a volcanic remnant) that rise approximately 1980 feet above the surrounding valley floor.

According to the USGS *Geologic Map of the Sacramento Quadrangle* (Wagner, 1981), the project site is underlain by the Quaternary-aged Levee and Channel Deposits (Qa). The California Geological Survey's (CGS) *Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle, California* (Gutierrez, 2011) identifies the area underlying the site as Holocene Alluvium Deposits (Qha). The geologic materials that comprise both the Levee and Channel Deposits and Alluvium Deposits are primarily sands, silts, and gravels and were likely deposited by Arcade Creek located 100 to 300 feet south of the college campus. The alluvial materials are likely underlain by older Pleistocene-aged alluvium of the Turlock Lake formation. The mapped geology was found to be generally consistent with the subsurface soil conditions encountered within the borings performed at the site to the approximate depth explored of 51½ feet below site grades.

A portion of the 2011 *Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle, California* is shown in Figure 4. A geologic cross section of the site is included in this report as Figure 5.

Subsurface Soil Conditions

On August 22, 2019 six borings (D1 through D6) were drilled and sampled at the site to depths ranging from about 10 to 51½ feet below existing site grades. The approximate locations of these borings are shown on the Site Plan, Figure 2.

The borings were drilled within asphalt concrete pavement areas. The thickness of the asphalt concrete at the boring locations ranged from about 4 to 5 inches. The asphalt concrete was underlain by about 2 to 3 inches of aggregate base. The soils beneath the pavements generally consist of loose to medium dense, silty sand to depths ranging from about 7 to 18 feet below existing site grades. These granular soils are generally underlain by alternating layers of medium dense to dense, clayey and silty sands and sandy silt and stiff to very stiff, lean clay with varying amounts of sand to the maximum depth explored of about 51½ feet below existing site grades.

The soil conditions described above are generally consistent with the mapped geology. At the completion of our field exploration activities, with the exception of boring D5 (about 51½ feet deep), the remaining borings (about 10 to 21½ feet deep) were backfilled with the excavated soil cuttings. Boring D5 was backfilled a slurry of neat cement, bentonite, and water in accordance with Sacramento County Environmental Management Department (SCEMD) requirements. The surface of all borings (about the upper 6 inches) was restored with an asphalt cold patch. For specific information regarding the soil conditions at a specific boring location, please refer to the Logs of Soil Borings, Figures 6 through 11.

Faulting

No indication of surface rupture or fault-related surface disturbance was observed at the site during our site reconnaissance or review of aerial photographs. The site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Parrish, 2018).

Using the USGS and CGS, 2006, Quaternary fault and fold database for the United States, accessed on October 1, 2019, from USGS web site: http://earthquake.usgs.gov/hazards/qfaults/ and the *Revised 2002 California Probabilistic Seismic Maps* (Cao, et al, 2003), we have prepared Table 2 containing faults and fault systems within 120 kilometers (about 75 miles) of the site that are considered capable of producing earthquakes with moment magnitude (M_W) of 6.5 or greater. A fault map is presented as Figure 13. The nearest of these faults are associated with the Foothills Fault System, which is used to model seismic sources along the west side of the Sierra Nevada. A segment of the Foothills Fault System is located about 32.8 kilometers (20.4 miles) northeast of the site.

Table 2: Faults Influential to the Site

Fault Name	Distance		Maximum Magnitude (Mw)
	Miles	Kilometers	
Foothills Fault System	20.4	32.8	6.5
Great Valley 4a, Trout Creek	36.1	58.1	6.6
Great Valley 3, Mysterious Ridge	38.0	61.1	7.1
Great Valley 4b, Gordon Valley	38.7	62.2	6.8
Great Valley 5, Pittsburg Kirby Hills	42.2	67.8	6.7
Hunting Creek-Berryessa	47.8	77.0	7.1
Green Valley Connected	48.5	78.1	6.8
West Napa	57.5	92.5	6.7
Greenville Connected	58.7	94.5	7.0
Great Valley 2	59.4	95.6	6.5
Great Valley 7	63.5	102.2	6.7
Mount Diablo Thrust	64.9	104.5	6.6
Bartlett Springs	65.5	105.4	7.1
Great Valley 1	66.9	107.6	6.7
Calaveras; CN	67.5	108.6	6.8
Calaveras: CN + CC	67.5	108.6	6.9
Calaveras: CN + CC + CS	67.5	108.6	6.9
Hayward-Rodgers Creek; RC+HN	70.3	113.1	7.1
Hayward-Rodgers Creek; RC	70.3	113.1	7.0
Hayward-Rodgers Creek; RC+HN+HS	70.3	113.1	7.3
Hayward-Rodgers Creek; HN + HS	70.4	113.3	6.9
Hayward-Rodgers Creek; HN	70.4	113.3	6.5
West Tahoe	71.6	115.2	7.3
Collayomi	72.6	116.9	6.5
Maacama-Garberville	72.9	117.6	7.5
Hayward-Rodgers Creek; HS	73.4	118.2	6.7

The CGS defines an active fault as one that has had surface displacement within Holocene time (the past 11,700 years). In addition to the faults listed above, review of the *Fault Activity Map of California* (Jennings, 2010) indicates there are two other active faults located within a 100 kilometer (about 62 miles) radius of the site. An unnamed fault associated with the Dunnigan Hills fault zone which is modeled as part of the Great Valley Fault System, and is located about 35 miles northwest of the site. The Cleveland Hill fault is located about 50 miles north of the site and is part of the northern end of the Foothills Fault system.

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 (CGS, 2008). We are not aware of any mapping of geologic hazards for this area based on review of currently published maps available at the CGS website.

Historic Seismicity

Seismological data regarding significant historical earthquakes affecting the site was obtained using the commercially available software program EQSEARCH (Blake, 2000; database updated to August of 2018). The EQSEARCH database was developed by extracting records of events greater than magnitude 4.0 from the Division of Mine and Geology 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 100 kilometers (about 62 miles) was specified for this analysis. A historic earthquake epicenter map is presented as Figure 14.

An examination of the tabulated data suggests that the site has experienced ground shaking equivalent to Modified Mercalli Intensity (MMI) VII[1]. According to the tabulated data, the most intense earthquake ground shaking within 100 kilometers (about 62 miles) of the site resulted from an M_R 6.4 earthquake on April 19, 1892, with an epicenter located approximately 63 kilometers (about 39 miles) southwest of the site. The August 24, 2014, Napa Valley earthquake epicenter has been located approximately 95 kilometers (about 60 miles) southwest of the site and produced a M_W of 6.0.

Groundwater

Groundwater was not observed during the drilling operations performed at the site on August 20, 2019. To supplement our groundwater data, we reviewed available groundwater information at the California Department of Water Resources (DWR) website. The DWR periodically monitors groundwater levels in wells across the state. Their website shows a well located approximately 1¼-mile northwest of the site. The well is identified as Well No. 386484N1213715W001 with a ground surface elevation of about +83 feet NAVD 88, similar to the project site. Groundwater data for this well was recorded from March 13, 1978 to at least November 25, 2003. Data shows the

[1] Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.

highest recorded groundwater elevation was about -7 feet NAVD 88 (about 90 feet below the ground surface at the well location) at the well on March 13, 1978. The lowest recorded groundwater elevation was about -81 feet NAVD 88 (about 164 feet below the ground surface at the well location) at the well on April 14, 2003. Based on review of this information, groundwater at the site during this time fluctuated from about 87 to 161 feet below existing site grades.

We also reviewed available groundwater information at the DWR *Groundwater Information Center Interactive Map Application* website. The interactive map application shows groundwater elevations for the Central Valley of California based on groundwater level data collected from wells monitored by the DWR. Based on review of the available groundwater level contour maps, dated Spring of 2011 through Fall of 2018, groundwater at the site during this time fluctuated from about 110 to 135 feet below existing site grades.

Based on the available groundwater data pertinent to the site, for the purposes of this study we have assumed the historical high groundwater at the site to be at an elevation of about -7 feet NAVD 88, corresponding to a depth of about 87 feet below existing site grades at the site.

CONCLUSIONS

Seismic Hazards

No active or potentially active faults are known to underlie the project site based on the published geologic maps or aerial photographs that we reviewed. The site is not located within an Alquist-Priolo Earthquake Fault Zone or a seismic hazard zone pursuant to the Seismic Hazard Zone Mapping Act, and we observed no surface evidence of faulting during our site reconnaissance. Therefore, it is our opinion that ground rupture at the site resulting from seismic activity is unlikely.

Volcanic Hazards

The project site is not located within a volcanic hazard zone (e.g., pyroclastic flow, volcanic debris flow, lava flow, base surge, tephra, etc.) associated with potential volcanic eruptions of Mt. Shasta, Clear Lake, Lassen Peak or the Mono Lake - Long Valley Volcanic areas (Miller, 1989). Therefore, the risk to the site associated with volcanic hazards is very low.

Landslides

Based on visual observation of the site and review of topographic data, the topography across the site is relatively flat. The stream channel for Arcade Creek is located more than 100 feet south of the site, south of Winding Way. Based on the relatively flat site topography and the nonexistence of significant slopes within the site, it is our opinion that the potential for landslides is nonexistent.

Flood Hazards

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map number 06067C0088H, dated August 16, 2012, prepared for Sacramento County, California (FEMA, 2012), the major portion of the site is located within Zone X, which is described as "0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile" (see Figure 15).

Dam Inundation

The Health and Safety Element of the Sacramento County General Plan (Sacramento County, 2011) identifies four major dams and two minor dams that have the potential for human injury or loss of life in the county if failure were to occur. The site lies approximately 11 miles southwest of the Folsom Dam, which is within an area likely to be affected by failure of Folsom Dam. The California Office of Emergency Services indicated that the site also would be affected by the failure of the Folsom, Nimbus, and Oroville Dams.

Tsunamis and Seiches

The site is not covered by the publically available "Tsunami Inundation" maps developed by the CGS. Due to the fact that the site is not located near a coastal region or near a large body of standing water, we consider the occurrence of tsunamis or seiches to be unlikely.

Subsidence and Hydrocollapse

Subsidence occurs when a large land area settles due to the extensive withdrawal of groundwater, oil, natural gas or oxidation of peat. Based on subsurface sampling and the mapped geology, the subsurface soil conditions underlying the site generally consists of alternating layers of silty sand, clayey sand, sandy silt and lean clay with varying amounts of sand to the maximum depth explored of about 51½ feet below existing site grades. The Department of Water Resources (DWR) has mapped the entire Central Valley of California as having potential (low to high) for future land subsidence; however, DWR indicates the mapping is intended to be advisory only to assist state and local agencies in defining areas of potential subsidence that may require additional study. Based on the subsurface conditions encountered at the site, it is our opinion that settlement at the site due to subsidence is very unlikely, provided the recommendations of this report are carefully followed.

Naturally Occurring Asbestos (NOA)

Review of geologic maps indicates that the site is directly underlain by Holocene Alluvium Deposits (Qha), which primarily consist of sands, silts, and gravels. Review of *A General Location Guide* *for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos*, CGS Open-File Report 2000-19 (Churchill and Hill, 2000) revealed the site is not underlain by ultramafic rocks likely to contain asbestos. Therefore, It is our opinion that the existence of NOA at the site is unlikely.

Radon-222 Gas

Radon is a naturally occurring radioactive gas that is produced from radioactive decay of uranium and thorium, most abundant in coastal marine sedimentary rocks and felsic, granitic and volcanic rocks. *Geologic Controls on the Distribution of Radon in California* (Churchill, 1991) does not identify Sacramento County as an area containing common indicators of naturally occurring radon gas.

According to the Environmental Protection Agency's Map of Radon Zones, the site is located within Zone 3, meaning the site has a predicted average indoor screening level less than 2 picocuries per liter (pCi/L) of air. Therefore, there is a low potential for radon gas at the site. Based on the regional geology of the site and review of available data, we consider the presence of naturally occurring radon gas to be unlikely at the site.

Seismic Site Class

Based on Table 20.3-1 of *American Society of Civil Engineers (ASCE) Standard 7-10*, a seismic Site Class D applies to sites with average Standard Penetration Test (SPT) blow counts between 15 and 50 blows per foot for the upper 100 feet of the ground surface. SPT blow counts obtained within the upper 51½ feet at Boring D5 varied from 6 to 41 blows per foot. Based on the site geology, the SPT blow counts obtained within the upper 51½ feet at Boring D5, conservatively assuming that the deeper materials at Boring D5 (from depths of 51½ to 100 feet) have SPT blow counts of at least 15 blows per foot, the average SPT blow count for the upper 100 feet is at least 16 blows per foot, in accordance with Section 20.4 of *ASCE Standard 7-10*. Based on this information, it is our opinion the soils at this site can be designated as Site Class D in determining seismic design forces for this project.

2016/2019 California Building Code Seismic Design Parameters

The 2016 edition of the *California Building Code (CBC)* references *ASCE Standard 7-10* for seismic design. We understand that in January of 2020, the new 2019 *CBC* will be adopted. The 2019 *CBC* references *ASCE Standard 7-16* for seismic design. To assist with the structural design of this project, we have provided seismic design parameters for the 2016 *CBC* and the 2019 *CBC*; both sets of seismic design parameters have been determined based on the site location and the Seismic Design Maps public domain computer program developed by the Structural Engineers Association of California (SEAOC) and the California Office of Statewide

Health Planning and Development (OSHPD) (https://seismicmaps.org). Since S_1 is greater than 0.2g, the 2019 *CBC* coefficient values F_v, S_{M1}, and S_{D1} presented below in Table 1 are valid for seismic design of this project, provided the requirements in Exception Note No. 2 in Section 11.4.8 of *ASCE 7-16* apply, specifically if T (fundamental period of the building) ≤ $1.5T_S$ (S_{D1}/ S_{DS}). If not, a site-specific ground motion hazard analysis is required.

The seismic design parameters summarized below in Table 3 may be used for seismic design of the planned improvements at the site. Our firm should be given the opportunity to review project documents and confirm the parameters presented below in Table 1.

Table 3: Seismic Design Parameters

Latitude: 38.6451° N Longitude: 121.3497° W	*ASCE 7-10/7-16* Table/Figure	2016/2019 *CBC* Figure/Section/Table	Factor/ Coefficient	2016 *CBC* Values	2019 *CBC* Values
0.2-second Period MCE	Figure 22-1	Figures: 1613.3.1(1)/1613.2.1(1)	S_S	0.558 g	0.472 g
1.0 second Period MCE_R	Figure 22-2	Figures: 1613.3.1(2)/113.2.1(2)	S_1	0.264 g	0.229 g
Soil Class	Table 20.3-1	Sections: 1613.3.2/1613.2.2	Site Class	D	D
Site Coefficient	Table 11.4-1	Tables: 1613.3.3(1)/1613.2.3 (1)	F_a	1.354	1.423
Site Coefficient	Table 11.4-2	Tables: 1613.3.3(2)/1613.2.3(2)	F_v	1.871	2.142*
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equations: 16-37/16-36	S_{MS}	0.755 g	0.671 g
	Equation 11.4-2	Equations: 16-38/16-37	S_{M1}	0.495 g	0.491 g*
Design Spectral Acceleration Parameters	Equation 11.4-3	Equations: 16-39/16-38	S_{DS}	0.504 g	0.447 g
	Equation 11.4-4	Equations: 16-40/16-39	S_{D1}	0.330 g	0.327 g*
Seismic Design Category	Table 11.6-1	Tables: 1613.3.5(1)/1613.2.5(1)	Risk Category I to III	D	C
			Risk Category IV	D	D
	Table 11.6-2	Tables: 1613.3.5(2)/1613.2.5(2)	Risk Category I to IV	D	D

Notes: MCE_R = Risk-Targeted Maximum Considered Earthquake; g = gravity
* = The value is valid provided the requirements in Exception Note No. 2 in Section 11.4.8 of ASCE 7-16 are met. If not, a site-specific ground motion hazard analysis is required.

Liquefaction Potential

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 shown on liquefaction susceptibility maps published by the USGS in cooperation with the California Geological Survey (CGS). In addition, to our knowledge, there have been no reported instances of liquefaction having occurred within the greater Sacramento area during the major earthquake events of 1892 (Vacaville-Winters), 1906 (San Francisco), 1989 (Loma Prieta) and 2014 (West Napa). Based on the site geological and seismologic conditions, relatively dense and cohesive soil conditions encountered at the borings performed at the site, **the absence of historical groundwater within the upper 87 feet at the site** (based on available groundwater information pertinent to the site), and our experience in the local area, it is our opinion the potential for liquefaction at the site is very low.

Liquefaction Analysis and Seismic Induced Settlement

Using the methodology of Pradel (1998), the mapped geometric mean peak ground acceleration (PGA_M) of 0.264 g (determined in accordance with Equation 11.8-1 of *ASCE Standard 7-10*), and a mode magnitude earthquake of 6.53 (determined using the 2014 USGS National Seismic Hazzard Mapping Project [NSHMP] Probabilistic Seismic Hazard Analysis [PSHA] Interactive Deaggregation website), we estimated the potential for dry sand settlement of the upper 50 feet of the soil column at Boring D5 to be about ¼ of an inch.

Based on the relatively high penetration resistance of the subsurface soils encountered in the borings drilled and sampled at the site, we do not anticipate seismic settlements will adversely affect the performance of the improvements, provided the recommendations included in this report regarding site clearing, subgrade preparation, and engineered fill placement are carefully followed.

Effects of Existing Development on New Construction

The site is currently developed with an asphalt concrete paved parking lot and associated improvements. From a geotechnical standpoint, the most effective method of mitigating the impact of the pavements, any existing underground utilities and any other associated improvements on new construction is to completely remove all existing surface and subsurface structures within the new construction areas, including all associated backfill soils, and restoring the site to grade using properly compacted engineered fill. We have provided specific recommendations regarding surface and subsurface structure removal in this report.

Bearing Capacity and Building Support

The results of our site investigation revealed that approximately the upper three to four feet across the site are in a relatively loose condition. We anticipate the upper three to four feet across the site will be disturbed during site clearing activities, which will result in further relatively loose soil conditions. In our opinion, the upper three to four feet across the site are not considered capable of providing adequate or uniform support for the planned building and associated improvements (e.g. foundations, slab-on-grade concrete, pavements, etc.) in their current condition without experiencing significant total and/or differential settlement, which can potentially result in structural damage.

Over-excavation, processing and compaction of the exposed soils following site clearing activities in accordance with the recommendations of this report will be required so that the surface and near-surface soils are capable of supporting foundations, slab-on-grade concrete and pavements associated with the planned corporation yard. Specific recommendations to over-excavate, scarify, moisture condition, and compact the near-surface soils have been provided in the Subgrade Preparation section of this report.

Based on our field investigation and laboratory test results, it is our opinion the undisturbed, denser soils underlying the relatively loose near-surface soils and engineered fill, properly placed and compacted in accordance with the recommendations of this report, are capable of supporting structural improvements (e.g. foundations, slab-on-grade concrete, pavements, etc.) associated with the planned corporation yard, provided the recommendations included in this report regarding site clearing, subgrade preparation, and engineered fill placement are carefully followed.

Soil Expansion Potential

Particle size distribution laboratory testing revealed the representative soil samples within the upper five feet at Borings D1, D2, D4 and D5 are granular materials when tested in accordance with the American Society for Testing and Materials (ASTM) D6913 test method (see Figures B4 and B5). Laboratory testing performed on the near-surface granular soils collected from Boring D2 also indicates these soils possess a very low expansion potential when tested in accordance with ASTM D4829 test method (see Figure B7). These near-surface soils are considered to be very low-expansive; therefore, special reinforcement of foundations and floor slabs, or special moisture conditioning during site grading to mitigate against soil expansion pressures, are not considered necessary for this project.

Clayey soils were not observed in the upper seven feet at the borings performed at the site. Pockets or other accumulations of clayey soils, if encountered at the site during earthwork operations, will likely possess a significant expansion potential and can exert significant expansion pressures on foundations and concrete slabs. Therefore, clayey soils, if encountered, should <u>not</u> be used within the upper two feet of final subgrades to support interior and exterior concrete slabs.

<u>Pavement Subgrade Quality</u>

A representative bulk sample of near-surface soils present at the site was subjected to Resistance ("R") value testing in accordance with California Test 301. Laboratory testing of the sample revealed these near-surface soils possess an R-value of 34 (see Figure B8). Based on the laboratory test results, the near-surface soils are considered good subgrade quality material for support of asphalt concrete pavements. It is our opinion an R-value of 30 is appropriate for design of pavements at the site.

<u>Groundwater Effect on Development</u>

Groundwater was not observed in the borings performed at the site on August 22, 2019 to the maximum explored depth of 51½ feet below existing site grades. Based on review of available historical groundwater data pertinent to the site, we have assumed the historical high groundwater elevation at the site to be about -7 feet NAVD 88, corresponding to a depth of about 87 feet below existing site grades. Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation and the time of year. However, in our opinion groundwater should not be a significant factor in the design or construction of this project.

Perched water may be encountered in excavations during the winter or spring, particularly after heavy rainfall. If perched groundwater is encountered during construction, dewatering of the excavations should be performed to lower the water level at least two feet below the bottom of excavations. An adequate dewatering method can best be determined during site work when subsurface conditions are fully exposed. However, localized dewatering of perched groundwater can likely be accomplished by standard sump pit and pumping procedures. Dewatering, if required, should be the contractor's responsibility.

<u>Seasonal Water</u>

During the wet season, infiltrating surface runoff water will create a saturated surface condition within the near-surface soils. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soil, intended for use as engineered fill, could require a prolonged period of dry weather and/or considerable aeration to reach a moisture content that allows achieving the required compaction. This should be considered in the construction schedule for the project.

Soils located beneath existing pavements will likely be at elevated moisture contents regardless of the time of year of construction and also require drying. Such wet soils should be anticipated and considered in the construction schedule for this project.

Excavation Conditions

The near-surface soils at the site should be readily excavatable with conventional earthmoving and trenching equipment. Surface pavements present at the site may be slower to excavate and could require increased effort; however, specialized excavation equipment is not anticipated.

Based on the explorations performed at the site, excavations associated with building foundations, shallow trenches for utilities, and any other excavations less than five feet deep, should stand vertically for short period of time (i.e. less than one day) required for construction, unless cohesionless, saturated or disturbed soils are encountered. These unstable conditions may result in caving or sloughing; therefore, the contractor should be prepared to brace or shore the excavations as needed. Excavations left open for more than a day may also be susceptible to caving or sloughing; therefore, such excavations should be evaluated by the contractor on a daily basis and determine if it is necessary to brace or shore the excavations. Bracing or shoring of excavations, if necessary, should conform to current Occupational Safety and Health Administration (OSHA) requirements.

Excavations or trenches exceeding five feet in depth that will be entered by workers should be sloped, braced or shored to conform to current OSHA requirements. 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.

Temporarily sloped excavations, if any, should be constructed no steeper than a one-and-a-half horizontal to one vertical (1½H:1V) inclination. Temporary slopes likely will stand at this inclination for the short-term duration of construction. However, due to the presence of cohesionless/granular soils at the site, flatter slopes could be required.

Excavated materials should not be stockpiled directly adjacent to an open excavation to prevent surcharge loading of the excavation sidewalls. Truck and equipment traffic should be avoided near excavations. If material is stored or heavy equipment is stationed and/or operated near an excavation, a shoring system must be designed to resist the additional pressure due to the superimposed loads.

<u>On-site Material Suitability</u>

From a geotechnical standpoint, the on-site soils are considered suitable for use as engineered fill provided that they do not contain significant quantities of organics, rubble and deleterious debris, clay and are at a proper moisture content to achieve the desired degree of compaction. However, clayey soils, if encountered beneath the site, are not considered suitable for use as fill material within the upper two feet of final soil subgrades to support interior or exterior concrete slabs or pavements, and are also not suitable for backfill behind retaining walls or other below-grade walls.

Existing asphalt concrete pavements that are not incorporated into the design of the planned improvements may be pulverized, mixed with underlying base materials, and used as engineered fill, provided existing pavements are processed into fragments less than three inches in largest dimension, mixed with the soil to form a compactable mixture, and are approved by the owner.

<u>Soil Corrosion Potential</u>

A sample of near-surface soil collected from Boring D2 was submitted to Sunland Analytical Lab of Rancho Cordova, California for testing to determine minimum resistivity, pH, and chloride and sulfate concentrations to help evaluate the potential for corrosive attack upon reinforced concrete and buried metal. The results of the corrosivity testing are summarized in Table 4. Copies of the corrosion test reports are presented in Figures B9 and B10.

Table 4: Corrosion Test Results

Analyte	Test Method	D2 (1' – 5')
pH	CA DOT 643 Modified*	6.12
Minimum Resistivity	CA DOT 643 Modified*	4,560 ϑ-cm
Chloride	CA DOT 422	0.8 ppm
Sulfate	CA DOT 417	5.2 ppm
	ASTM D 516	5.9 ppm

Notes: * = Small cell method; ϑ-cm = Ohm-centimeters; ppm = parts per million; mg/kg = milligrams per kilogram

The California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, *Corrosion Guidelines*, Version 2.1, dated January 2015, 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 taken: has 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 tested are not considered corrosive to steel reinforcement properly embedded within Portland cement concrete (PCC).

Table 19.3.1.1 – *Exposure Categories and Classes*, of American Concrete Institute (ACI) 318-14, Section 19.3 – Concrete Durability Requirements, as referenced in Section 1904.1 of the 2016 *CBC*, indicates the severity of sulfate exposure for the sample tested is Exposure Class *S0*. Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover as detailed in ACI 318-14, Section 20.6.1.3 is maintained for all reinforcement.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the proposed improvement areas a corrosion engineer should be consulted.

RECOMMENDATIONS

General

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 early spring months and will not be compactable without drying by aeration, chemical treatment or geogrid stabilization. Should the construction schedule require work during wet conditions, additional recommendations can be provided, as conditions require. Also, please note that soils under existing pavements will likely be wet regardless of the time of year of construction.

Site preparation should be accomplished in accordance with the provisions of this report and the appended guide specifications. A representative of the Geotechnical Engineer should be present during all earthwork operations to evaluate compliance with our recommendations and the guide specifications included in this report. The Geotechnical Engineer of Record referenced herein should be considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Clearing

Prior to grading, the pavements and improvements associated with the existing parking lot should be completely demolished. Planned construction areas should be cleared of any existing surface and subsurface structures to expose undisturbed soils, as determined by the Geotechnical Engineer's representative. The area to be cleared should extend at least five feet beyond the edge of all exterior foundations or the footprint of the planned building, whichever is greater, and also at least five feet beyond any exterior flatwork or pavements, where practical. Rubble and demolition debris should be removed from the site, or used as engineered fill, provided it is processed per the recommendations included in this report.

Any existing underground utilities designated to be removed or relocated should include all trench backfill and bedding materials. The resulting excavations should be restored with engineered fill placed and compacted in accordance with the recommendations included in this report. Alternatively, existing pipes may be grouted in place on a case-by-case basis, upon written approval from the Geotechnical Engineer.

If light poles associated with the existing parking lot are to be removed, the voids should be backfilled with an approved sand-cement grout up to two feet below the final soil subgrade elevation, and topped off with engineered fill placed and compacted in accordance with the recommendations included in this report.

On-site wells, septic systems, or below-grade tanks were not noted at the site during the time our field exploration was performed; however, if any of these items are discovered, they should be properly abandoned in accordance with SCEMD requirements.

Existing pavements designated for removal may be broken up, pulverized and reused as engineered fill, or removed from the site. If pavement rubble is to be reused as engineered fill, it should be pulverized to fragments less than three inches in largest dimension, mixed with soil to form a compactable mixture, and must be approved by the owner.

Surface vegetation/organics and organically laden soil (e.g. existing landscaped areas) within construction areas should be stripped from the site. Debris from the stripping should <u>not</u> be used in general fill construction within areas supporting structures, concrete slabs or pavements. With prior approval from the Geotechnical Engineer, strippings may be used in landscape areas, provided they are kept at least five feet from the building footprint, pavements, concrete slabs and other surface improvements, and are moisture conditioned and compacted.

Any trees and bushes within existing landscaped areas designated for removal should include the entire rootball and roots larger than ½-inch in diameter. Adequate removal of debris and roots may require laborers and handpicking to clear the subgrade soils to the satisfaction of the Geotechnical Engineer's representative.

Depressions resulting from site clearing operations, as well as any loose, soft, disturbed, saturated, or organically contaminated soils, as identified by the Geotechnical Engineer's representative, should be cleaned out to firm, undisturbed soils and backfilled with engineered fill placed and compacted in accordance with the recommendations in this report. It is important that the Geotechnical Engineer's representative be present during site clearing operations to verify adequate removal of the surface and subsurface items, as well as the proper backfilling of resulting excavations.

Subgrade Preparation

Approximately the upper three to four feet of surface soils across the site are in a relatively loose condition. We anticipate the upper three to four feet across the site will be disturbed during site clearing activities, which will result in further relatively loose soil conditions. In our opinion, the soils within the upper three to four feet across the site are not considered capable of providing adequate or uniform support for the planned building and associated improvements (e.g. foundations, slab-on-grade concrete, pavements, etc.) in their current condition without experiencing significant total and/or differential settlement, which can potentially result in structural damage.

Therefore, following site clearing activities, all construction areas, including building pad areas, exterior flatwork, pavements, etc., should be over-excavated to a depth of at least three feet below existing site grades. The over-excavation areas should extend at least five feet beyond the edge of all exterior foundations or the footprint of the planned building, whichever is greater, and also at least five feet beyond any exterior flatwork or pavements, where practical. We recommend that the extents of the required over-excavation be clearly marked on the civil engineering or grading plans. Any debris exposed by the required over-excavation should be removed from the site.

After over-excavation operations have been performed, the Geotechnical Engineer's representative should evaluate the exposed subgrade soils to determine if additional over-excavation is required. After the Geotechnical Engineer's representative has evaluated the bottom of the required over-excavations, the exposed subgrade soils, as well as any other surfaces to receive fill, achieved by excavation or remain at grade, should be scarified to a depth of at least 12 inches, thoroughly moisture conditioned to at least the optimum moisture content, and uniformly compacted to at least 90 percent relative compaction. Relative compaction should be based on the maximum dry density as determined in accordance with the ASTM D1557 Test Method.

If needed, alternative recommendations to stabilize the bottom of excavations can be provided upon request based on actual field conditions. The use of chemical stabilization or use of geotextile geogrids is typically recommended to stabilize soils during construction.

Compaction of soil subgrades should be performed using a heavy, self-propelled, sheep's-foot compactor capable of achieving the required compaction and must be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of subgrade under compactive load. Difficulty in achieving subgrade compaction may be an indication of loose, soft or unstable soil conditions associated with prior site activities. If these conditions exist, the soft or unstable materials should be excavated to expose stable soils. The resulting excavations should be backfilled with engineered fill compacted in accordance with the recommendations in this report.

Engineered Fill Construction

Engineered fill consisting of on-site or approved import materials should be placed in lifts not exceeding six inches in compacted thickness, with each lift being thoroughly moisture conditioned to at least optimum moisture content, maintained in that condition, and uniformly compacted to at least 90 percent relative compaction.

From a geotechnical standpoint, the on-site soils are considered suitable for use as engineered fill provided that they do not contain significant quantities of organics, rubble, deleterious debris, and clay, and are at a proper moisture content to achieve the desired degree of compaction. Organically laden topsoil should not be reused as engineered fill. Clayey soils, if encountered during earthwork operations, should not be used as fill material within the upper two feet of final soil subgrades to support interior or exterior concrete slabs or pavements, and are also not suitable for backfill behind retaining walls or other below-grade walls.

Imported fill materials should be compactable, well-graded, granular soils with a Plasticity Index of 15 or less when tested in accordance with ASTM D4318; an Expansion Index of 20 or less when tested in accordance with ASTM D4829, and should not contain particles greater than three inches in maximum dimension. Imported fill materials to be used within pavement areas should possess a minimum Resistance value of 30 when tested in accordance with California Test 301. In addition, with the exception of imported aggregate base and bedding/initial fill materials for underground utilities, we recommend that the contractor provide appropriate documentation for all imported fill materials that designates the import materials do not contain known contaminants per Department of Toxic Substances Control's guidelines for clean fill and have corrosion characteristics within acceptable limits. Imported soils should be approved by the Geotechnical Engineer prior to being transported to the site.

All soils supporting interior or exterior slab-on-grade concrete should be uniformly compacted to at least 90 percent relative compaction at a moisture content of at least the optimum moisture content, regardless of whether final grade is established by excavation, engineered fill or left at grade. The upper six inches of pavement subgrades should be uniformly compacted to at least 95 percent relative compaction at a moisture content of at least the optimum moisture content, regardless of whether final grade is established by excavation, engineered fill or left at grade. Additional recommendations for pavement subgrades are provided in the Pavement Design section of this report.

Please note that compacted subgrades consisting sandy soils present at the site will become very loose if they become dry and are disturbed by construction traffic or activities. Subgrades for support of concrete slabs-on-grade and pavements should be kept lightly moistened and protected from disturbance or rutting until covered by capillary break material or aggregate base. Disturbed or rutted subgrade soils may require additional processing and recompaction, depending on the level of disturbance.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2H:1V) and should be protected from potential erosion by suitable methods prior to the rainy season (i.e. vegetation, netting, straw bale sediment barriers, silt filter fences, etc.). Slopes should be over-built and cutback to design grades and inclinations. Please note that granular materials as those encountered at the site are susceptible to erosion even if they are properly compacted. Therefore, we recommend that in addition to the erosion protection measures described above permanent, slopes should be track-walked upon completion to further reduce the potential of erosion.

All earthwork operations should be accomplished in accordance with the recommendations contained within this report and the *Guide Earthwork Specifications* provided in Appendix C. We recommend the Geotechnical Engineer's representative be present on a regular basis during <u>all</u> earthwork operations to observe and test the engineered fill and to verify compliance with the recommendations of this report and the project plans and specifications.

<u>Utility Trench Backfill</u>

Utility trench backfill should be mechanically compacted as engineered fill in accordance with the following recommendations. Bedding of utilities and initial backfill around and over the pipe should conform to the manufacturer's recommendations for the pipe materials selected and applicable sections of the governing agency standards. If open-graded, crushed rock is used as bedding or initial backfill, an approved geotextile filter fabric should be used to separate the crushed rock from finer-grained soils. The intent of this recommendation is to reduce the potential of piping, resulting in trench settlement.

On-site soil or approved import material should be used as trench backfill. Utility trench backfill should be placed in relatively thin lifts, thoroughly moisture conditioned to at least the optimum moisture content, and uniformly compacted to at least 90 percent relative compaction. Within the upper six inches of pavement subgrade soils, compaction should be increased to at least 95 percent relative compaction. The lift thickness should be dependent on the type of compaction equipment used.

Regardless of the time of year, materials excavated from trenches could be at elevated moisture contents and could require significant aeration or a period of drying to reach a compactable moisture content. We recommend bid documents contain a unit price for the removal and drying of saturated soils or replacement with approved import soils.

Underground utility trenches aligned nearly parallel with new foundations be at least three feet from the foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at a one horizontal to one vertical (1H:1V) inclination below the foundations. Additionally, 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.

Our field explorations revealed that the upper 10 to 18 feet at the site generally consist of granular materials (silty sand and clayey sand). Granular materials, specifically silty sand, used in utility trenches have the potential to transmit water due to the permeable nature of the granular materials. Where water migration through permeable/granular utility trench materials is deemed detrimental, such as beneath the building, a trench dam (also known as a trench plug) consisting of concrete, controlled density fill or other suitable material should be constructed in the utility trench to inhibit the flow of water through the permeable/granular materials. The trench dam should be at least one foot thick, extend at least six inches below the bottom of the trench, and extend at least 12 inches into the sides of the trench. The top of the trench dam should be about one foot above the top of any permeable/granular material used in the trenches, such as bedding, initial backfill or granular/permeable fill material.

Foundation Design

In our opinion, the planned corporation yard building can be supported upon a continuous perimeter foundation with continuous and/or isolated interior spread foundations embedded at least 12 inches below lowest adjacent soil grade, provided the building pad has been prepared in accordance with the Subgrade Preparation and Engineered Fill Construction sections of this report. Lowest soil grade is defined as either the adjacent exterior soil grade or the soil subgrade beneath the building, whichever is lower. A continuous, reinforced foundation should be utilized for the perimeter of the building to act as a “cut-off” to help minimize moisture infiltration and variations beneath the interior slab-on-grade areas of the building. Continuous foundations should maintain a minimum width of 12 inches and isolated spread foundations should be at least 24 inches in plan dimension.

Foundations embedded at least 12 inches below lowest adjacent soil grade may be sized for maximum allowable “net” soil bearing pressures of 3,000 pounds per square foot (psf) for dead plus live loads, with a 1/3 increase to include the short-term effects of wind or seismic forces. The weight of the foundation concrete extending below lowest adjacent soil grade may be disregarded in sizing computations.

Please note that due to the granular nature of the on-site soils, conventional shallow foundations may require forms for proper construction. If necessary, the foundation contractor should be prepared to use forms to size and construct the foundations in accordance with the approved structural engineering plans.

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

Interior Floor Slab Support

Interior concrete slab-on-grade floors for the planned building can be supported upon soil subgrade prepared in accordance with the Subgrade Preparation and Engineered Fill Construction sections of this report, provided the subgrade soils are maintained in a moist condition and protected from disturbance.

Interior concrete slab-on-grade floors should be at least five inches thick; however, the project structural engineer or slab designer should determine the final floor slab thickness. Interior floor slabs should be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The project structural engineer or slab designer also should determine final reinforcing requirements and joint spacing. Temporary loads exerted during construction from vehicle traffic, construction equipment, storage of palletized construction materials, etc. should be considered in the design of the thickness and reinforcement of the interior concrete slabs-on-grade.

If any interior floor slabs for the planned building will be subjected to vehicular or forklift traffic, those floor slabs should be designed as pavements in accordance with the Pavement Design section of this report.

Interior floor slabs for the planned building that are not subject to vehicular or forklift traffic should be underlain by a layer of free-draining crushed rock/gravel, serving as a deterrent to migration of capillary moisture. The crushed rock/gravel layer should be between four- and six-inches-thick and graded such that 100 percent passes a one-inch sieve and less than five percent passes a No. 4 sieve. Additional moisture protection may be provided by placing a vapor retarder membrane (at least 10-mils thick) directly over the crushed rock/gravel. The water vapor retarder membrane should meet or exceed the minimum specifications as outlined in ASTM E1745 and be installed in strict conformance with the manufacturer’s recommendations.

Floor slab construction practice over the past 30 years or more has included placement of a thin layer of sand or pea gravel over the vapor retarder membrane. The intent of the sand/pea gravel is to aid in the proper curing of the slab concrete. However, debate over excessive moisture vapor emissions from floor slabs includes concern of water trapped within the sand. As a consequence, we consider use of the sand/pea gravel 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 reduce significant soils-related cracking of interior concrete slabs-on-grade. Equally important to the performance and appearance of Portland cement concrete slabs is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized and spacing of control joints.

Floor Slab Moisture Penetration Resistance

It is likely that floor slab subgrade soils will become saturated at some time during the life of the structure, especially when slabs are constructed during the wet seasons, or when constantly wet ground or poor drainage conditions exist adjacent to the building. For this reason, it should be assumed that interior concrete slab-on-grade floors will require protection against moisture/water or moisture vapor penetration. Standard practice includes placing a layer of gravel/crushed rock and a vapor barrier membrane (and possibly a layer of sand or pea gravel) as discussed above. However, the gravel and plastic water vapor retarder offer only a limited, first-line of defense against soil-related moisture. Recommendations contained in this report concerning slab-on-grade floor design are presented as minimum requirements only from the geotechnical engineering standpoint.

It is emphasized that the use of gravel/crushed rock and plastic membrane below the slab will not “moisture/water proof” the slab-on-grade floor slabs, nor will it assure that slab moisture transmission levels will be low enough to prevent damage to floor coverings or other building components. Please note that we are not slab moisture/water proofing or moisture protection experts. The sub-slab gravel/crushed rock and vapor barrier membrane simply offers a first line of defense against soil-related moisture. We recommend that a concrete moisture protection specialist or moisture/water-proofing expert be consulted for adequate protection against moisture vapor or water penetration of the slab-on-grade floor slabs. The design team should consider all available measures for floor slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

Retaining Walls

Foundations for retaining walls or below-grade walls for the loading docks could be supported on a continuous foundation at least 12 inches wide, extending at least 12 inches below lowest adjacent soil grade. Continuous footings for retaining walls may be designed based on maximum allowable "net" soil bearing pressures of 3,000 psf for dead plus live load conditions, which may be increased by one-third for effects of wind or seismic forces.

Retaining walls or below grade walls for the loading docks should be designed to resist the lateral soil pressures of the retained soils. Retaining walls that are fixed/restrained at the top should be capable of resisting an "at-rest" lateral soil pressure equal to an equivalent fluid pressure of 60 psf per foot of the wall height (fully drained conditions). Retaining walls that will be allowed to slightly rotate about their base (unrestrained at the top or sides) should be capable of resisting an "active" lateral soil pressure equal to an equivalent fluid pressure of 40 psf per foot of wall height (fully drained conditions). Walls supporting sloping backfill, up to a two horizontal to one vertical (2H:1V) inclination, should be designed by adding an additional 20 psf per foot of wall to the pressures presented above. The equivalent fluid pressures in this section assume no hydrostatic pressure or surcharge loads behind the wall.

Based on recent research (Lew, et al. 2010), the seismic increment of earth pressure may be neglected if the maximum peak ground acceleration at the site is 0.4 g or less. Our analysis indicates the maximum peak ground acceleration (PGA_M) at the site will be about 0.264 g; therefore, the seismic increment of lateral earth pressure may be neglected, and retaining walls may be designed using the lateral earth pressures presented above.

If structural elements encroach the one horizontal to one vertical (1H:1V) projection from the bottom of retaining walls or below-grade walls, the walls should account for surcharge loads resulting from those structural elements. Additionally, retaining walls or below-grade walls should also account for surcharge loads resulting from construction equipment, vehicles, palletized materials, etc. that encroach the one horizontal to one vertical (1H:1V) projection from the bottom of the below-grade walls. Surcharge loading under the circumstances described above should be evaluated by the retaining wall designer on a case-by-case basis and be included in their design of the walls. The retaining wall designer should evaluate the surcharge load distribution, magnitude of the surcharge resultant force to be applied on the walls, and the location of where the resultant force should be applied on the walls. Surcharge loading on the retaining walls will depend on the specific surcharge load type (e.g. point load, distributed load, etc.) and distance away from the retaining walls.

Retaining walls or below grade walls should be fully drained to prevent the build-up of hydrostatic pressures behind the walls. Retaining walls should be provided with a drainage blanket of Class 2 permeable material, *Caltrans Standard Specification, Section 68-2.02F(3)*, at least one foot wide extending from the base of wall to within one foot of the top of the wall. The top foot above the drainage layer should consist of compacted on-site or imported engineered fill materials, unless covered by a concrete slab or pavement. Weep holes or perforated rigid pipe, as appropriate, should be provided at the base of the wall to collect accumulated water. Drainpipes, if used, should slope to discharge at no less than a one percent fall to suitable drainage facilities. Open-graded ½- to ¾-inch crushed rock may be used in lieu of the Class 2 permeable material, if the rock and drain pipe are completely enveloped in an approved non-woven, geotextile filter fabric. Alternatively, approved geotextile drainage composites such as MiraDRAIN® may be used in lieu of the drain rock layer. If used, geotextile drainage composites should be installed in accordance with the manufacturer's recommendations.

If efflorescence (discoloration of the wall face) or moisture/water penetration of the retaining walls is not acceptable, moisture/water-proofing measures should be applied to the back face of the walls. A moisture/water-proofing specialist should be consulted to determine specific protection measures against moisture/water penetration through the walls.

Structural backfill materials for retaining walls or below-grade walls within a one horizontal to one vertical (1H:1V) projection from the bottom of the walls (other than the drainage layer) should consist of on-site or imported, compactable granular material that does not contain significant quantities of rubbish, rubble, organics and rock over six inches in size. Clay soils, pea gravel and/or crushed rock should not be used for structural wall backfill. Structural wall backfill should be placed in lifts not exceeding 12 inches in loose thickness, moisture conditioned to at least the optimum moisture content, and should be mechanically compacted to at least 90 percent relative compaction.

Exterior Flatwork (Non-Pavement Areas)

Soil subgrade areas to support exterior concrete flatwork (i.e. sidewalks, patios, etc.) should be prepared and constructed in accordance with the Subgrade Preparation and Engineered Fill Construction recommendations included in this report. Exterior flatwork subgrade soils should be maintained in a moist condition and protected from disturbance. Exterior flatwork should be directly underlain by at least four inches of aggregate base compacted to at least 95 percent relative compaction to provide stability during slab construction and to protect the sandy soils from disturbance during construction.

Exterior flatwork concrete should be at least four inches thick. Consideration should be given to thickening the edges of the slabs at least twice the slab thickness where wheel traffic is expected over the slabs. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of other structural elements by the placement of a layer of felt material between the flatwork and the structural element. The slab designer should determine the final thickness, strength and joint spacing of exterior slab-on-grade concrete. The slab designer should also determine if slab reinforcement for crack control is required and determine final slab reinforcing requirements.

Practices recommended by the Portland Cement Association (PCA) for proper placement, curing, joint depth and spacing, construction, and placement of concrete should be followed during exterior concrete flatwork construction.

Exterior flatwork that is subject to vehicular or forklift traffic should be designed as pavement per the recommendations presented below.

Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from the planned building and prevent ponding of water adjacent to foundations, concrete slabs or pavements. The grade adjacent to planned building should be sloped away from foundations at a minimum two percent slope for a distance of at least five feet, where possible. We recommend connecting all roof drains to solid drainage pipes which are connected to available drainage features to convey water away from the building or discharging the drains onto paved or hard surfaces that slope away from the foundations. Discharging or ponding of surface water should not be allowed adjacent to the building, concrete slabs or pavements. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward the building.

Pavement Design

Based on laboratory testing results, a Resistance (“R”) value of 30 was used for design of pavements subgrades consisting of on-site or imported granular soils. The pavement sections presented in Table 5 have been calculated using the above R-value and traffic indices (TIs) assumed to be appropriate for this project. The procedures used for pavement design are in general conformance with Chapters 600 to 670 of the *California Highway Design Manual*, dated November 20, 2017 (Caltrans, 2017). The project civil engineer should determine the appropriate traffic index and pavement section based on anticipated traffic conditions. If needed, we can provide alternative pavement sections for different traffic indices.

Table 5 – Pavement Design Alternatives

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 30		
		Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Automobile Parking Only	2½*	6	--
		--	6	4
6.0	Automobile, Light Truck Traffic, and Fire Lanes	3	9	--
		3½*	8	--
		--	8	5
7.0	Moderate Truck Traffic, Trash Enclosures, Entry/Exit Driveways, and Storage/Loading Areas	3½	11	--
		4*	10	--
		--	8	6

Notes: * = *Asphalt concrete thickness contains the Caltrans safety factor.*

We emphasize that the performance of pavement is critically dependent upon uniform and adequate compaction of the soil subgrade, as well as all engineered fill and utility trench backfill within the limits of the pavements. We recommend that final pavement subgrade preparation (i.e. scarification, moisture conditioning and compaction) be performed after underground utility construction is completed and just prior to aggregate base placement.

The upper six inches of pavement subgrade soils should be compacted to at least 95 percent relative compaction at no less than the optimum moisture content, maintained in a moist condition and protected from disturbance. All aggregate base should be compacted to at least 95 relative compaction.

Pavement subgrades should be stable and unyielding under heavy wheel loads of construction equipment. To help identify unstable subgrades within the pavement limits, a proof-roll should be performed with a fully-loaded, water truck on the exposed subgrades prior to placement of aggregate base. The proof-roll should be observed by the Geotechnical Engineer's representative.

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using Portland cement concrete (PCC) pavements in areas subjected to concentrated heavy wheel loading, such as entry/exit driveways, in front of trash enclosures, and/or in storage/loading areas. Alternate PCC pavement sections have been provided above in Table 5.

We suggest concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards (ACI, 2016), latest edition. Reinforcing for crack control, if desired, should be provided in accordance with ACI guidelines. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform to the current PCA or ACI guidelines. PCC should achieve a minimum compressive strength of 3,500 pounds per square inch at 28 days.

All pavement materials and construction methods of structural pavement sections should conform to the applicable provisions of the *Caltrans Standard Specifications*, latest edition.

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. Weep holes could be provided at drainage inlets, located at the subgrade-base interface, to allow accumulated water to drain from beneath the pavements.

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.

Geotechnical Engineering Observation and Testing of Earthwork Construction

Site preparation should be accomplished in accordance with the recommendations of this report and the *Guide Earthwork Specifications* provided in Appendix C. Geotechnical testing and observation during construction is considered a continuation of our geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site clearing, preparation, earthwork, and foundation construction for the project to verify compliance with this report and the project plans and specifications and to provide consultation as required during construction. These services are beyond the scope of work authorized for this investigation. We would be pleased to submit a proposal to provide these services upon request.

The 2016 *CBC* requires that the geotechnical engineering report provide a number and frequency of field compaction tests to determine compliance with the recommended minimum compaction. Many factors can affect the number of tests that should be performed during the course of construction, such as soil type, soil moisture, season of the year and contractor operations/performance. Therefore, it is crucial that the actual number and frequency of testing be determined by the Geotechnical Engineer during construction based on their observations, site conditions, and construction conditions encountered.

In the event that Wallace-Kuhl & Associates is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide these services, in conformance with Section 1704.7 and 1704.9 of the 2016 edition of the CBC, should indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary. A final report by the "Geotechnical Engineer" should be prepared upon completion of the project as required by the CBC Section 1704.7.1.

Additional Future Services

We recommend that Wallace-Kuhl & Associates be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents. We can prepare a proposal to provide these services upon request.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed project, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used engineering judgment based upon the information provided and the data generated from our investigation. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. 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 our boring locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

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

Wallace - Kuhl & Associates

REGISTERED PROFESSIONAL ENGINEER
MAURICIO LUNA
No. 81151
Exp. 9/30/21
CIVIL
STATE OF CALIFORNIA

Mauricio Luna, CE
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
DAVID R. GIUS, JR.
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PROFESSIONAL GEOLOGIST
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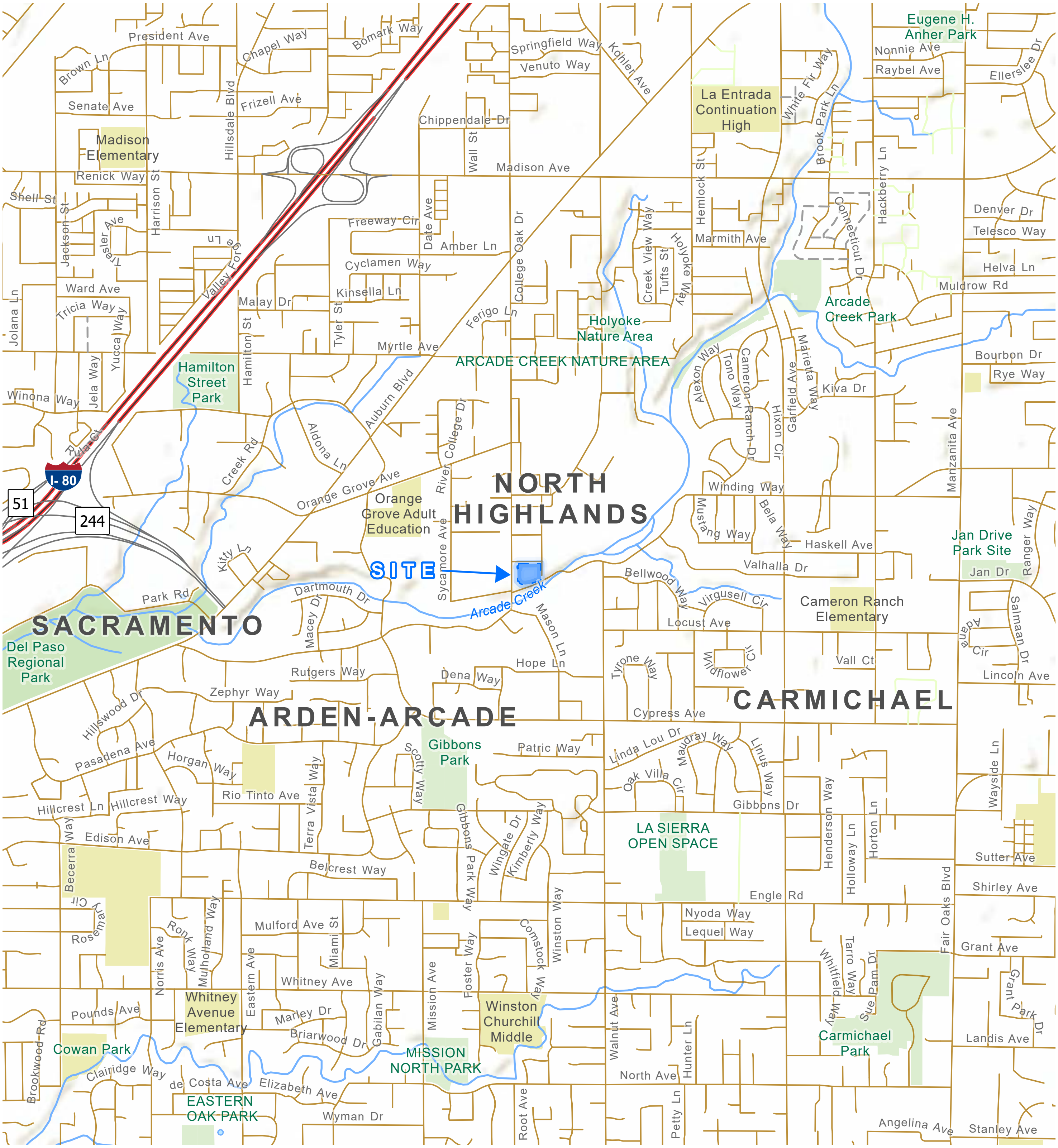
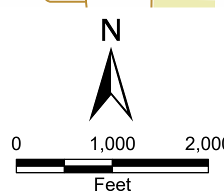
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

David R. Gius, Jr., GE, CEG
Principal Engineer/Senior Engineering Geologist

Joseph R. Ybarra
Staff Geologist





Spatial Data provided by Esri, NOAA, and USGS.
Projection: NAD 83, California State Plane, Zone II.

WallaceKuhl & ASSOCIATES

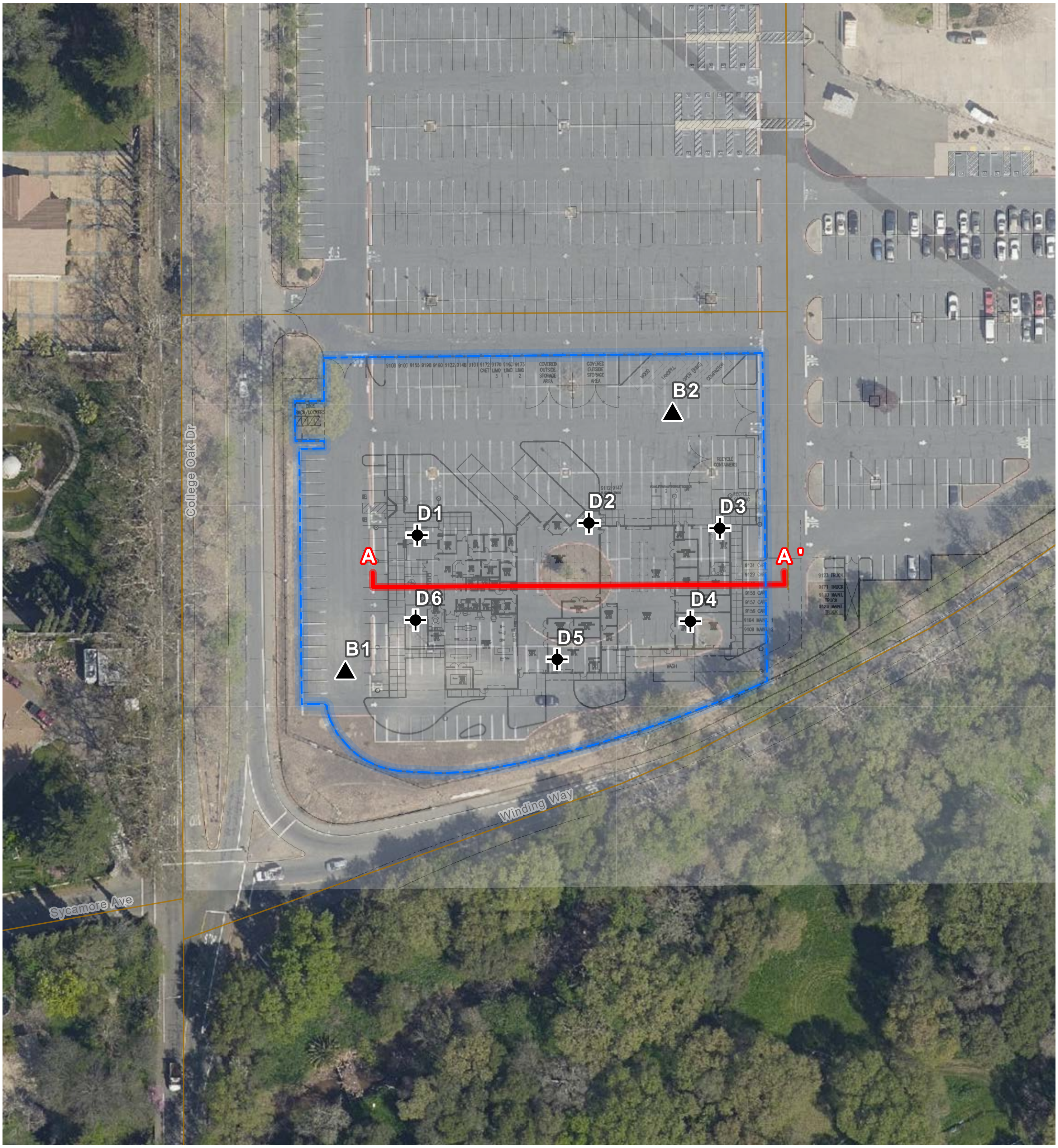
VICINITY MAP

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	1
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19

WKA NO. 12456.01P



Approximate Site Boundary
Approximate Boring Location
Approximate Bulk Sample Location
Approximate Cross Section Line

N

0 50 100
Feet

Aerial imagery provided by Esri.
Site Plan adapted from a drawing prepared by GRA Architecture, dated 05/28/2019.
Projection: NAD 83, California State Plane, Zone II.

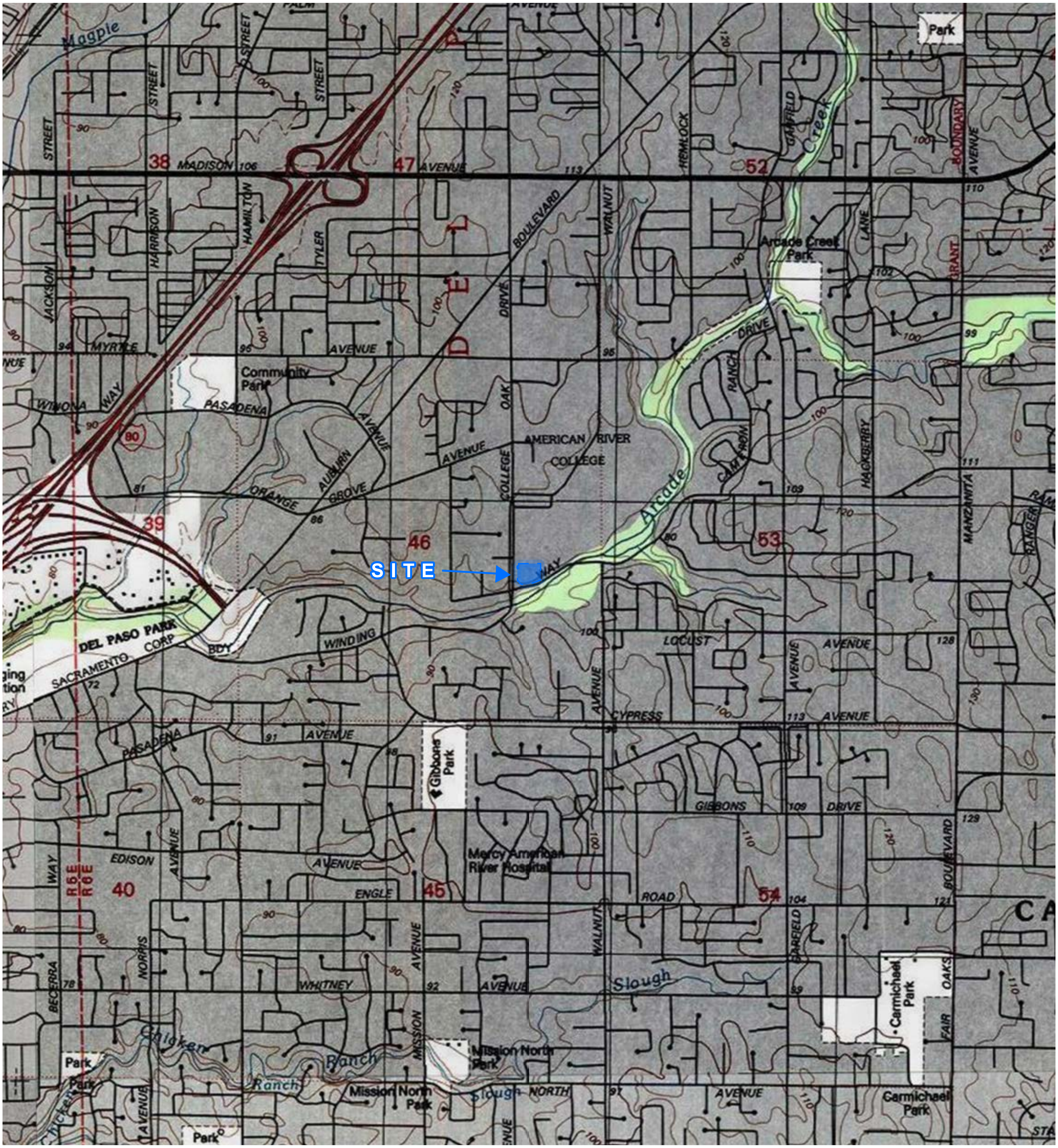
WallaceKuhl & ASSOCIATES

SITE PLAN

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	2
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	



Elevations in feet, NAVD 88.
Contour interval = 10 feet.

Topographic Map adapted from the Citrus Heights Quadrangle, 7.5-minute series, dated 1992.
Projection: NAD 83, California State Plane, Zone II.

N

0 1,000 2,000
Feet

WallaceKuhl & ASSOCIATES

TOPOGRAPHIC MAP

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	3
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	

Riverbank Formation

Qr	Qr_3	Qr - Undivided
	Qr_2	Qr_3 - Upper unit
	Qr_1	Qr_2 - Middle unit
		Qr_1 - Lower unit

Qtl	Turlock Lake Formation
Qha	Holocene alluvium
Qhc	Latest Holocene stream channel deposits
t	Dredge tailings

N

0 2,000 4,000
Feet

Geologic Map adapted from the Preliminary Geologic Map of the Sacramento 30'x60' Quadrangle, California (Gutierrez,2011). Projection: NAD 83, California State Plane, Zone II.

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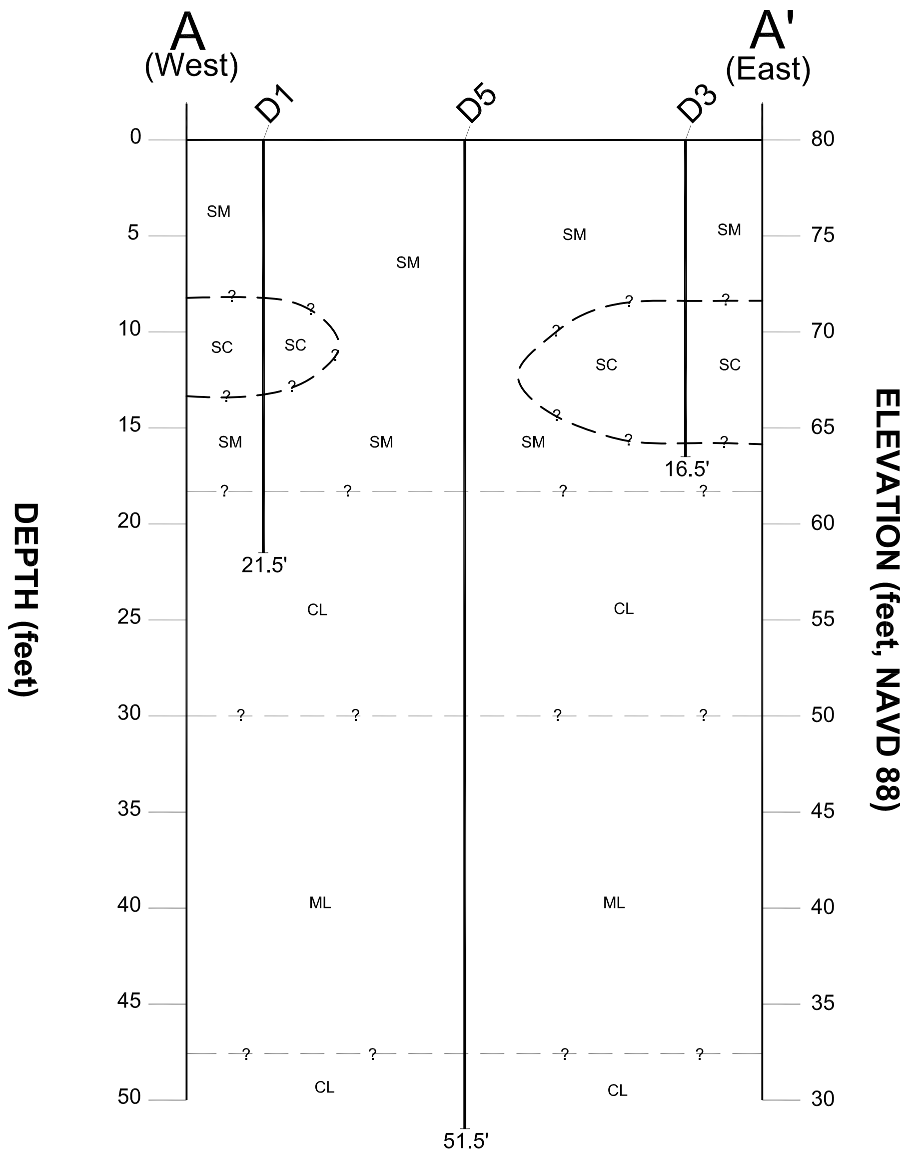
GEOLOGIC MAP

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	4
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	



Key:

Geologic contact, solid where well located, dashed where approximate, queried where inferred.

(XX) Dominant soil types and symbols (USCS shorthand after ASTM D2487-5)*

* Detailed soil logs and dsecriptions available in Boring Logs, Figures 6 through 11

* Elevations, Depths, and Locations are approximate.

Scale
Horizontal: 1" = 100'
Vertical: 1" = 10'

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GEOLOGIC CROSS SECTION A-A'

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	5
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO.12456.01P	

Project: American River College Corporation Yard
Project Location: Sacramento, California
WKA Number: 12456.01P

LOG OF SOIL BORING D1

Date(s) Drilled	**8/22/19**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling, Inc.**	Total Depth of Drill Hole	**21.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk Sample (1'-5'); GR, < 0.075mm = 44%; RV = 34.**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			4 inches of Asphalt Concrete						
			2 inches of Aggregate Base						
			Brown, moist, loose, silty SAND (SM); fine to coarse sand						
					D1-1I	9	10.5	111	
	5		medium dense						
					D1-2I	23	10.4	107	
			Brown, moist, medium dense, clayey SAND (SC); fine to coarse sand						
	10				D1-3I	23	19.5	102	
			Olive brown, moist, medium dense, silty SAND with gravel (SM); fine to coarse sand; fine gravel						
	15				D1-4I	25	21.5	87	
			Brown, moist, very stiff, lean CLAY with sand (CL)						
	20				D1-5I	37			

Boring terminated at about 21 1/2 feet below site grade. Groundwater was not observed.

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FIGURE 6

Project: American River College Corporation Yard
Project Location: Sacramento, California
WKA Number: 12456.01P

LOG OF SOIL BORING D2

Date(s) Drilled	**8/22/19**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling, Inc.**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk Sample (1'-5'); GR, < 0.075mm = 32%; EI = 0.**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		5 inches of Asphalt Concrete						
			2 inches of Aggregate Base						
			Brown, moist, loose, silty SAND (SM); fine to coarse sand		D2-1I	8	11.8	97	
			medium dense		D2-2I	12	7.8	105	
	5								
	10				D2-3I	32	9.3	104	
			Boring terminated at about 10 feet below site grade. Groundwater was not observed.						

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FIGURE 7

Project: American River College Corporation Yard
Project Location: Sacramento, California
WKA Number: 12456.01P

LOG OF SOIL BORING D3

Date(s) Drilled	**8/22/19**	Logged By	**JRY**	Checked By	**DRG**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling, Inc.**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			4 inches of Asphalt Concrete						
			2 inches of Aggregate Base						
			Brown, moist, loose, silty SAND (SM); fine to coarse sand						
					D3-1I	10			
	5		medium dense						
					D3-2I	26			
			Brown, moist, medium dense, clayey SAND (SC); fine to coarse sand						
	10								
					D3-3I	23			
	15		Olive brown, moist, medium dense, silty SAND (SM); fine to coarse sand						
					D3-4I	30			

Boring terminated at about 16 1/2 feet below site grade. Groundwater was not observed.

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FIGURE 8

Project: American River College Corporation Yard **Project Location: Sacramento, California** **WKA Number: 12456.01P**	**LOG OF SOIL BORING D4** Sheet 1 of 1

Date(s) Drilled	**8/22/19**	Logged By	**JRY**	Checked By	**DRG**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling, Inc.**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk Sample (1'-5'); GR, < 0.075mm = 30%.**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			4 inches of Asphalt Concrete						
			2 inches of Aggregate Base						
			Brown, moist, loose, silty SAND (SM); fine to coarse sand; trace of fine gravel		D4-1I	6	11.3	110	TR
			medium dense						
	5				D4-2I	12	9.6	118	TR
			Brown, moist, medium dense, clayey SAND (SC); fine to coarse sand; trace of gravel						
	10				D4-3I	36	11.2	122	
			Boring terminated at about 10 feet below site grade. Groundwater was not observed.						

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FIGURE 9

Project: American River College Corporation Yard
Project Location: Sacramento, California
WKA Number: 12456.01P

LOG OF SOIL BORING D5

Date(s) Drilled	**8/22/19**	Logged By	**JRY**	Checked By	**DRG**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V&W Drilling, Inc.**	Total Depth of Drill Hole	**51.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**1.4" Standard Penetration Test (SPT)**	Drill Hole Backfill	**Neat Cement Grout**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			4 inches of Asphalt Concrete						
			3 inches of Aggregate Base						
			Brown, moist, loose, silty SAND (SM); fine to coarse sand		D5-1I	6			GR <0.075mm 37%
	5		medium dense		D5-2I	12			GR <0.075mm 34%
	10		brown to reddish brown		D5-3I	22	16.4		
	15		olive brown		D5-4I	23	15.3		
			Brown with black mottling, moist, very stiff, sandy, silty CLAY (CL-ML)						
	20				D5-5I	18			PI=5 FC <0.075mm 51%
	25		light brown, stiff		D5-6I	16	43.2		
	30		Light brown, moist, medium dense, sandy SILT (ML)		D5-7I	25			GR <0.075mm 53%

LOG OF SOIL BORING D5

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			dense		D5-8I	41	31.1		
	40		medium dense		D5-9I	24	27.8		
	45				D5-10I	17	29.5		
			Dark brown, moist, very stiff, lean CLAY with sand (CL)						
	50				D5-11I	29	23.4		
			Boring terminated at about 51 1/2 feet below site grade. Groundwater was not observed.						

Project: American River College Corporation Yard
Project Location: Sacramento, California
WKA Number: 12456.01P

LOG OF SOIL BORING D6

Sheet 1 of 1

Date(s) Drilled	**8/22/19**	Logged By	**JRY**	Checked By	**DRG**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling, Inc.**	Total Depth of Drill Hole	**11.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Observed**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			4 inches of Asphalt Concrete						
			2 inches of Aggregate Base						
			Brown, moist, loose, silty SAND (SM); fine to coarse sand						
					D6-1I	11	12.7	109	
	5		medium dense						
					D6-2I	20	8.9	106	
			Olive brown, moist, medium dense, clayey SAND (SC); fine to medium sand						
	10								
					D6-3I	26	22.0	103	
			Boring terminated at about 11 1/2 feet below site grade. Groundwater was not observed.						

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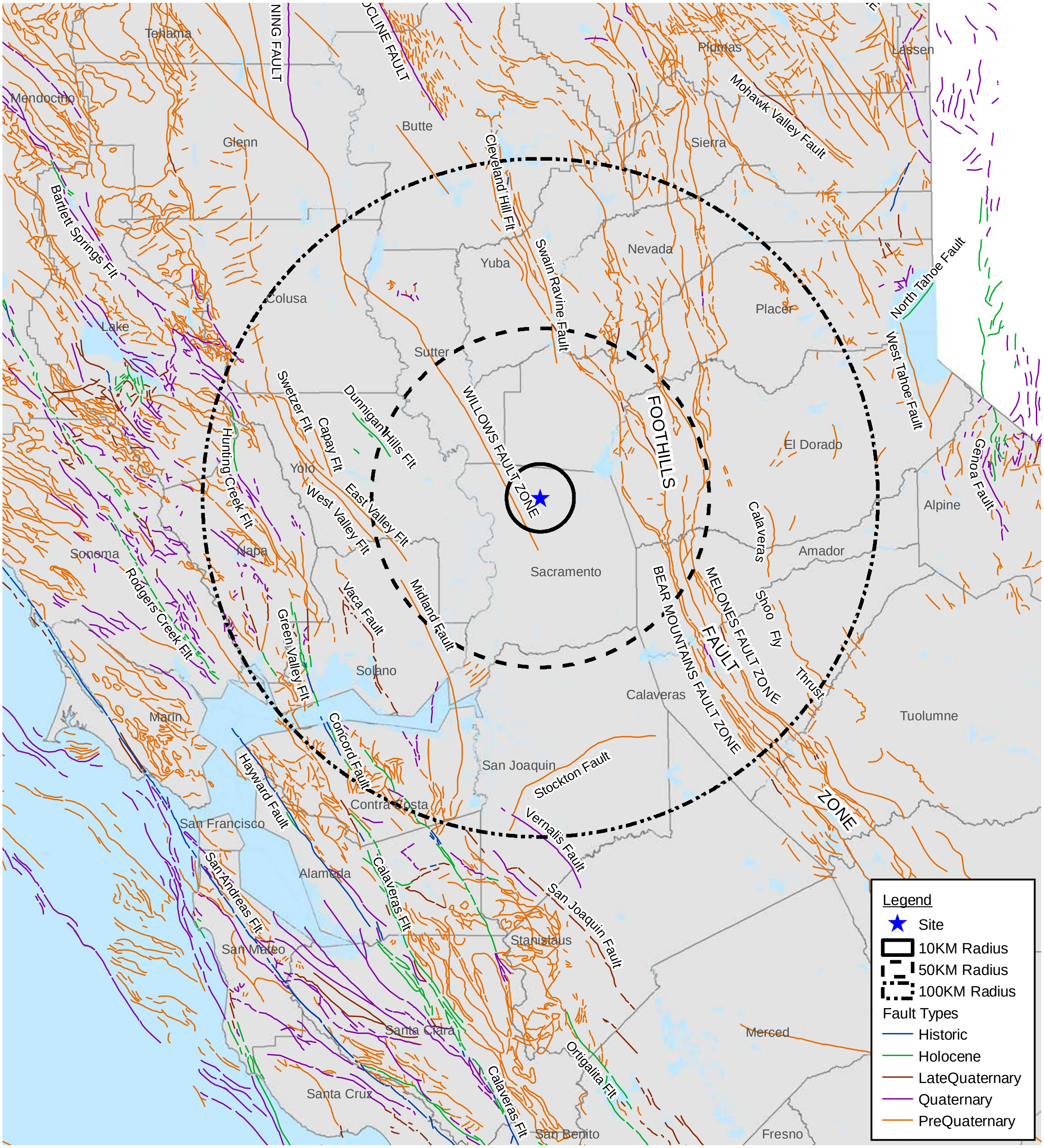
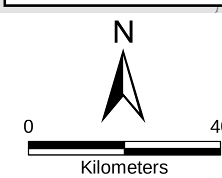


Modified from Division of Mines and Geology, CD-ROM 2000-08 (2000), Digital database of faults from the Fault Activity Map of California and Adjacent Areas and from the USGS Quaternary Fault and Fold Database of the United States, dated November 3, 2010.
Projection: NAD 83, California State Plane, Zone II.



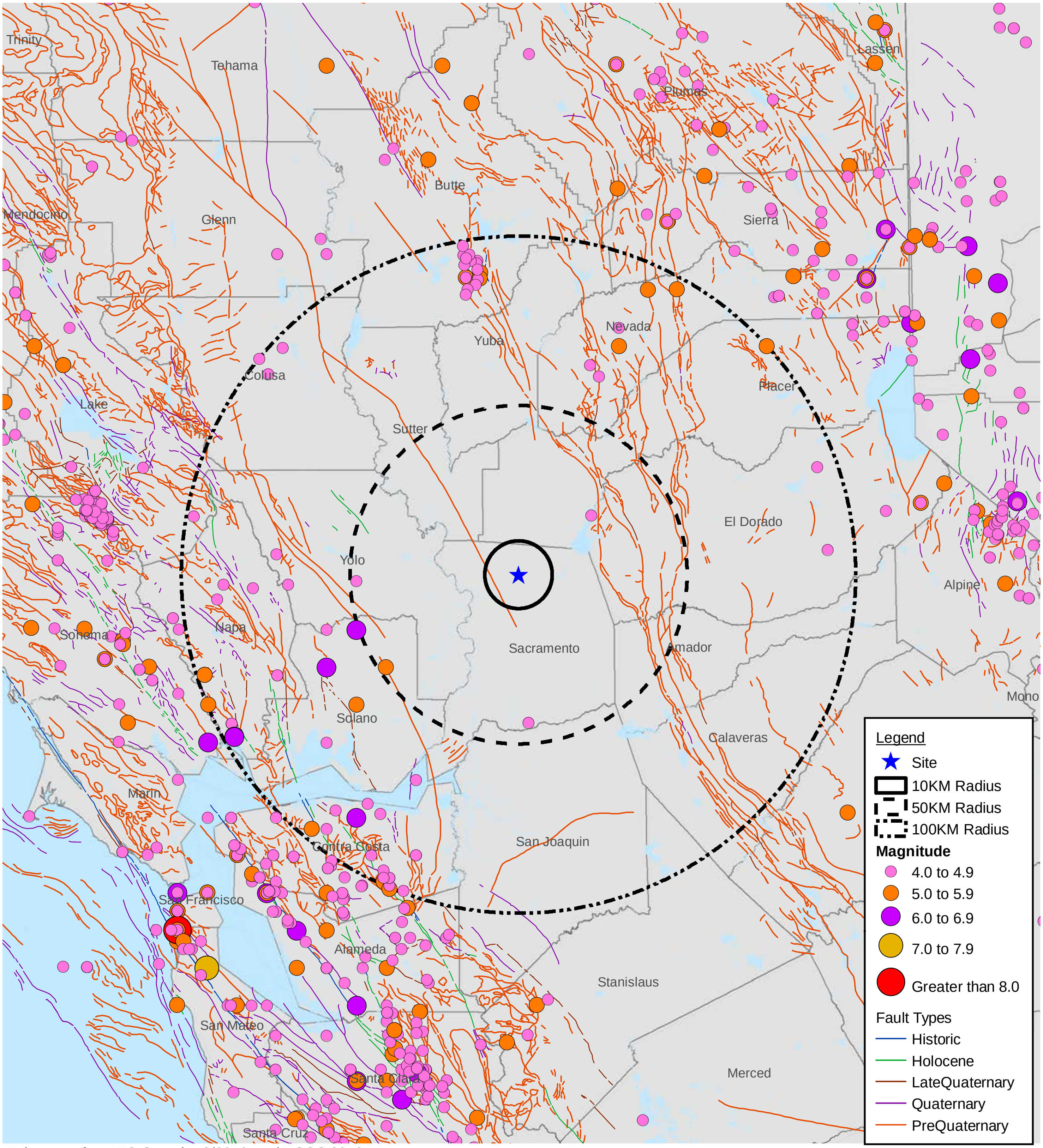
WallaceKuhl & ASSOCIATES

FAULT MAP

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	13
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	



Epicenters from EQ Search 12/2016, and USGS 2016.
Projection: NAD 83, California State Plane, Zone II.

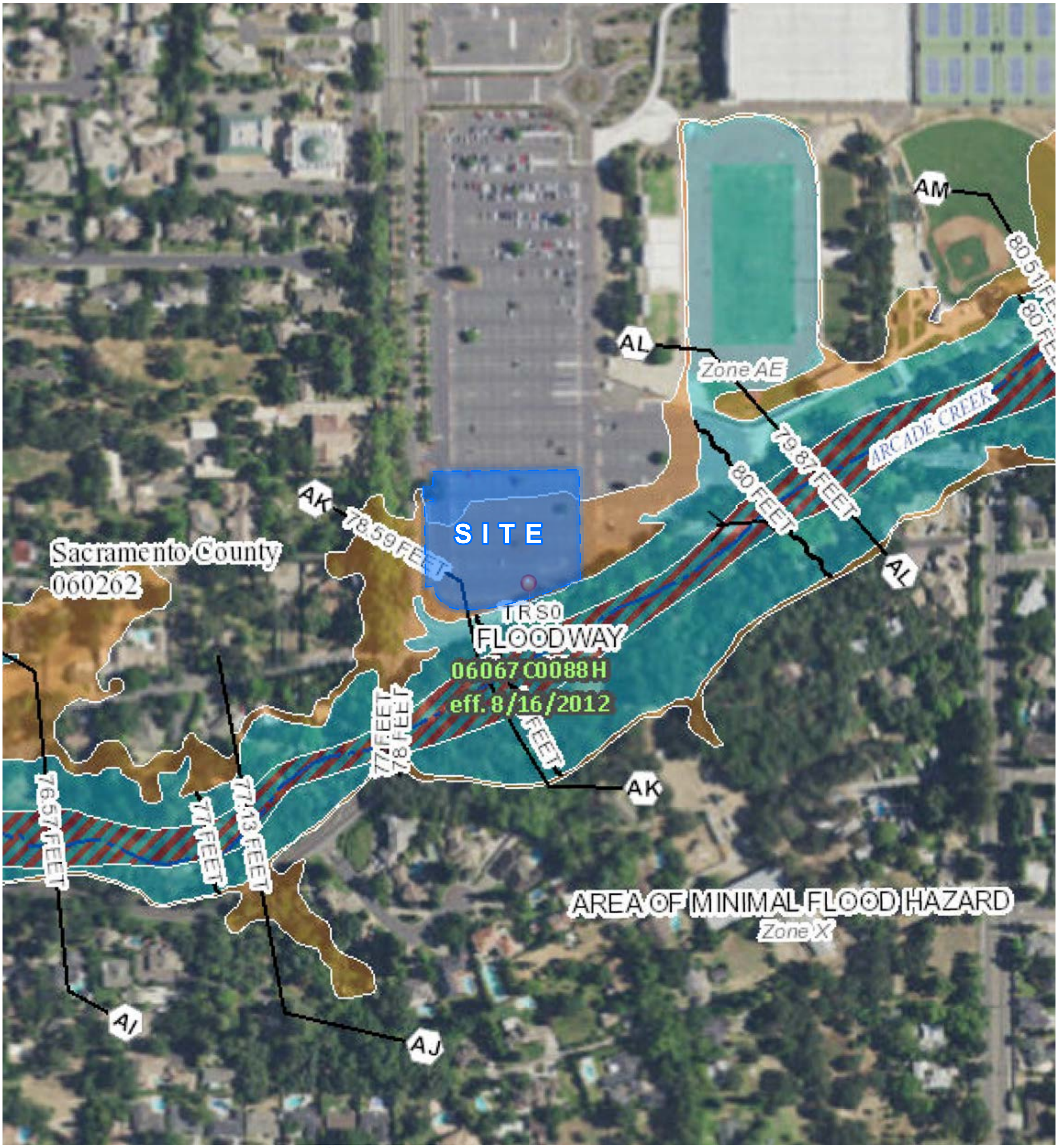
WallaceKuhl & ASSOCIATES

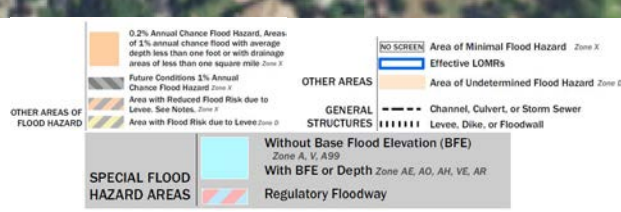
EPICENTER MAP

AMERICAN RIVER COLLEGE CORPORATION YARD

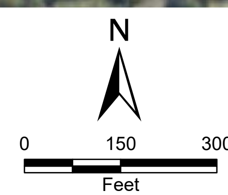
Sacramento, California

FIGURE	14
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	MSM
DATE	10/19
WKA NO. 12456.01P	





Flood Map prepared by NFHL and FEMA, dated 08/16/2012.
Projection: NAD 83, California State Plane, Zone II.



FEMA FLOOD MAP

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

WallaceKuhl & ASSOCIATES

FIGURE	15
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	

APPENDICES

APPENDIX A

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APPENDIX A – REFERENCES

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APPENDIX B

General Project Information, Laboratory Testing and Results

APPENDIX B

A. GENERAL INFORMATION

The performance of a geotechnical engineering investigation and geologic hazards study for the proposed corporation yard to be constructed in the southwest portion of the American River College College campus, located at 4700 College Oak Drive in Sacramento, California was authorized by the Los Rios Community College District on August 14, 2019. Authorization was for a study as described in our proposal letter dated August 5, 2019, sent to our client Los Rios Community College District, whose address is 3753 Brandview Drive in Sacramento, California, 95827.

In performing this investigation, we made reference to a *Site Plan* drawing, dated May 28, 2019, prepared by GRA Architecture of Sacramento, California (project architect).

B. FIELD EXPLORATION

As part of our study for the proposed corporation yard, our field explorations included the drilling and sampling of six borings (D1 through D6) at the approximate locations shown on Figure 2.

The borings were drilled on August 22, 2019 to depths ranging from about 10 to 51½ feet below existing site grades utilizing a CME-75 truck-mounted drill rig equipped with six-inch-diameter, solid stem and eight-inch-diameter, hollow stem augers. At various intervals soil samples were recovered from the borings with a 2½-inch outside diameter (O.D.), 2-inch inside diameter (I.D.), modified California; o,r a 2-inch outside diameter (O.D.), 1⅜-inch inside diameter (I.D.), Standard Penetration Test (SPT) split-spoon sampler. The samplers were driven by an automatic 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long samplers each 6-inch interval was recorded. The sum of the blows required to drive the sampler the lower 12-inch interval is designated the penetration resistance or "blow count" for that particular drive. The modified California samples were retained in 2-inch diameter by 6-inch long, thin walled brass tubes contained within the sampler. The SPT samples were retained in plastic zip-lock bags. After recovery, the field representative visually classified the soil recovered in the tubes and plastic bags. After the samples were classified, the tubes and plastic bags were sealed to preserve the natural moisture contents.

In addition to the driven samples, representative bulk samples of near-surface soils also were collected and retained in plastic bags. Driven and bulk samples were taken to our laboratory for additional soil classification and selection of samples for testing.

At the completion of our field exploration activities, with the exception of Boring D5, the borings (about 10 to 21½ deep) were backfilled with the excavated soil cuttings. Boring

D5 (about 51½ feet deep) was backfilled a slurry of neat cement, bentonite, and water in accordance with Sacramento County Environmental Management Department (SCEMD) requirements. The surface of the borings (about the upper 6 inches) was restored with asphalt concrete cold patch.

The Logs of Soil Borings containing descriptions of the soils encountered in each boring are presented as Figures 6 through 11. A Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained in Figure 12.

LABORATORY TESTING

Selected driven samples of the near-surface soils were tested to determine the dry unit weight (ASTM D2937) and natural moisture content (ASTM D2216). The results of these tests are included in the Logs of Soil Borings at the depth each sample was obtained.

One bulk sample of near-surface soils was subjected to Modified Proctor moisture density testing (ASTM D1557). The results of the test are presented in Figure B1.

Two driven samples of near-surface soils were tested to determine the shear strength by triaxial compression testing (ASTM D4767). The results of the triaxial shear strength testing are presented in Figures B2 and B3.

Seven samples of near-surface soils were tested for particle-size distribution (ASTM D6913). The results of the particle-size distribution tests are contained in Figures B4 and B5. The percent passing the No. 200 sieve is included on the Logs of Soil Borings at the depth the samples were obtained.

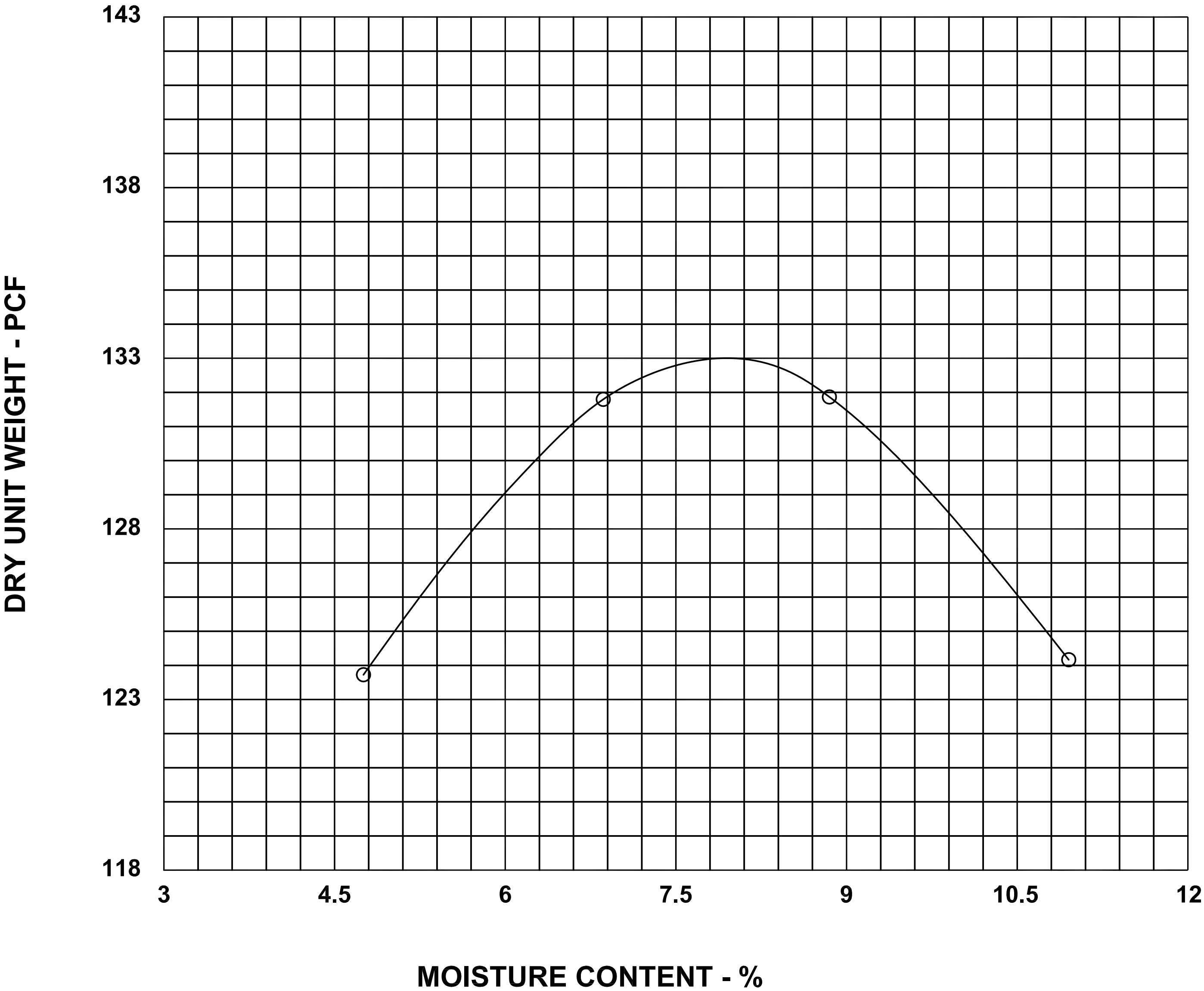
One sample of near-surface soils was subjected to Atterberg Limits tests (ASTM D4318). The results of these tests are presented in Figure B6.

One bulk sample of near-surface soils was subjected to Expansion Index testing (ASTM D4829). The results of the test are presented in Figure B7.

One bulk sample of anticipated pavement subgrade soils was subjected to Resistance-value ("R-value") testing in accordance with California Test 301. The results of the R-value test, which were used in the pavement design, are presented in Figure B8.

One bulk sample of representative near-surface soils was submitted to Sunland Analytical Lab, Inc. of Rancho Cordova, California to determine the soil pH and minimum resistivity (California Test 643), Sulfate concentration (California Test 417 and ASTM D516) and Chloride concentration (California Test 422). The results of these tests are presented in Figures B9 and B10.

MOISTURE-DENSITY RELATION



Material: Brown, silty sand

Location: D1 (1'-5')

Test Method: ASTM D1557 "A"

Maximum Dry Unit Weight - PCF: 133

Optimum Moisture - %: 8.0

WallaceKuhl & ASSOCIATES

COMPACTION CURVE

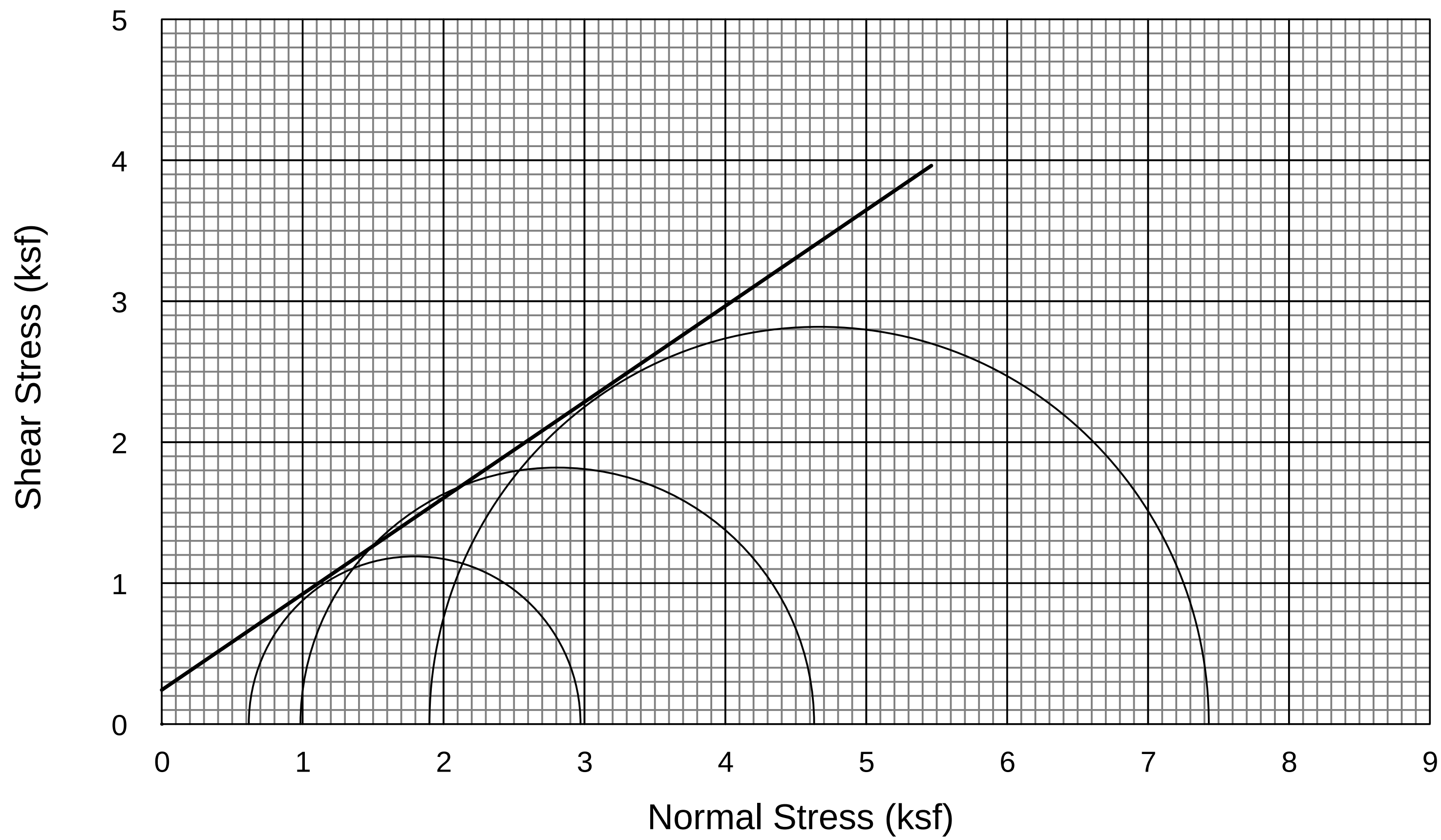
AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	B1
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO.12456.01P	

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. : D4-1I

SAMPLE CONDITION : Undisturbed

SAMPLE DESCRIPTION : Silty sand

DRY DENSITY (PCF) : 110
INITIAL MOISTURE (%) : 11.3
FINAL MOISTURE (%) : 15.9

ANGLE OF INTERNAL FRICTION (ø) : 34°
COHESION (PSF) : 235

WallaceKuhl & ASSOCIATES

TRIAXIAL COMPRESSION TEST RESULTS

AMERICAN RIVER COLLEGE CORPORATION YARD

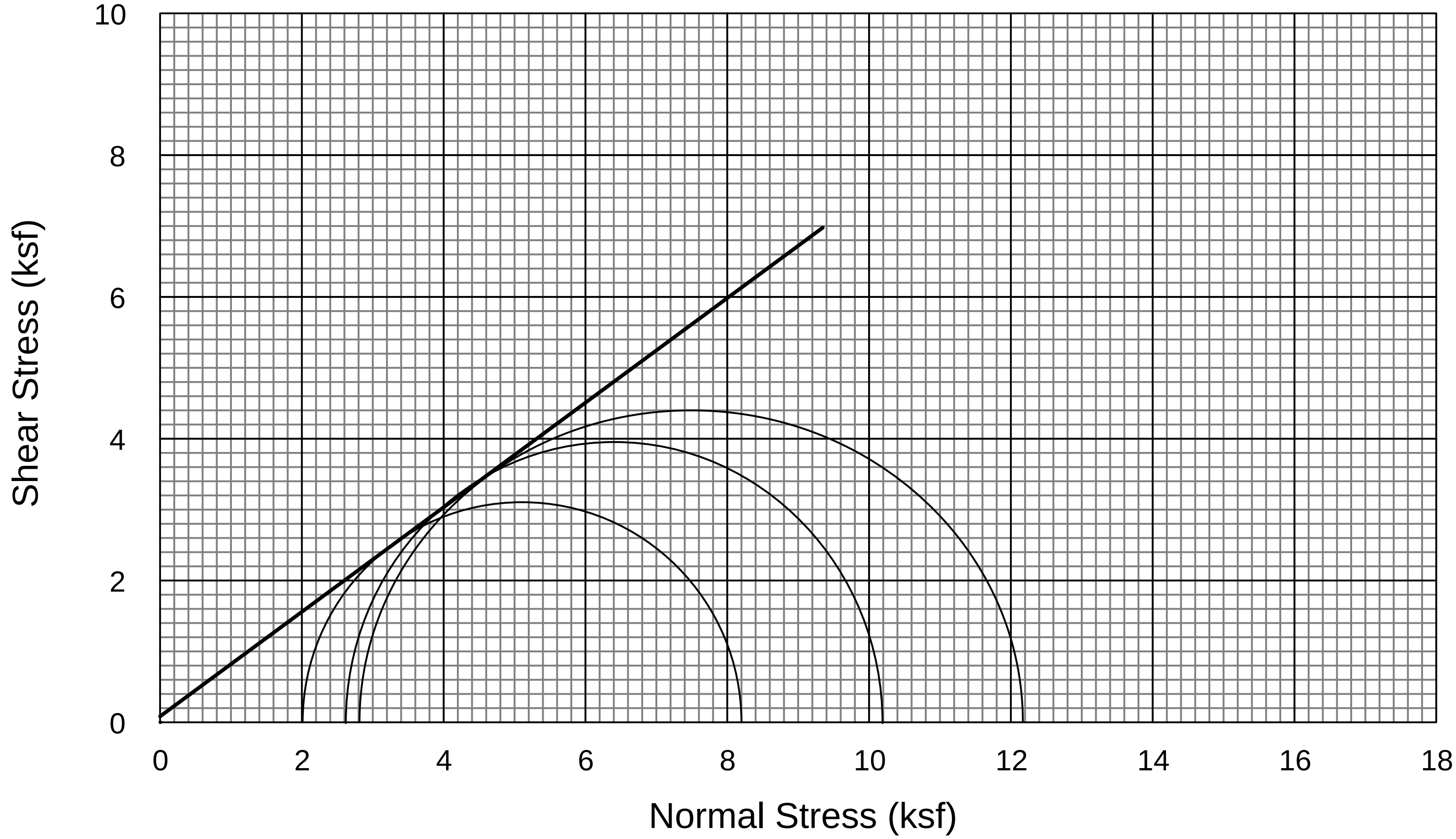
Sacramento, California

FIGURE	B2
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19

WKA NO.12456.01P

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. : D4-2I

SAMPLE CONDITION : Undisturbed

SAMPLE DESCRIPTION : Silty sand

DRY DENSITY (PCF) : 118
INITIAL MOISTURE (%) : 9.6
FINAL MOISTURE (%) : 14.9

ANGLE OF INTERNAL FRICTION (ø) : 36°
COHESION (PSF) : 80

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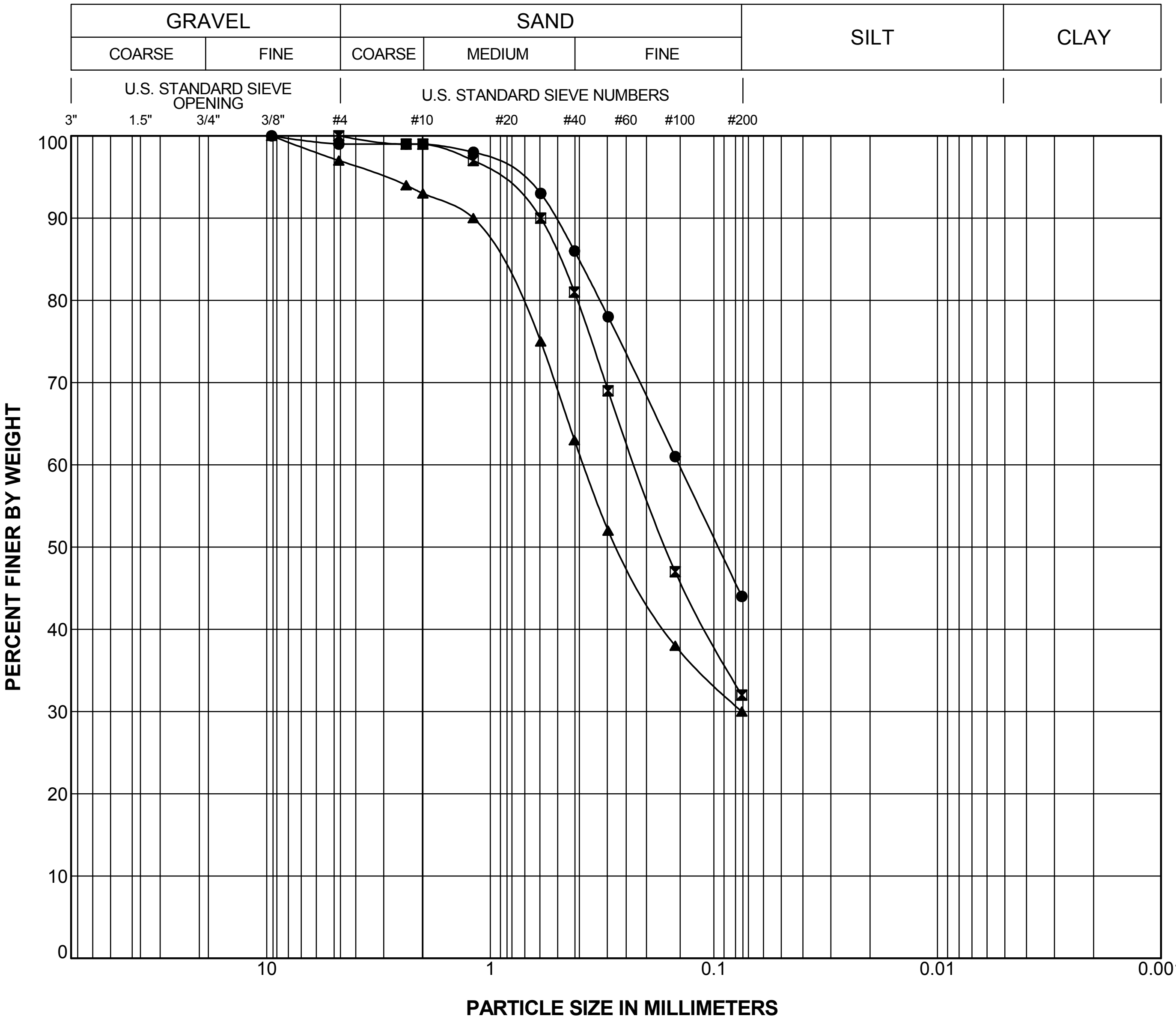
TRIAXIAL COMPRESSION TEST RESULTS

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	B3
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO.12456.01P	



Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
D1	D1 (1' - 5')	SM	1.0' - 5.0'	●			silty SAND
D2	D2 (1' - 5')	SM	1.0' - 5.0'	⊠			silty SAND
D4	D4 (1' - 5')	SM	1.0' - 5.0'	▲			silty SAND

PARTICLE SIZE DISTRIBUTION

Project: American River College Corporation Yard
WKA No. 12456.01P

FIGURE B4

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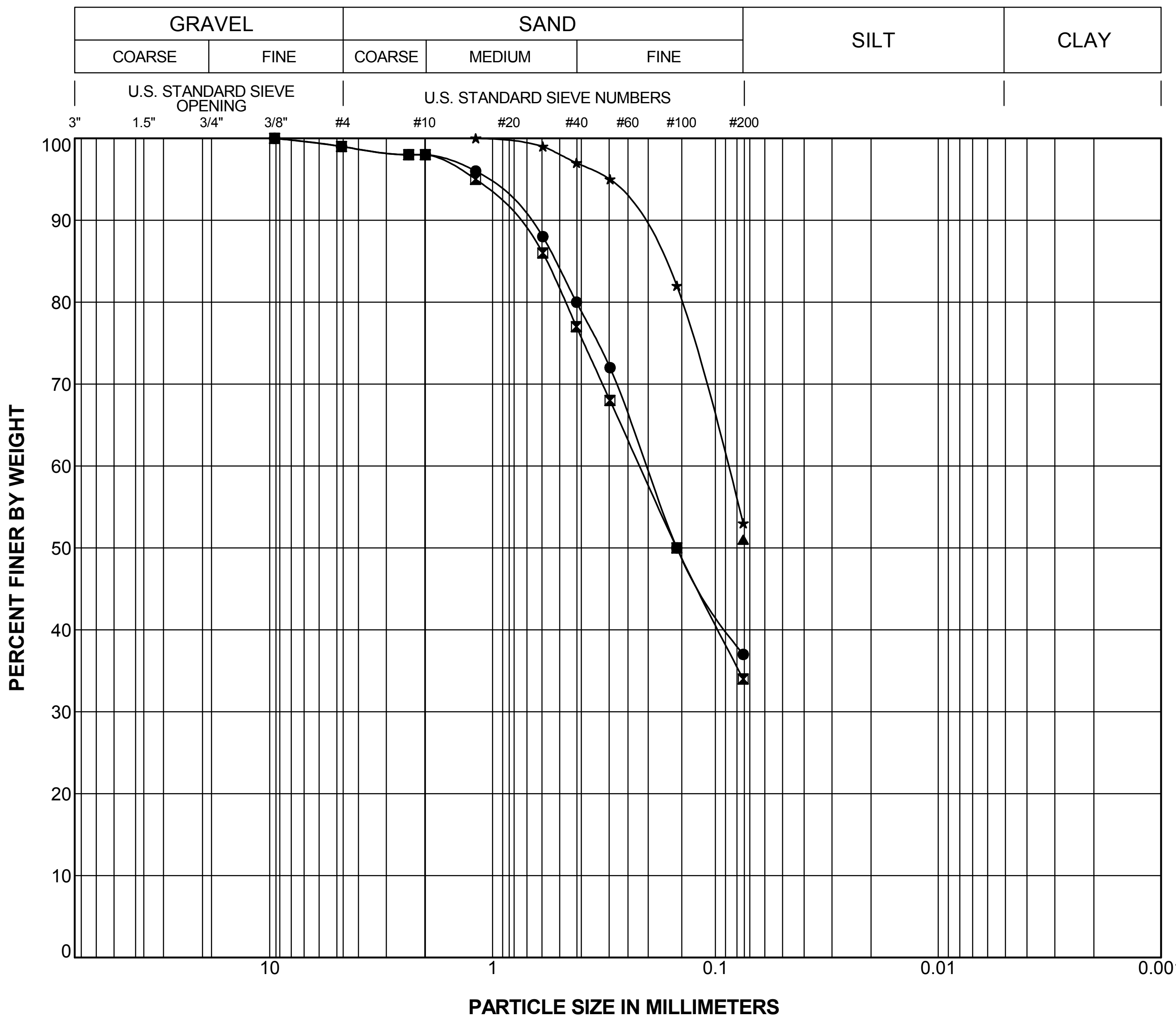

Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
D5	D5-1I	SM	2.0' - 3.5'	●			silty SAND
D5	D5-2I	SM	5.0' - 6.5'	⧗			silty SAND
D5	D5-5I	CL-ML	20.0' - 21.5'	▲	27	5	sandy, silty CLAY
D5	D5-7I	ML	30.0' - 31.5'	★			sandy SILT

PARTICLE SIZE DISTRIBUTION

Project: American River College Corporation Yard
WKA No. 12456.01P

FIGURE B5

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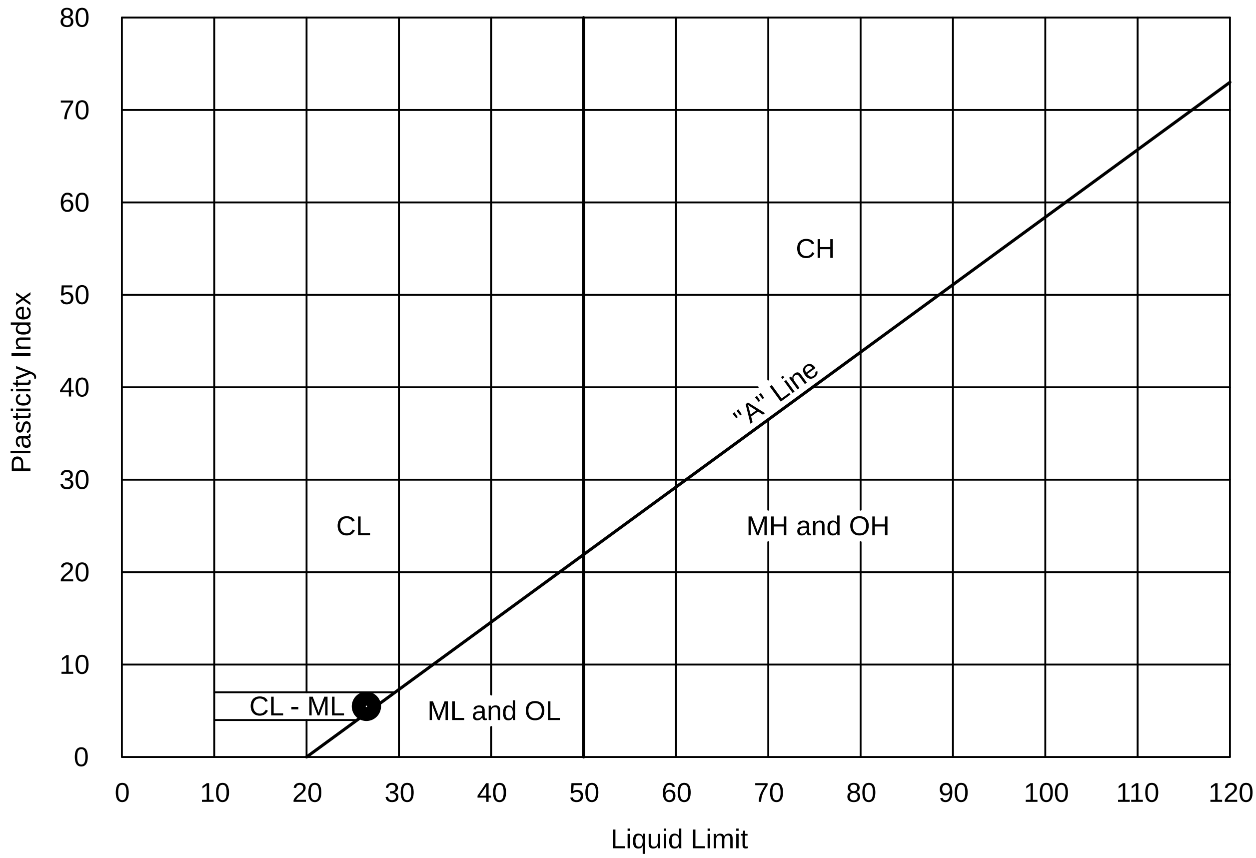
ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D5-5I	20' - 21.5'	---	27	5	51	CL - ML

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ATTERBERG LIMITS

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	B6
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO.12456.01P	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Brown, silty sand

LOCATION: D2 (1' - 5')

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
1' - 5'	8.0	11.6	119	**0**

CLASSIFICATION OF EXPANSIVE SOIL *

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

* From ASTM D4829, Table 1

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EXPANSION INDEX

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	B7
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Brown, silty sand

LOCATION: B1 (1'-5')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
10	133	7.2	480	0	0	72
11	131	7.9	236	0	0	25
12	131	7.5	315	0	0	37

* R-Value at 300 psi Exudation Pressure: **34**

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RESISTANCE VALUE TEST RESULTS

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	B8
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19

WKA NO. 12456.01P

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/30/2019
Date Submitted 08/26/2019

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12456.01P ARC CORP Y Site ID : D2@1-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 80460-168154.

EVALUATION FOR SOIL CORROSION

Soil pH	6.12	
Minimum Resistivity	4.56 ohm-cm (x1000)	
Chloride	0.8 ppm	00.00008 %
Sulfate	5.2 ppm	00.00052 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

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CORROSION TEST RESULTS

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

FIGURE	B9
DRAWN BY	JBV
CHECKED BY	ML
PROJECT MGR	DRG
DATE	10/19
WKA NO. 12456.01P	

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/30/2019
Date Submitted 08/26/2019

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12456.01P ARC CORP Y Site ID : D2@1-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 80460-168155.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	5.9	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

CORROSION TEST RESULTS	FIGURE	B10
AMERICAN RIVER COLLEGE CORPORATION YARD	DRAWN BY	JBV
Sacramento, California	CHECKED BY	ML
	PROJECT MGR	DRG
	DATE	10/19
	WKA NO. 12456.01P	

WallaceKuhl & ASSOCIATES

APPENDIX C

Guide Earthwork Specifications

APPENDIX C

GUIDE EARTHWORK SPECIFICATIONS

AMERICAN RIVER COLLEGE CORPORATION YARD

Sacramento, California

WKA No. 12456.01P

PART 1: GENERAL

1.1 SCOPE

A. General Description

This item shall include all clearing of existing surface and subsurface structures, pavements, utilities, vegetation, rubbish, rubble, 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 site 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 the Geotechnical Engineer or his or her representative.

1.2 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by 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. Measures shall be taken to protect storm drains in adjacent depressed areas such that minimum siltation occurs in the drainage system.

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

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

1.3 <u>GEOTECHNICAL REPORT</u>

A. A *Geotechnical Engineering and Geologic Hazards Report* (WKA No. 12456.01P, dated October 3, 2019) has been prepared for this site by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California [(916) 372-1434]. A copy is available for review at the office of Wallace - Kuhl & Associates.

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

1.4 <u>EXISTING SITE CONDITIONS</u>

The Contractor shall become 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 <u>SEASONAL LIMITS</u>

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 that do not contain significant quantities of rubble, rubbish and vegetation, and having been tested and approved by the Geotechnical Engineer prior to use.

B. Clayey soil, if encountered during earthwork operations, shall not be used as fill material within the upper two feet (2') of final subgrades to support interior and exterior concrete slabs and pavements.

C. Imported fill materials shall be approved by the Geotechnical Engineer; they shall meet the above requirements; shall have a Plasticity Index not exceeding fifteen (15) when tested in accordance with ASTM D4318, an 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 materials to be used in pavement areas shall have a Resistance ("R") value greater than thirty (30). Import materials also shall not contain known contaminants and be within acceptable corrosion limits, with appropriate documentation provided by the contractor.

D. Capillary barrier material under floor slabs shall be provided to the thickness shown on the Drawings. This material shall be clean gravel or crushed rock of one-inch (1") maximum size, with less than five percent (5%) material passing a Number Four (#4) sieve.

E. Other products, such as aggregate base, asphalt concrete and related asphaltic seal coats, tack coat, etc., shall comply with the appropriate provision of the State of California (Caltrans) Standard Specifications, latest edition.

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 prior to beginning actual earthwork operations.

3.2 CLEARING, STRIPPING, AND PREPARING BUILDING PAD AND OTHER STRUCTURAL AREAS

A. All surface and sub-surface items associated with existing site development, including utilities and associated backfill, vegetation, debris, and other items encountered during site work and deemed unacceptable by the Geotechnical Engineer, shall be removed and disposed of so as to leave the disturbed areas with a neat and finished appearance, free from unsightly debris. All demolition debris shall be hauled off site, or used as engineered fill, provided it is processed per the recommendations in *Geotechnical Engineering and Geologic Hazards Report*.

B. Excavations and depressions resulting from the removal of such items, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soils and backfilled with suitable materials in accordance with these specifications.

C. If light poles associated with the existing parking lot are to be removed, the voids shall be backfilled with an approved sand-cement grout up to two feet (2') below the final soil subgrade elevation, and topped off with engineered fill placed and compacted in accordance with these specifications.

D. All structural areas (building pad, pavements, exterior flatwork, etc.) shall be stripped of vegetation and organically laden topsoil. With prior approval from the Geotechnical Engineer, strippings may be used in landscaped areas, provided they are kept at least five feet (5') from buildings pads and other surface improvements, moisture conditioned and compacted.

E. Trees and bushes designated for removal shall include the rootball and all surface roots larger than one-half inch (½") in diameter. Adequate removal of debris and roots may require laborers and handpicking to clean the subgrade soils to the satisfaction of the Geotechnical Engineer's on-site representative, prior to further site preparation.

F. Following site clearing activities, all construction areas, including the building pad, exterior flatwork and pavement areas, shall be over-excavated as described in the *Geotechnical Engineering and Geologic Hazards Report*.

G. After over-excavation operations have been performed, the Geotechnical Engineer’s representative shall evaluate the exposed subgrade to determine if additional over-excavation is required.

H. Following over-excavation and evaluation of the exposed subgrade, as described in the *Geotechnical Engineering and Geologic Hazards Report*, the exposed subgrade within the construction areas shall be scarified to a depth of at least twelve inches (12”), thoroughly moisture conditioned to at least the optimum moisture content, and uniformly compacted to at least ninety percent (90%) of the maximum dry density as determined by the ASTM D1557 Test Method.

I. Compaction operations for all soil subgrades shall be undertaken with a heavy, self-propelled, sheepsfoot compactor capable of achieving the compaction requirements included in the *Geotechnical Engineering and Geologic Hazards Report*.

J. When the moisture content of fill material is less than the optimum moisture content as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

K. 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.

L. Site clearing and subgrade preparation operations shall extend at least five feet (5’) beyond the edge of exterior foundations or the footprint of the planned building, whichever is greater, and also at least five feet (5’) beyond any exterior flatwork areas, pavements and any other structural areas, where practical.

M. 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 saturated soils and 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 CONSTRUCTION OF SUBGRADES

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

B. Organically laden topsoil shall not be reused as engineered fill.

C. When the moisture content of the fill material is less than the optimum moisture content as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

D. 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.

E. After each layer of fill has been placed, mixed and spread evenly, each layer of fill shall be thoroughly compacted to at least ninety percent (90%) as determined by the ASTM D1557 Compaction Test. Compaction shall be undertaken with equipment 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.

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

3.4 CONSTRUCTION OF FINAL SUBGRADE

A. Final building pad and exterior flatwork subgrades shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. Clayey soil, if encountered during earthwork, shall not be used as fill material within the upper two feet (2') of final subgrades to support interior and exterior concrete slabs.

B. Final subgrade for pavements shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. Clayey soil, if encountered during earthwork, shall not be used as fill material within the upper two feet (2') of final subgrades to support pavements. The upper six inches (6") of final pavement subgrades shall be brought to at least the optimum moisture content and uniformly compacted to at least ninety-five percent (95%) as determined by ASTM D1557 Compaction Test.

3.5 TESTING AND OBSERVATION

A. Site clearing and 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, the necessary readjustments shall be made by the Contractor 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.

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