Appendix B:
Geotechnical Investigation

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Geotechnical Engineering and
Geologic Hazards Investigation

SOLANO COMMUNITY COLLEGE –
NEW SCIENCE BUILDING

Fairfield, California
WKA No. 10642.01P
September 4, 2015

Prepared For:
Solano Community College District
360 Campus Lane
Fairfield, California 94534

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Geotechnical Engineering and Geologic Hazards Investigation
SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING
4000 Suisun Valley Road
Fairfield, California
WKA No. 10642.01P
September 4, 2015

INTRODUCTION

We have completed a geotechnical engineering and geologic hazards study for the proposed science building to be constructed at the central portion of the Solano Community College campus located at 4000 Suisun Valley Road in Fairfield, California (see Figure 1). The purpose of this study has been to explore the existing soil, geologic, and groundwater conditions at the site, and to provide geologic hazards and geotechnical engineering conclusions and recommendations regarding design and construction of the proposed science building and associated improvements. This report presents the results of our study.

Scope of Services

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

1. Perform a site reconnaissance;
2. Review of United States Geological Survey (USGS) topographic maps, historical aerial photographs and available groundwater information;
3. Review of geologic maps and fault maps;
4. Review of seismic activity within 100 kilometers (62 miles) of the site;
5. Perform subsurface explorations, including the drilling and sampling of six borings to depths ranging from approximately 10 to 51½ feet below the existing site grades;
6. Collect representative bulk samples of near-surface soils from the proposed building pad areas;
7. Perform laboratory testing of selected soil samples;
8. Perform engineering analyses; and,
9. Prepare this report.

Figures and Attachments

The following figures are included with this report:

Table 1: Figures

Figure	Title	Figure	Title
1	Vicinity Map	7	Epicenter Map
2	Site Plan	8 through 13	Logs of Soil Boring D1 through D6
3	USGS Topographic Map	14	Unified Soil Classification System
4	Geologic Map	15	FEMA Flood Map
5	Geologic Cross Section	16	Dam Inundation Map
6	Fault Map		

Appended to this report are:

- General information regarding project concepts, exploratory methods used during our field investigation and laboratory test results not included on the Logs of Soil Borings (Appendix A).
- A list of references cited (Appendix B).
- Liquefaction analysis results and seismic hazard deaggregation (Appendix C).
- *Guide Earthwork Specifications* that may be used in the preparation of contract documents (Appendix D).

Proposed Development

Based on our review of a *Preliminary Floor Plan*, dated July 8, 2015, provided to us by Kitchell (Kitchell, 2015), site development will cover a total area of approximately 58,700 square feet (sf); however, the construction the new science building will only encompass a total area of approximately 30,400 sf. Based on conversations with Mr. Eric Berger of Kitchell, we understand the new building will be one-story, constructed of steel-framing with an interior concrete slab-on-grade floor, and will be divided into several classrooms and laboratories. Structural loads for the science building were not available when this report was prepared; however, the loads are anticipated to be relatively light based on this type of construction. Heavy floor loads and below grade basements are not anticipated for the science building. Associated development will include construction of underground utilities, landscaping, an outdoor classroom area, a courtyard, exterior flatwork, and asphalt concrete pavements. The layout of the planned improvements is shown on Figure 2.

A grading plan was not available when this report was prepared. However, based on the existing relatively flat topography of the site and our understanding of the proposed construction, we anticipate cuts and fills on the order of about one to three feet for development of the new science building.

Supplemental Information

Supplemental information used in the preparation of this report included review of the following reports prepared by our firm and others for the subject Solano Community College campus and projects in the vicinity of the project site:

- Harding, Miller, Lawson & Associates, *Soil Investigation Solano College Near Fairfield* (HMLA No. 4101.2, dated February 16, 1968), prepared for the subject Solano Community College campus, which included the project site;
- Wallace-Kuhl & Associates, *Geologic Hazards Report* (WKA No. 9469.01P, dated June 27, 2012), prepared for the Rockville Road School site, located about 2,000 feet northeast of the project site;
- Wallace-Kuhl & Associates, *Geotechnical Engineering Report* (WKA No. 10031.01, dated April 15, 2014), prepared for the Fairfield Commons apartments, located about a 1,300 feet southeast of the project site;
- Ninyo& Moore, *Geotechnical Evaluation and Geologic Hazards Assessment* (NM Project No. 402202001, dated September 18, 2013), prepared for Modernization and Expansion of Building 600 at the subject Solano Community College campus, located about 500 west of the project site; and,
- Ninyo & Moore, *Geotechnical Evaluation and Geologic Hazards Assessment* (NM Project No. 402329001, dated May 28, 2014), prepared for New Building P2 and Building 1200 Theater Renovation at the subject Solano Community College campus, located about 400 northwest of the project site.

FINDINGS

Site Description

The subject Solano Community College campus is located at 4000 Suisun Valley Road in Fairfield, California. The proposed science building site is located in the central portion of the campus and is bounded to the north by a landscaped area and exterior flatwork, beyond which is Building 1400; to the east by a landscaped area and exterior flatwork, beyond which is

Building 1700A; to the south by Building 1500; and, to the west by a landscaped area and exterior flatwork, beyond which is Building 100.

At the time of our field exploration on July 30, 2015, the site was vacant, covered with sod and enclosed with exterior concrete walkways. Several metal picnic tables and relatively large landscape boulders were observed in the southern portion of the site. Mature trees were observed in the northwest, southeast and southwest corners of the site. Light poles were observed in the northeast, northwest and southern portions of the site. Evidence of several underground utilities was observed across several areas of the site, as shown on Figure 2. Please note the existing utility alignments shown on Figure 2 were provided by Mr. Eric Berger of Kitchell; irrigation lines are not shown.

The site is located at approximately 38.2350º north latitude and 122.1224º west longitude. The site is relatively flat, and based on review of the topographic map of the *Fairfield South, California Quadrangle*, published by the USGS, dated 1949 (photorevised 1980), the elevation of the site is approximately +40 feet relative to mean sea level (msl). A portion of the USGS topographic map containing the site is presented as Figure 3.

Historical Aerial Photographic Review

We reviewed historical aerial photographs of the site available from our files and the Google Earth software (Google Earth, 2015). Available photographs were taken in the years 1962, 1993, and 2002 through 2015.

Review of the photograph taken in 1962 shows portions of the campus to be vacant land, while other portions support orchards. We understand the original campus was constructed in 1971.

Review of the photograph taken in 1993 shows the campus generally as observed during our field exploration. The area for the proposed science building is covered with sod and mature trees are shown in the corners of the building site. Review of the remaining photographs shows the site has generally remained unchanged since 1993.

General Site Geology

The site is located in the Vaca Mountains in the Coast Ranges geomorphic province of California. The Coast Ranges of California are generally composed of two northwest trending mountain ranges, located north and south of the San Francisco Bay depression. Rock units within the northern Coast Range geomorphic province consist of Mesozoic to Cenozoic marine sedimentary rocks. These sediments have been uplifted, terraced, and wave-cut. The sedimentary rocks on the eastern portion of the Coast Ranges dip at moderate angles to the east (Norris and Webb, 1990).

Surface elevations within the Coast Ranges generally range from several feet below mean sea level to more than 6,000 feet above sea level.

The site is mapped as underlain by Holocene-aged alluvial fan deposits (Qhf), based on review of available published geologic maps of the Quaternary Deposits in the Central San Francisco Bay Region, California (Bezore, et al, 1998, Witter, et al., 2006; see Figure 4). The sediments were probably derived from the eastern slopes of the California Coast Ranges and deposited by the Suisun Creek and alluvial terraces and fans. The alluvial fan deposits are likely underlain by bedrock of the Pliocene Sonoma Volcanics that are exposed immediately west of the site.

The geologic deposits mapped on the site are consistent with the soils data obtained from the current and previous subsurface investigations performed at the campus. A geologic cross section is included in this report as Figure 5.

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 Geologic and Seismic Hazards discussion in the Health and Safety Element of the *Solano County General Plan* (Solano County, 2008) indicates that the site is in an area having the "highest potential for earthquake damage". The site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 2007). However, the site is located within close proximity of several surface faults that are presently zoned as active or potentially active by the California Geological Survey (CGS) pursuant to the guidelines of the Alquist-Priolo Earthquake Fault Zoning Act (Jennings, 2010; Hart and Bryant, 2007). The nearest Alquist-Priolo Earthquake Fault Zone has been established around the Cordelia fault. Based on review of the *California Special Studies Zone* maps of the *Cordelia and Fairfield South Quadrangles* (CDMG, 1993a and 1993b). The edge of the fault zone is located approximately 0.8 kilometers (0.5 miles) west of the site. The Cordelia fault is part of the Concord/Green Valley fault zone.

Using the *Revised 2002 California Probabilistic Seismic Maps* (Cao, et al, 2003), we have prepared Table 2 containing faults and fault systems within 100 kilometers (62miles) 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 6. The nearest of these faults are associated with the Concord/Great Valley Fault System, which trend north-northwest to south-southeast, and are located approximately 1.4 kilometers (0.9 miles) west of the site.

Table 2: Faults Influential to the Site

Fault Name	Distance		Maximum Magnitude (M_w)
	Miles	kilometers	
CONCORD/GV (CON+GVS)	0.5	1.0	6.6
CONCORD/GV (GVS+GVN)	0.5	1.0	6.5
CONCORD/GV (CON+GVS+GVN)	0.5	1.0	6.7
GREAT VALLEY 5	7.1	11.4	6.5
GREAT VALLEY 4	7.1	11.4	6.6
WEST NAPA	8.1	13.0	6.5
HUNTING CREEK - BERRYESSA	15.6	25.1	7.1
HAYWARD (HN+RC)	19.8	31.9	7.1
HAYWARD (HS+HN)	19.8	31.9	6.9
HAYWARD (RC)	19.8	31.9	7.0
HAYWARD (FLOATING)	19.8	31.9	6.9
HAYWARD (HS+HN+RC)	19.8	31.9	7.3
HAYWARD (HN)	19.8	31.9	6.5
MOUNT DIABLO (MTD)	21.7	34.9	6.7
GREAT VALLEY 3	27.7	44.6	6.9
HAYWARD (HS)	28.6	46.0	6.7
GREENVILLE (GN)	28.9	46.5	6.7
CALAVERAS (CS+CC+CN)	29.5	47.5	6.9
CALAVERAS (CN)	29.5	47.5	6.8
SAN ANDREAS (FLOATING)	37.6	60.5	6.9
SAN ANDREAS (SAP+SAN+SAO)	37.6	60.5	7.8
SAN ANDREAS (SAS+SAP+SAN+SAO)	37.6	60.5	7.9
SAN ANDREAS (SAN+SAO)	37.6	60.5	7.7
SAN ANDREAS (SAP+SAN)	37.6	60.5	7.7
SAN ANDREAS (SAN)	37.6	60.5	7.5
SAN ANDREAS (SAS+SAP+SAN)	37.6	60.5	7.8
MAACAMA - GERBERVILLE	38.9	62.6	7.5
SAN ANDREAS (SAS+SAP)	39.6	63.7	7.4
SAN ANDREAS (SAP)	39.6	63.7	7.2
POINT REYES	40.6	65.3	7.0
SAN GREGORIO (SGN)	41.9	67.4	7.2
SAN GREGORIO (FLOATING)	41.9	67.4	6.9
SAN GREGORIO (SGS+SGN)	41.9	67.4	7.4

Fault Name	Distance		Maximum Magnitude (M_w)
	Miles	kilometers	
GREENVILLE (GS)	45.4	73.1	6.6
GREENVILLE (GS+GN)	45.4	73.1	6.9
GREAT VALLEY 7	47.4	76.3	6.7
COLLAYOMI	48.3	77.7	6.5
BARTLETT SPRINGS FAULT SYSTEM	52.5	84.5	7.6
MONTE VISTA - SHANNON	55.6	89.5	6.7

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). Based on review of currently published maps available at the CGS website, there are no geologic hazards maps for the City of Fairfield.

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 December 2012). 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 (62 miles) was specified for this analysis. A historic earthquake epicenter map is presented as Figure 7.

An examination of the tabulated data suggests that the site has experienced ground shaking equivalent to Modified Mercalli Intensity (MMI) VIII[1]. According to the tabulated data, the most intense earthquake ground shaking within 100 kilometers of the site resulted from the M_R (Richter Scale Magnitude) 8.25 San Francisco earthquake of April 18, 1906, with an epicenter located approximately 68.0 kilometers (42.3 miles) southwest of the site.

[1] Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.

The closest earthquake to the site is indicated to be an M_R 4.0 earthquake that occurred on May 21, 1902, with an epicenter located approximately 11.3 kilometers (7.1 miles) southeast of the site. Other notable events include: the M_R 6.4 Winters Vacaville earthquake of April 19, 1892, with an epicenter located approximately 21.2 kilometers (13.2 miles) east of the site; and the M_W 6.0 Napa earthquake of August 24, 2014, located approximately 16.7 kilometers (10.4 miles) west of the site.

Subsurface Soil Conditions

Six exploratory borings (D1 through D6) were performed on July 30, 2015 at the approximate locations shown on the Site Plan, Figure 2. Generally, the surface and near-surface soils at our borings consisted of stiff to hard, silty and sandy clay to depths ranging from 10 to 24½ feet below existing site grades. Beneath the clay soils, we generally encountered medium dense sand mixed with varying amounts of silt to a depth of about 30 feet below existing site grades, underlain by very stiff to hard, silty and sandy clay to the maximum explored depth of 51½ feet below existing site grades. These soil conditions are generally consistent with those encountered in previous studies performed for the college campus and the general vicinity of the site, and also with the mapped geology.

Artificial fill was encountered in Borings D2, D5 and D6 to depths ranging from about three to 4½ feet below existing site grades. The fill consisted of very stiff, clayey silt, very stiff silty clay and medium dense, silty sand. The fill soils encountered in Boring D2 are likely associated with trench backfill for adjacent underground utilities. The source of the fill soils encountered in Borings D5 and D6 is undetermined.

Review of the borings included in the geotechnical report prepared for the original construction of the college campus indicates soil conditions beyond a depth of 51½ feet generally consist of alternating layers of stiff, sandy and silty clay, medium dense, clayey sand and dense, sandy gravel to a depth of about 85 feet below previous site grades (HMLA No. 4101.2).

For specific information regarding the soil conditions at a specific location, please refer to the Logs of Soil Borings, Figures 8 through 13.

Groundwater

Groundwater was observed in Borings D1 and D3 at a depth of approximately 11 feet below existing site grades on July 30, 2015. These borings were left open for several hours; however, the borings may not have been left open long enough for groundwater to reach static equilibrium.

The subject Solano Community College campus is located within the Suisun-Fairfield valley basin, in the San Francisco Bay Hydrological Region, as defined by the California Department of Water Resources (DWR) Groundwater Bulletin 118 (DWR, 2003).

To supplement our groundwater data, we reviewed available groundwater information at the California Department of Water Resources (DWR) website (DWR, 2014). The DWR periodically monitors groundwater levels in wells across the state. Their website shows a well located approximately ½ of a mile southeast of the site. The well is identified as Well No. 04N02W06A001M with a ground surface elevation of +38 feet msl, similar to the subject site. Valid groundwater data for this well was recorded from January 21, 1920 to at least June 27, 1985. Data shows the highest recorded groundwater elevation was about +34 feet msl at the well (about four feet below the ground surface at the well) on May 29, 1974. The lowest recorded groundwater elevation was about -2 feet msl at the well (about 40 feet below the ground surface at the well) on August 3, 1921. Therefore, groundwater depths at the site have likely ranged from approximately six to 42 feet below site grades. Please note that other groundwater data was recorded for this DWR well; however, the data is identified as "questionable data" and was not considered for this project.

The groundwater conditions described above are consistent with the groundwater levels observed during our field exploration and previous studies performed for the college campus and in the general vicinity of the site.

CONCLUSIONS

Seismic Site Class

Based on the soil conditions encountered at the boring locations and our experience with similar soil conditions within the vicinity of the site, the soils at this site can be designated as Site Class D in determining seismic design forces for this project in accordance with Section 1613A.3.2 of the 2013 California Building Code (CBC), which references Chapter 20 of American Society of Civil Engineers (ASCE) Standard 7-10.

2013 CBC/ASCE 7-10 Seismic Design Criteria

Section 1613A of the 2013 edition of the CBC references ASCE Standard 7-10 for seismic design. The seismic design parameters provided in Table 3 are based on the site latitude and longitude using the United States Seismic Design Maps public domain computer program developed by the USGS (USGS, 2014). The 2013 CBC parameters provided below should be used for seismic design of the proposed science building.

Table 3: 2013 CBC/ASCE 7-10 Seismic Design Parameters

Latitude: 38.2350° N Longitude: 122.1224° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2 second	Figure 22-1	Figure 1613.3.1(1)	S_S	2.108 g
1.0 second Period MCE	Figure 22-2	Figure 1613.3.1(2)	S_1	0.746 g
Soil Class	Table 20.3-1	Section 1613.3.2	Site Class	D
Site Coefficient	Table 11.4-1	Table 1613.3.3(1)	F_a	1.000
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.500
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-37	S_{MS}	2.108 g
	Equation 11.4-2	Equation 16-38	S_{M1}	1.118 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	1.405 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.746
Seismic Design Category	Table 11.6-1	Section 1613.3.5(1)	Risk Category I to IV	D
	Table 11.6-2	Section 1613.3.5(2)	Risk Category I to IV	D

Notes:
MCE = Maximum Considered Earthquake
g = gravity

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 results of our subsurface soil exploration at the site indicates the underlying soils generally consist of clays with interbedded sand layers extending to the explored depth of 51½ feet below the existing ground surface. Laboratory testing performed on the clay layers encountered at Boring D3 indicates these soils are not susceptible to liquefaction based on published literature (i.e., soils not susceptible to liquefaction if water content to liquid limit ratio [w_c/LL] < 0.85 and plasticity index [PI] > 12, Bray & Sancio, 2006). However, relatively loose, silty sand and poorly graded sand was encountered at Boring D3 from about 18 to 30 feet below the ground surface locations and historical high groundwater is indicated to be about six feet below the existing ground surface. These site conditions require than an evaluation of the liquefaction potential be performed at school sites per CGS Note 48 and Special Publication 117.

A liquefaction analysis to determine factors of safety against liquefaction was performed for the soil and groundwater conditions encountered at Boring D3.

<u>Liquefaction Analysis and Results</u>

We performed a liquefaction analysis of data obtained from the SPT blow counts measured in the hollow stem auger/rotary wash boring performed at the site for this evaluation. The boring was analyzed using LiqIT (Version 4.7), developed by GeoLogismiki, and the liquefaction analyses was performed utilizing the results of the National Center for Earthquake Engineering Research (NCEER) liquefaction evaluation methods summarized by Youd, et al (2001). A historical high static groundwater level of approximately six feet below the existing ground surface was used in our analysis, based on our review of historic groundwater levels near the site. A peak ground acceleration (PGA) of 0.80 g was used in the liquefaction analysis, based on Equation 11.8-1 of ASCE 7-10. A mode magnitude earthquake of 6.6 was used for this analysis using the 2008 USGS National Seismic Hazard Mapping Project (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation web site.

The results of the liquefaction analysis indicates the granular soil layer encountered in Boring D3 at a depth of 18 to 30 feet below existing site grades has a factor of safety against liquefaction below 1.3. A factor of safety below 1.3 requires a liquefaction-induced settlement analysis. Copies of the liquefaction analysis results performed for this investigation are presented in Appendix C.

Seismically Induced Settlement

Post-liquefaction settlement calculations within LiqIT are performed using the methodology of Ishihara and Yoshimine (1992). Given the results of our analysis performed for this investigation, the worst-case estimate of total and differential post-liquefaction settlement is calculated to be about 3¾ inches total seismic induced settlement and about two inches of differential settlement across 50 feet, or the shortest dimension of the structure, whichever is less. These estimates of post-liquefaction seismic settlements represent free-field ground settlement, not settlement of the proposed structure. The presence of stiff soils and interbedded clay layers near the ground surface will likely mitigate the impact of seismically induced settlement at the ground surface.

In our opinion, the science building should be designed to comply with California Administrative Code, Title-24, Section 4-301 to repairable architectural and structural damage from “worst-case scenario” total seismic settlements of 3¾ inches and differential settlements of two inches across 50 feet, or the shortest dimension of the structure, whichever is less.

Liquefaction potential at the site was also evaluated based on the Liquefaction Potential Index (LPI). The LPI is a measure of the liquefaction potential based on an analysis of the entire vertical soil profile not just discrete layers (Iwasaki, 1986; Toprak and Holzer, 2003). Factors taken into consideration for the LPI calculations include: thickness of the liquefied layer; proximity of the liquefied layer to the surface; and, the factor of safety. The LPI ranges from 0 to 100 with the value zero representing no liquefaction potential. Surface manifestations of liquefaction occur at LPI ≥ 5. The LPI for the soil conditions at Boring D3 was calculated to be 35.93, indicating liquefaction is likely during the design seismic event (mode magnitude earthquake of 6.6 and a PGA of 0.80g).

Based on the soil conditions encountered at the site and our liquefaction analysis, including LPI evaluations, it is our professional opinion that the potential for liquefaction of the soils beneath the site is moderate if the site experiences significant ground shaking during an earthquake.

Seismic Hazards

No active or potentially active faults are known to underlie the 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, but strong ground shaking should be anticipated during the design of the project improvements.

Volcanic Hazards

The site is not located within a volcanic hazard zone (e.g., pyroclastic flow, volcanic debris flow, lava flow, bas 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

The topography across the site is relatively flat based on visual observations and review of topographic maps. The USGS Topographic Map of the *Fairfield South, California Quadrangle* (USGS, 1980) indicates the surface elevation at the site is approximately +40 feet msl. Also, review of the Health and Safety Element of the *Solano County General Plan* (Solano County, 2008) revealed the site is not considered to be in an area of landslide potential. Based on the flat topography of the site and the lack of slopes in the vicinity of 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 for the City of Fairfield and Solano County, California (Community-Panel Numbers 06095C0451E, May 4, 2009, and subsequent Letter of Map Corrections or LOMCs), the site is located within ZONE X defined as "Areas of 0.2% annual chance of flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas with less than 1 square mile; and areas protected by levees from 1% annual chance flood" (see Figure 15).

Review of Figure HS-1 contained within the Health and Safety Element of the *Solano County General Plan* (Solano County, 2008) revealed that the site lies outside of a 100-year flood hazard area.

Dam Inundation

According to the Health and Safety Element of the *Solano County General Plan* (Solano County, 2008) there are 10 dams in Solano County that have the potential for human injury or loss of life in the event of failure. The California Office of Emergency Services has identified that the failure of dams at Lake Curry, Lake Frey, and Lake Madigan as having the potential to cause property damage, injury, or loss of human life in the Fairfield area.

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 very unlikely.

Subsidence and Hydrocollapse

Subsidence occurs when a large land area settles due to extensive withdrawal of groundwater, oil, natural gas or oxidation of peat. Based on subsurface sampling, the soil at the project site predominately consist of stiff to hard, silty clay to the maximum explored depth of 51½ feet below existing site grades. No documents indicate the site is subject to ground subsidence as a result of groundwater pumping or withdrawal of gas or oil. Review of the Health and Safety Element of the *Solano County General Plan* (Solano County, 2008) reveals that the site does not lie in an area of known subsidence. In our opinion, settlement at the site due to subsidence will not adversely affect the site provided the recommendations of this report are followed.

Naturally Occurring Asbestos (NOA)

The site is directly underlain by Quaternary-aged alluvial fan deposits (Qhf). 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.

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 Solano County as an area containing common indicators of naturally occurring radon gas.

According to the Environmental Protection Agency's Map of Radon Zones, the project site is located within Zone 3, meaning the site has a predicted average indoor screening level less than two picocuries per liter. 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.

Bearing Capacity

Based upon our field and laboratory testing, it is our opinion the undisturbed native soils, as well as engineered fill composed of native or approved import material, are capable of supporting the planned improvements, provided the recommendations in this report are followed.

The near-surface clay soil is considered capable of exerting significant expansion pressures on building foundations, floor slabs, exterior flatwork, and pavements. These soils will require sub-excavation, processing and recompaction for support of foundations, interior floor slabs, exterior flatwork and pavements. Sub-excavation, scarification and recompaction of near-surface soils in accordance with the recommendations of this report will be capable of supporting the proposed foundations, interior floor slabs, exterior flatwork and pavements.

Soil Expansion Potential

Laboratory testing of soils collected from the upper one to four feet at Boring D1 revealed the near-surface clay soils possess a “high” expansion potential when tested in accordance with the ASTM D4829 (see Figure A3). This is consistent with test results for near-surface clays collected for other studies located in the vicinity of the site. Based on these test results, the near-surface soils in the area of the proposed science building are considered capable of exerting significant expansion pressures on foundations, interior floor slabs, exterior flatwork and pavements. Therefore, at least 12 inches of imported, compactable, non-expansive soils will be required beneath concrete slabs-on-grade, including sidewalks. Chemical amendment of the clay soils (i.e., lime-treatment) also could be considered to reduce the expansion potential of the on-site clays.

Pavement Subgrade Quality

A representative bulk sample collected from the upper one to four feet at the site was subjected to Resistance (“R”) value testing in accordance with California Test 301. Laboratory testing of the sample revealed the near-surface materials possess an R-value of 5 (see Figure A4). Based on these results, the near-surface soils at the site are considered poor quality materials for support of asphalt concrete pavements. It is our opinion an R-value of 5 is appropriate for design of pavements at the site supported on untreated subgrades.

Based on our experience with similar soil conditions in the vicinity of the site, we anticipate lime-treatment of the clay soils can improve its support quality and reduce the required base material thickness for pavement sections. Recommendations regarding lime-treatment of the pavement subgrade soils are provided in the Pavement Design section of this report.

Effect of Previous Development on Planned Construction

The site currently is developed with landscaping, light poles and several underground utilities associated with previous development of the site. From a geotechnical standpoint, the most effective method of mitigating the impact of existing landscaping, light poles and underground utilities on the new construction is to completely remove all existing landscaping and 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 sub-surface structure removal in the Site Clearing section of this report.

There are existing buildings in relatively close proximity to the area of the proposed science building. It is our opinion that excavations associated with the proposed construction should not affect the existing buildings, provided the new excavations do not encroach within a 1 horizontal to 1 vertical (1H:1V) projection from the bottom of the existing structure foundations.

Excavation Conditions

The surface and near-surface soils at the site should be readily excavatable with conventional earthmoving and trenching equipment. Based on our borings, excavations associated with building foundations, shallow trenches for utilities, and other excavations less than five feet deep associated with the proposed construction, should stand vertically for short periods 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, if necessary.

Excavations or trenches exceeding five feet in depth that will be entered by workers should be sloped, braced or shored to conform to current Occupational Safety and Health Administration (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 should be constructed no steeper than a one horizontal to one vertical (1H:1V) inclination. Temporary slopes likely will stand at this inclination for the short-term duration of construction, provided significant pockets of loose and/or saturated granular soils are not encountered. Flatter slopes would be required if these conditions are encountered.

Excavated materials should not be stockpiled directly adjacent to an open excavation to prevent surcharge loading of the excavation sidewalls. Excessive 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.

Groundwater Effect on Development

Groundwater was observed in Borings D1 and D3 at a depth of approximately 11 feet below existing site grades on July 30, 2014. Review of previous explorations performed at the subject campus and other available groundwater data revealed the historical high groundwater elevation at the site is likely about six feet below the existing ground surface (approximate elevation of +34 feet msl). Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation, local pumping, and other factors.

Based on current and previous explorations performed at the subject campus and historical groundwater data, we anticipate excavations greater than six feet below existing site grades may encounter groundwater and require dewatering (depending on the time of year). For design purposes, groundwater should be anticipated at an elevation of +34 feet msl. If groundwater is encountered, the use of sumps, submersible pumps, deep wells or a well point system could be used as methods to lower the groundwater level. The dewatering method used will depend on the soil conditions, depth of the excavation and amount of groundwater present within the excavation. Dewatering, if required, should be the contractor’s responsibility. The dewatering system should be designed and constructed by a dewatering contractor with local experience. We recommend the selected dewatering system lower the groundwater level to at least two feet below the bottom of the proposed excavations.

Seasonal Water

During the wet season, infiltrating surface runoff water will create a saturated surface condition due to the relatively low permeability of 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. Also, soils exposed beneath existing concrete slabs designated for removal (if any) may be at elevated moisture contents regardless of the time year constructed. Such soil, intended for use as engineered fill, will require a prolonged period of dry weather and/or considerable aeration to reach a moisture content suitable to achieve required compaction. This should be considered in the construction schedule for the project.

On-site Soil Suitability for Use in Fill Construction

The existing 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, and are at a proper moisture content to achieve the desired degree of compaction. Organically laden topsoil should not be reused as engineered fill.

The clay soils present beneath the site are not suitable for direct support of interior or exterior slab-on-grade concrete. Specific recommendations for subgrade preparation have been presented in this report to mitigate the effect of expansive clay on the planned structure and slabs.

Soils beneath existing exterior flatwork designated for removal (if any) will likely be at an elevated moisture content regardless of the time of year of construction and may require drying before compaction or use as fill.

Existing concrete flatwork within areas to be demolished (if any) may be broken up for use as fill, provided the Portland cement concrete (PCC) is processed into fragments less than three inches in largest dimension, is mixed with soil to form a compactable mixture, and is approved by the college district.

Soil Corrosion Potential

One sample of near-surface soil 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 A5 and A6.

Table 4: Corrosion Test Results

Analyte	Test Method	D1 (0’ – 4’)
pH	CA DOT 643 Modified*	6.62
Minimum Resistivity	CA DOT 643 Modified*	1,310 Ω-cm
Chloride	CA DOT 422	37.5 ppm
Sulfate	CA DOT 417	76.9 ppm
	ASTM D516	89.1 ppm

Notes:
** = Small cell method* *CA DOT = California Department of Transportation*
Ω-cm = Ohm-centimeters *ppm = Parts per million*

The California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, Corrosion Guidelines (Version 2.0, dated November of 2012), considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples 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 soil is not considered corrosive to steel reinforcement properly embedded within Portland cement concrete (PCC) for the sample tested.

Table 4.2.1 – *Exposure Categories and Classes*, American Concrete Institute (ACI) 318-11, Section 4.2, as referenced in Section 1904.1 of the 2013 CBC, indicates the severity of sulfate exposure for the sample tested is *Not Applicable*. Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover as detailed in ACI 318-11, Section 7.7 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 site 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 or chemical treatment. Should the construction schedule require work to continue during the wet months, additional recommendations can be provided, as conditions dictate.
Site preparation should be accomplished in accordance with the provisions of this report and the appended specifications. A representative of the Geotechnical Engineer should be present during all earthwork operations to evaluate compliance with the 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 construction areas should be cleared of all existing surface and sub-surface structures associated with previous site development to expose firm and stable soils. The area of removal should include the entire building pad and an area extending at least five feet beyond all exterior foundations, where practical. Demolition debris should be removed from the site, or used as engineered fill, provided it is processed per the recommendations in this section. Existing underground utilities to be removed or relocated should include removal of all trench backfill and be replaced with engineered fill.

Existing slabs-on-grade designated for removal (if any) may be broken up, pulverized and reused as engineered fill, or removed from the site. If concrete rubble is to be reused as engineered fill, it should be pulverized to fragments less than three inches in largest dimension and contain sufficient intermediate sized particles to form a compactable mixture, and must be approved by the geotechnical engineer and owner.

Any trees and other vegetation 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.

Existing surface vegetation and organically laden soil within construction areas should be removed by stripping. Stripping should not be used in general fill construction areas supporting structures, interior/exterior concrete slabs, or pavements. Debris from the stripping should not be used in general fill construction areas supporting the proposed building or concrete slabs. With prior approval the Geotechnical Engineer, strippings may be used in landscape areas, provided they are kept at least five feet from the building pad and other surface improvements, moisture conditioned, and compacted.

Depressions resulting from site clearing operations, as well as any loose, 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 in accordance with the recommendations of this report. We consider it essential that the Geotechnical Engineer's representative be present during site clearing activities to verify the adequate removal of surface and subsurface structures and observe and evaluate the condition of the existing on-site materials.

<u>Subgrade Preparation</u>

The near-surface clay soils are considered capable of exerting significant expansion pressures on planned improvements. Following site clearing activities, surface clay soils should be sub-excavated from all structural areas of the site (i.e. building pad, flatwork, etc.) to a depth of at least 12 inches below final soil subgrade elevation. The final subgrade elevation is defined as the surface on which capillary break gravel or aggregate base are placed for support of slab concrete. The required excavation should extend at least five feet beyond the planned building footprint and two feet beyond concrete slabs that will be used for vehicle access.

Any debris exposed by the required sub-excavation should be removed. The soils exposed following the recommended sub-excavation, 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 two percent above the optimum moisture content, and 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. Please note this sub-excavation recommendation is not necessary within asphalt concrete pavement areas (i.e. areas to support vehicle traffic).

The on-site surface clay soils encountered at the site are anticipated to react well with the addition of quicklime (high-calcium or dolomitic). As an alternative to the sub-excavation recommendations provided above, lime-treating the surface clay soils within structural areas of the site could mitigate the effect of expansion pressures produced by the untreated clay soils on the planned improvements. If lime-treatment of the clay soils is selected, we recommend the final subgrade elevation within structural areas is mixed with lime at a minimum spread rate of at least 4½ pounds of quicklime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer 12 inches thick. Lime-treatment of the subgrade soils should be performed in general conformance with Section 24 of the *Caltrans Standard Specifications*, latest edition. Lime-treated soil for support of concrete foundation slabs or exterior flatwork should be moisture conditioned to at least two percent above the optimum moisture content and compacted to not less than 90 percent of the ASTM D1557 maximum dry density, and maintained in that condition until covered by capillary break gravel or aggregate base.

Subgrade preparation operations should extend at least five feet beyond the building pad, including adjacent flatwork, where practical. Compaction should be performed using a heavy, self-propelled, sheepsfoot compactor capable of achieving the required compaction. Difficulty in achieving subgrade compaction may be an indication of loose, soft or unstable soil conditions associated with prior site development. If these conditions exist, the loose, soft or unstable materials should be excavated to expose firm and stable soils. The resulting excavations should be backfilled with engineered fill as described in the Engineered Fill Construction section of this report.

We recommend construction bid documents contain a unit price (per cubic yard) for additional excavation and replacement with engineered fill.

Engineered Fill Construction

Engineered fill consisting of native or import materials should be placed in lifts not exceeding six inches in compacted thickness, with each lift being thoroughly moisture conditioned to at least two percent above the optimum moisture content for clay soils and to the optimum moisture content for granular soils (import fill materials), maintained in that condition, and uniformly compacted to at least 90 percent relative compaction.

On-site soils encountered in our explorations are considered suitable for use as engineered fill, provided they are at a workable moisture content to achieve required compaction, and do not contain rubbish, rubble, deleterious debris, and organics. However, clay soils should not be used in fills within the upper 12 inches of final subgrade for the building pad or exterior flatwork, unless the clay soils are lime-treated as described in the Subgrade Preparation section of this report. Building pad and exterior flatwork final subgrade is the surface in which aggregate base or capillary break materials are placed.

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. In addition, we recommend that the contractor provide appropriate documentation that the imported fill materials do not contain known contaminants and have corrosion characteristics within acceptable limits. Imported soils should be approved by the Geotechnical Engineer prior to being transported to the site.

The upper 12 inches of final subgrade for the concrete foundation slabs and exterior flatwork should consist of imported compactable, non-expansive (Expansion Index < 20) granular soils, or, 12 inches of lime-treated soils as described in the Subgrade Preparation section of this report. All soils supporting interior or exterior slab-on-grade concrete should be uniformly compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557.

The upper six inches of untreated pavement subgrades should be uniformly compacted to at least 95 percent of the maximum dry density at a moisture content of at least two percent above the optimum moisture content, regardless of whether final grade is established by excavation, engineered fill or left at grade. Alternatively, the upper 12 inches of lime-treated subgrade soils should be compacted to at least 95 percent relative compaction at not less than two percent over the optimum moisture content. Regardless, final pavement subgrades must be stable under construction traffic prior to placement of aggregate base.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2H:1V).

All earthwork operations should be accomplished in accordance with the recommendations contained within this report and the *Guide Earthwork Specifications* provided in Appendix D. We recommend the Geotechnical Engineer's representative be present on a regular basis during all 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.

Utility Trench Backfill

Utility trench backfill within structural areas (e.g. building, exterior flatwork, pavements, etc.) 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 be provided in accordance with the manufacturer's recommendations for the pipe materials selected and applicable sections of the governing agency standards.

We recommend that native soil be used as trench backfill, especially below the non-expansive or lime-treated material within the footprint of the proposed building. Utility trench backfill should be placed in maximum eight-inch lifts (compacted thickness), thoroughly moisture conditioned to at least two percent above the optimum moisture content, and mechanically compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. Within the upper 12 inches of final subgrade for the building pad and exterior flatwork, trench backfill should consist of granular material as described in the Engineered Fill Construction section of this report, unless the lime-treatment alternative presented in the Subgrade Preparation section of this report is selected. If the top 12 inches of the improvement areas consist of lime-treated soils, the upper 12 inches of trench backfill should consist of controlled density fill (CDF) or aggregate base compacted to at least 95 percent relative compaction.

We recommend that all underground utility trenches aligned nearly parallel with existing or new foundations be at least three feet from the outer edge of 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 bottom of foundations. Additionally, trenches parallel to existing 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.

Foundations

As noted previously, seismically induced settlements of about 3¾ inches of total settlement and two inches of differential settlement across 50 feet (or the shortest dimension of the structure, whichever is less) should be anticipated. The foundation system to support the proposed science building should be designed to accommodate the calculated settlements.

The proposed science building may be supported upon a continuous perimeter foundation with continuous and/or isolated interior spread foundations embedded at least 18 inches below lowest adjacent soil grade, provided the subgrade 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 bearing in undisturbed or recompacted native soils, engineered fill, or a combination of those materials may be sized for maximum allowable "net" soil bearing pressures of 2500 pounds per square foot (psf) for dead plus live loads, with a 1/3 increase for total loads including 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.

We recommend that all foundations be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The structural engineer should determine final foundation reinforcing requirements.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.30, 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 300 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. We recommend that all foundation excavations be observed by the Geotechnical Engineer's representative prior to placement of reinforcement and concrete to verify firm bearing materials are exposed.

Interior Floor Slab Support

Interior concrete slab-on-grade floors for the science building can be supported upon the non-expansive imported soil or lime-treated soil subgrade prepared in accordance with the recommendations in this report, provided the subgrade soils are maintained in that condition (at least the optimum moisture content) and protected from disturbance. If this is not the case and the subgrade soils become dry, disturbed and/or desiccated, the building pad will require additional scarification, moisture conditioning and compaction prior to construction of the interior floor slabs.

The interior concrete slab-on-grade floor for the building should be at least five inches thick. We recommend that interior floor slabs be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The project structural engineer should determine final floor slab reinforcing requirements. 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 slab-on-grade floor.

Floor slabs that will receive moisture sensitive floor covering (e.g. vinyl covering, wood-laminate, etc.) should be underlain by a layer of free-draining gravel/crushed rock, serving as a deterrent to migration of capillary moisture. The gravel/crushed rock layer should be at least four 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 plastic, water vapor retarder (at least 10-mils thick) directly over the gravel/crushed rock. The water vapor retarder should meet or exceed the minimum specifications for plastic water vapor retarders 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 over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern for water trapped within the sand. As a consequence, we consider the use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

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

Floor Slab Moisture Penetration Resistance

It 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 structures. For this reason, it should be assumed that interior slabs that are intended for moisture-sensitive floor coverings or materials, require protection against moisture or moisture vapor penetration. Standard practice includes placing a layer of gravel/crushed rock and a vapor retarder membrane (and possibly a layer of sand) as discussed above. Recommendations contained in this report concerning foundation and floor slab design are presented as minimum requirements only from the geotechnical engineering standpoint.

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

Exterior Flatwork Construction

The upper 12 inches of final subgrade for exterior concrete flatwork should consist of imported compactable, non-expansive granular soils placed and compacted in accordance with the Engineered Fill Construction recommendations included in this report. Exterior flatwork subgrade soils should be maintained in a moist condition (at least the optimum moisture content) and protected from disturbance. If this is not the case and the subgrade soils become dry and disturbed, the exterior flatwork subgrade will require additional scarification, moisture conditioning and compaction prior to construction of the exterior flatwork.

Exterior flatwork concrete should be at least four inches thick. Consideration should be given to thickening the slabs to 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 perimeter building foundations by the placement of a layer of felt material between the flatwork and the foundation. 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.

Areas adjacent to new exterior flatwork should be landscaped to maintain more uniform soil moisture conditions adjacent to and under flatwork. We recommend final landscaping plans not allow fallow ground adjacent to exterior concrete flatwork.

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.

Retaining Walls

Retaining walls that will be allowed to slightly rotate about their base (unrestrained at the top or sides) should be capable of resisting "active" lateral earth pressure equal to an equivalent fluid pressure of 40 psf per foot of wall backfill for horizontal backfill and fully drained conditions. Retaining walls that are fixed at the top should be capable of resisting "at-rest" lateral earth pressure equal to an equivalent fluid pressure of 60 psf per foot of wall backfill, again assuming horizontal backfill and fully drained conditions. Walls supporting sloping backfill, up to a 2H:1V inclination, should be designed adding an additional 20 psf per foot of wall to the pressures presented above.

Retaining walls may be supported on a continuous foundation extending at least 18 inches below lowest adjacent soil grade. Continuous footings for retaining walls may be designed based upon the recommendations contained in the Foundation Design section of this report. Appropriate set-backs for structures constructed behind the walls should be maintained so that such structures do not surcharge the walls. To utilize the full allowable passive resistance, the minimum horizontal distance from the base of the wall footing to the face of a graded slope in front of the wall should be at least five feet.

Backfill behind retaining walls should be fully drained to prevent the build-up of hydrostatic pressures behind the wall. Retaining walls should be provided with a drainage blanket (Class 2 permeable material (Caltrans Specification Section 68-1.025) 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 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.

Structural backfill materials for retaining walls (other than the drainage layer) should consist of non-expansive (Expansion Index < 20), compactable granular material that does not contain significant quantities of rubbish, rubble, organics and rock over six inches in size. Clays should not be used for wall backfill. Structural backfill should be placed in lifts not exceeding 12 inches in compacted thickness, and should be mechanically compacted to at least 90 percent relative compaction. The lateral pressures recommended above assume that clay soils, if exposed during site excavations, will not be used as backfill behind retaining walls.

Pavement Design

Specific pavement areas are not shown on the available project drawings; however, we understand pavement design recommendations have been requested. Laboratory test results from near-surface clay soils encountered at the site exhibit poor support qualities for support of asphalt concrete pavements. Relatively thick pavement sections would be required for pavements unless the clays are lime-treated. The following pavement sections, presented as Table 5, have been calculated based on the untreated R-value test results, an assumed R-value for lime-treated subgrade soils, assumed traffic indices (TI), and the procedures contained within Chapters 600 to 670 of the California Highway Design Manual, dated May of 2012. The project civil engineer should determine the appropriate traffic index based on anticipated traffic conditions. If needed, we can provide additional pavement sections for different traffic indicies. An R-value of 5 was used for untreated native clay subgrades, and an R-value of 40 was assumed for preliminary design of clay subgrades amended with at least four percent high calcium or dolomitic quicklime.

Table 5: Pavement Design Alternatives for Asphalt Concrete Pavements

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 5		Lime-Treated Subgrades Soils (a) R-value = 40	
		Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	Automobile Parking	2½*	10	2½*	4
		3*	9	3*	4
6.5	Automobile and Light to Moderate Truck Traffic and Fire Lanes	3	16	3	8
		4*	14	4*	6

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

(a) = Lime-treated subgrade should be at least 12 inches thick and possess a minimum R-value of 40 when tested in accordance with California Test 301 (Caltrans, 2000).

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 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 untreated pavement subgrade soils and upper 12 inches of lime-treated subgrade soils should be compacted to at least 95 percent relative compaction at no less than two percent above the optimum moisture content, maintained in that condition (at least the optimum moisture content) and protected from disturbance. If this is not the case and the subgrade soils become dry, disturbed and/or desiccated, the pavement subgrade will require additional scarification, moisture conditioning and compaction prior to construction of the exterior flatwork. All aggregate base should be compacted to at least 95 percent 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 our 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 concrete cement (PCC) pavements in areas subjected to concentrated heavy wheel loading, such as entry driveways and in front of trash enclosures. The PCC pavements should be at least six inches thick, supported on at least eight inches of compacted Class 2 aggregate base over untreated subgrade (R-value = 5); or, six inches of PCC pavement, supported on at least four inches of compacted Class 2 aggregate base over lime-treated subgrade (R-value = 40). All aggregate base should be compacted to at least 95 percent relative compaction.

We recommend the concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards, 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.

Lime-treatment of Pavement Subgrade Soils

The native clay soils are anticipated to react well with the addition of quicklime (high-calcium or dolomitic) and could enhance the support characteristics of the subgrade and allow for a reduction in the aggregate base section. If lime-treatment of subgrade soils is selected, the lime-treatment of subgrade soils should be performed in general conformance with Section 24 of the *Caltrans Standard Specifications*. Additional testing should be performed during construction to verify that the design parameters are achieved in the field. Samples of the field-mixed soil and lime should be collected and tested for a minimum R-value of 40, when tested in accordance with California Test 301 (Caltrans, 2000). This additional testing will either verify the design parameters, or provide the opportunity to modify the pavement sections or spread rate based upon the test results.

<u>For estimating purposes only</u>, we recommend a minimum spread rate of at least 4½ pounds of quicklime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer 12 inches thick. Lime-treated subgrades should be compacted to not less than 95 percent of the ASTM D 1557 maximum dry density at a moisture content of at least two percent (2%) above the optimum moisture content.

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.

<u>Site Drainage</u>

Final site grading should be accomplished to provide positive drainage of surface water away from the science building and prevent ponding of water adjacent to foundations, slabs or pavements. The grades adjacent to the building should be sloped away from foundations at a minimum two percent gradient for at least 10 feet, where possible. We recommend connecting all roof drains to solid drainage pipes which are connected to available drainage features that convey water away from the building, or discharging the downspouts onto concrete or asphalt surfaces that slope away from the foundations. Discharging or ponding of surface water should not be allowed adjacent to the building, exterior flatwork or pavements. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward the building.

Geotechnical Engineering Observation and Testing During Earthwork Construction

Site preparation should be accomplished in accordance with the recommendations of this report and the *Guide Earthwork Specifications* provided in Appendix D. 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 at the project to verify compliance with this geotechnical 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; however, we would be pleased to submit a proposal to provide these services upon request.

Section 1803A.5.8 “Compacted Fill Material” of the 2013 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 effect 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 difficulties 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 should indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary (Form DSA-109). A final report by the “Geotechnical Engineer” should be prepared upon completion of the project.

Additional Services

Our scope of services includes review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents and two site visits during construction. If additional services are required from our firm, we would be pleased to submit a proposal to provide theses 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 prudent engineering and geologic judgment based upon the information provided and the data generated from our study. 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 relocated 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. This report should not be utilized for construction on any other site. This report is considered valid for the proposed construction for a period of two years following the date of this report. If construction has not started within two years, we must re-evaluate the recommendations of this report and update the report, if necessary.

Wallace - Kuhl & Associates

REGISTERED PROFESSIONAL ENGINEER
MAURICIO LUNA
No. 81151
Exp. 9/30/15
CIVIL
STATE OF CALIFORNIA

Mauricio Luna
Project Engineer

Joe Rosenbery, MS
Staff Geologist

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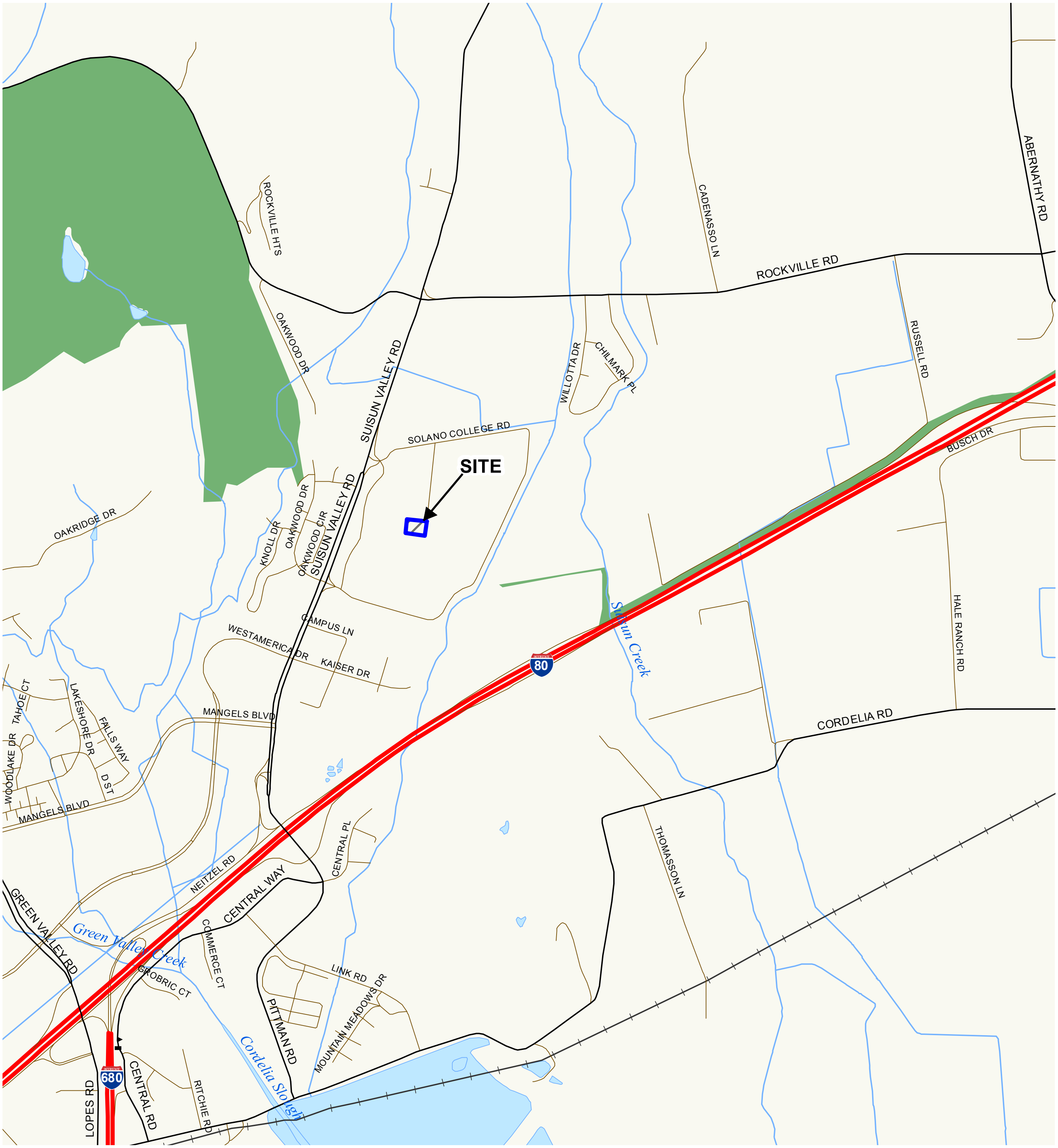
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
DAVID R. GIUS, JR.
No. #1681
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

David R. Gius, Jr.
Senior Engineer/Senior Engineering Geologist



Street data courtesy of ESRI, 2010.
Hydrography courtesy of the U.S. Geological Survey acquired from the GIS Data Depot, December, 2007.
Projection: NAD 83, California State Plane, Zone II

N

0 1,000 2,000
Feet

WallaceKuhl & ASSOCIATES

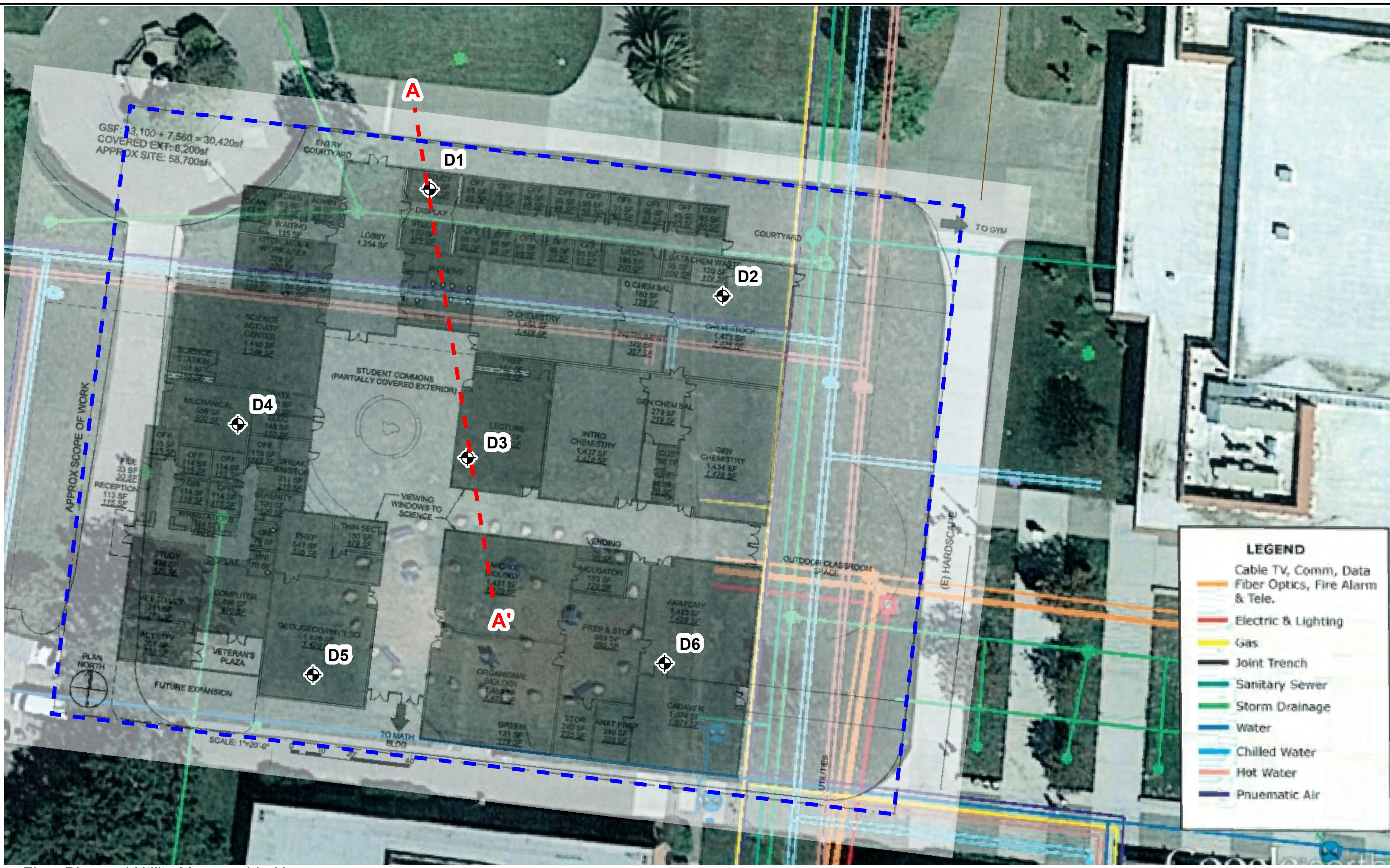
VICINITY MAP

SOLANO COMMUNITY COLLEGE NEW SCIENCE BUILDING

Fairfield, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15

WKA NO. 10642.01P



Preliminary Floor Plan and Utility Map provided by Kitchell, dated July 8, 2015.
Projection: NAD 83, California State Plane, Zone III

Legend

- Site Boundary
- Approximate Boring Location
- Cross Section

N

0 25 50
Feet

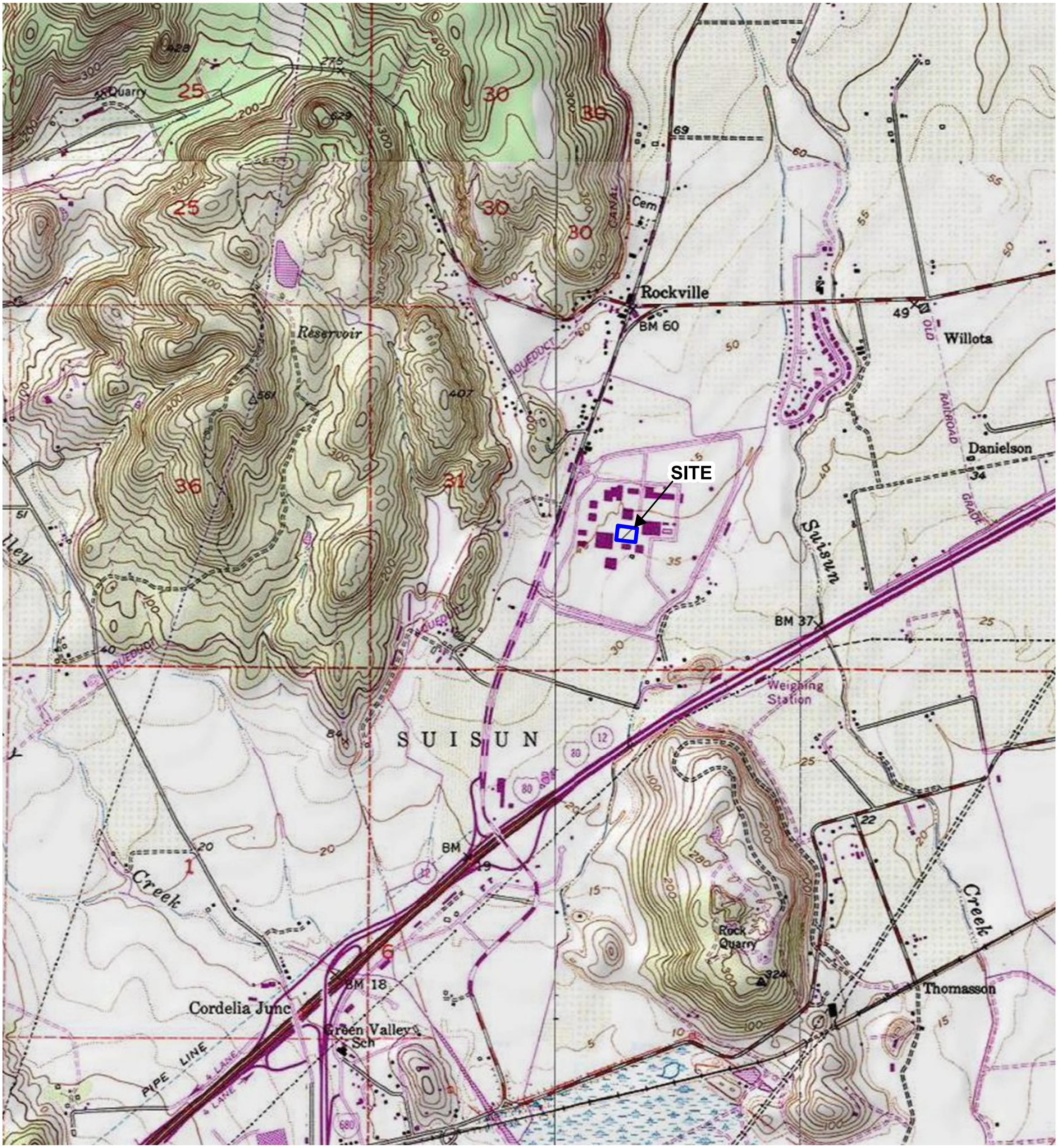
WallaceKuhl & ASSOCIATES

SITE PLAN

SOLANO COMMUNITY COLLEGE NEW SCIENCE BUILDING

Fairfield, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO.	10642.01P



Adapted from U.S. Geological Survey 7.5 minute topographic maps of the Fairfield South and Cordelia quadrangles, California, 1980, Fairfield North quadrangle, California, 1981 and Mount George quadrangle, California, 1969.
Projection: NAD 83, California State Plane, Zone II

N

0 1,000 2,000
Feet

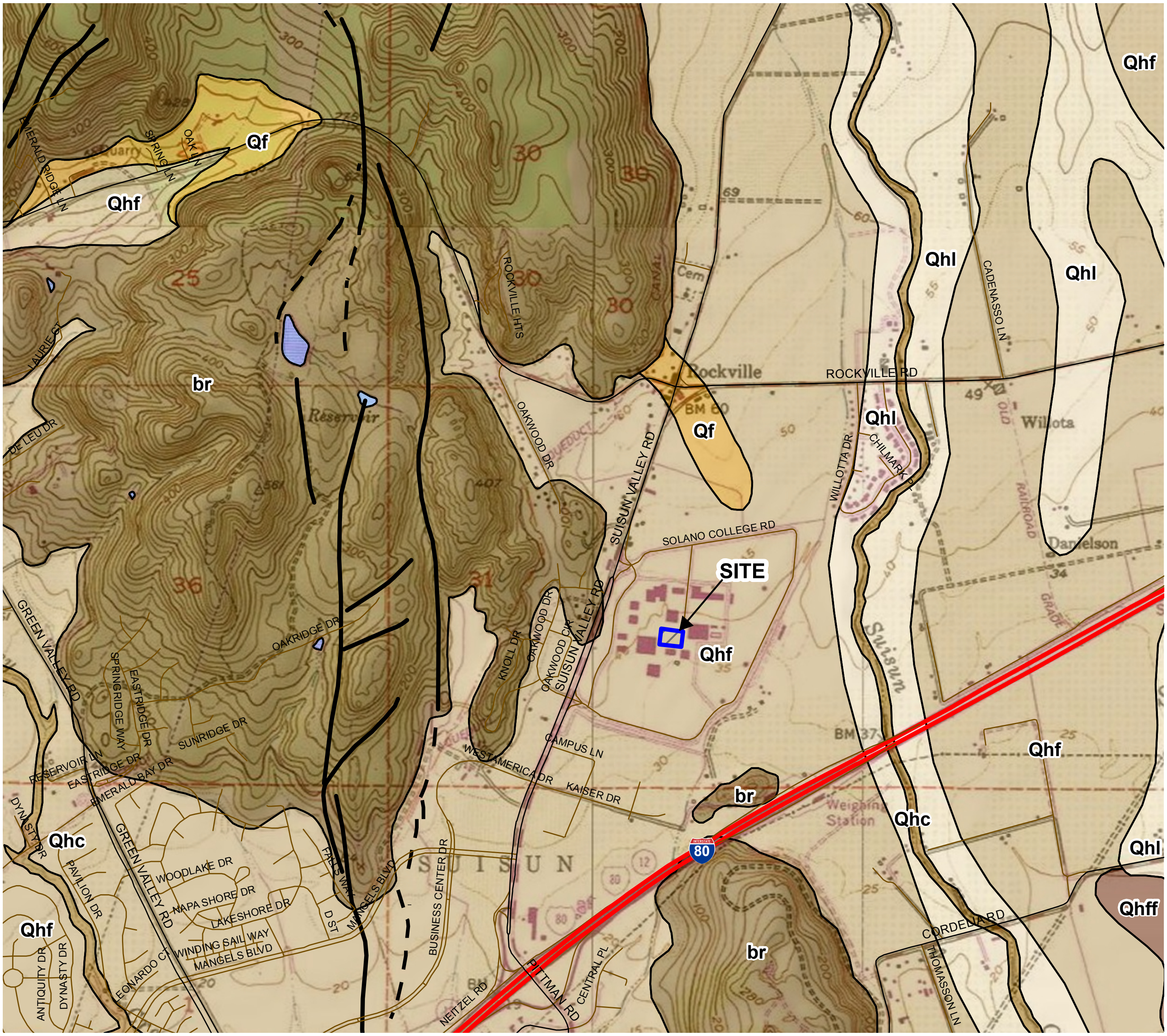
WallaceKuhl
& ASSOCIATES

USGS TOPOGRAPHIC MAP

SOLANO COMMUNITY COLLEGE NEW SCIENCE BUILDING

Fairfield, California

FIGURE	3
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO. 10642.01P	



Adapted from Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, U.S. Geological Survey Open-File Report 06-1037 (2006). Fault data from USGS Quaternary Fault and Fold Database of the United States, dated November 3, 2010.
Projection: NAD 83, California State Plane, Zone II

N

0 1,000 2,000
Feet

Legend

Approximate fault location

af - Artificial fill

Qhc - Modern stream channel deposits

Qhf - Holocene alluvial fan deposits

Qhff - Holocene alluvial fan deposits, fine grain facies

Qhl - Holocene fan levee deposits

Qf - Late Pleistocene to Holocene alluvial fan deposits

br - Early Quaternaryand older (>1.4 Ma) deposits and bedrock

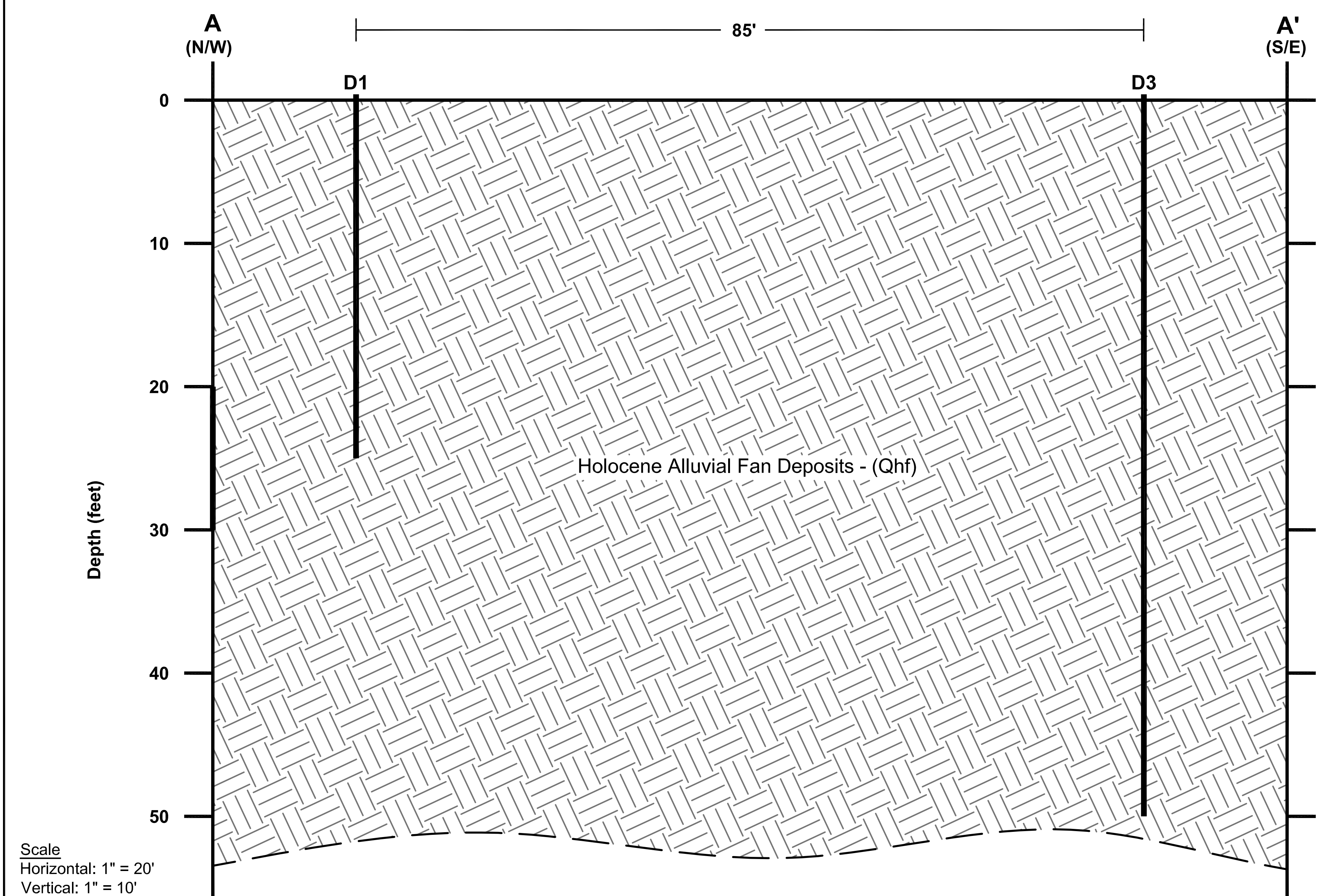
WallaceKuhl & ASSOCIATES

GEOLOGIC MAP

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	4
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO. 10642.01P	



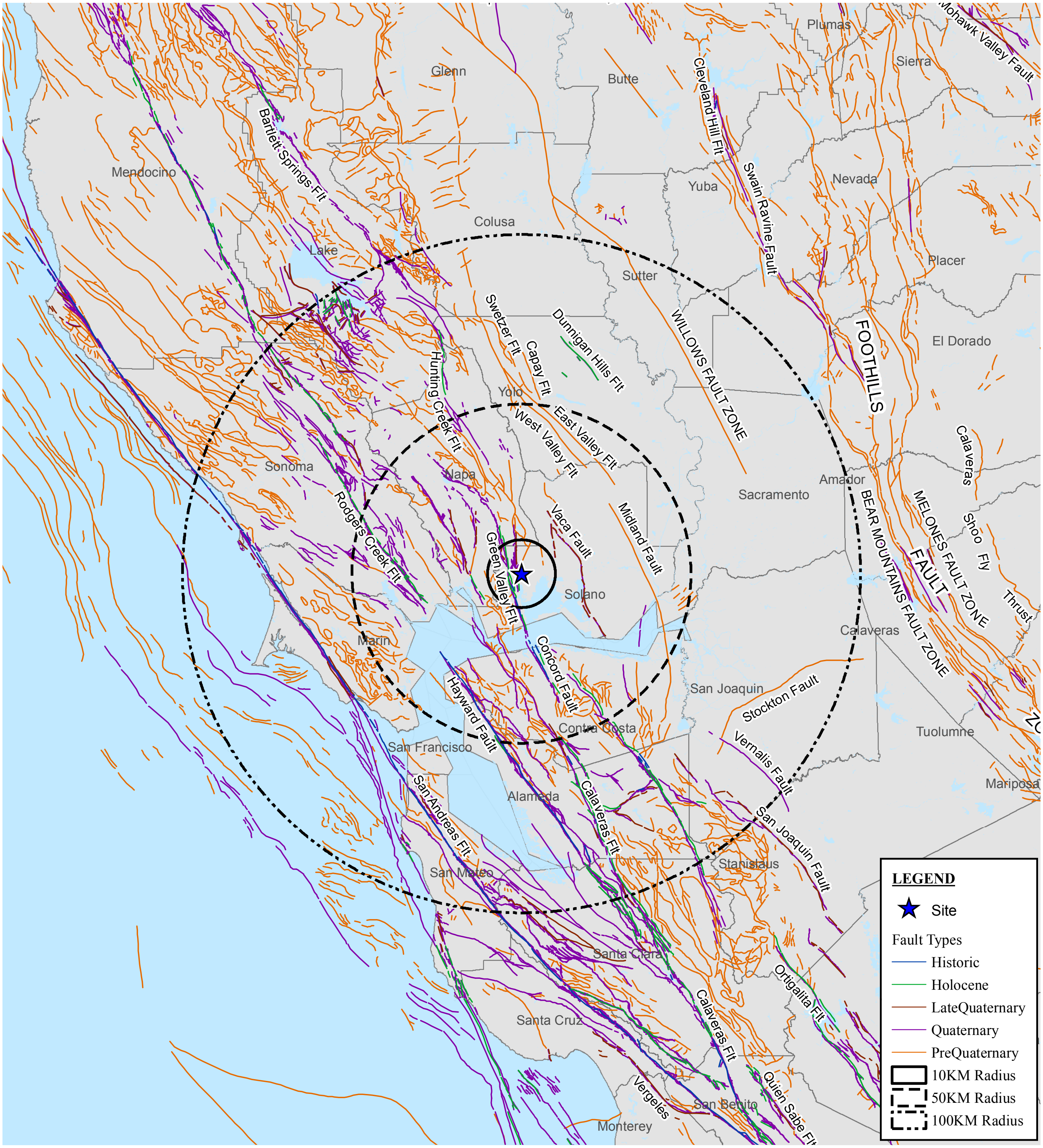
WallaceKuhl & ASSOCIATES

GEOLOGIC CROSS SECTION

SOLANO COMMUNITY CLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	5
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO. 10642.01P	



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

N

0 40
Kilometers

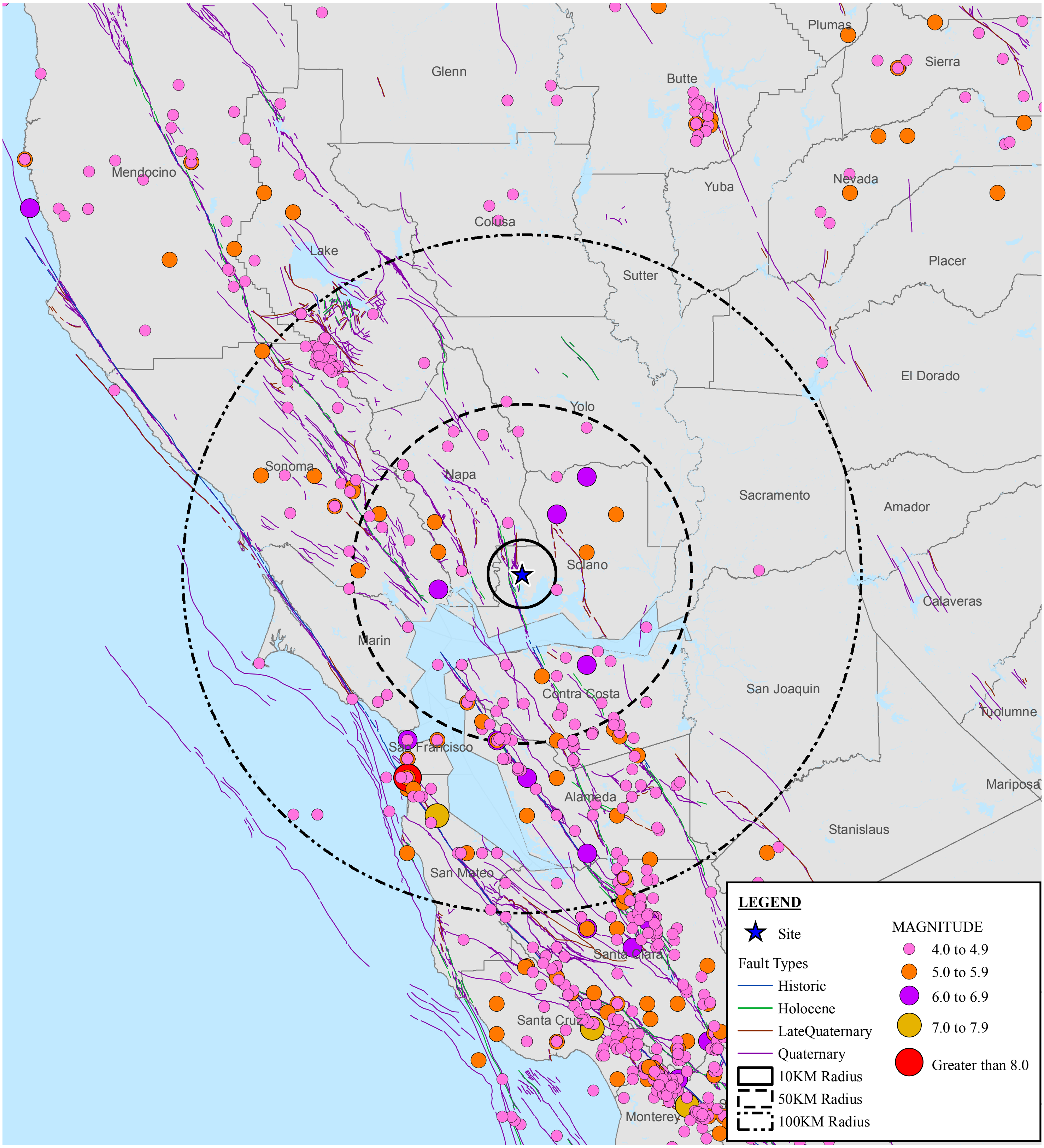
WallaceKuhl & ASSOCIATES

FAULT MAP

SOLANO COMMUNITY COLLEGE NEW SCIENCE BUILDING

Fairfield, California

FIGURE	6
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO.	9469.01P



Epicenters from EQ Search 12/2010.
Projection: NAD 83, California State Plane, Zone II

N

0 40
Kilometers

WallaceKuhl & ASSOCIATES

EPICENTER MAP

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	7
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO.	10642.01P

Project: Solano Coummunity College - New Sicence Building Project Location: Fairfield, California WKA Number: 10642.01P	**LOG OF SOIL BORING D1** Sheet 1 of 1

Date(s) Drilled	**7/30/15**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**V & W Drilling, Inc.**	Total Depth of Drill Hole	**24.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**11.0**	Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk sample D1 (0' - 4')**			Driving Method and Drop	**140 lb hammer, 30 inch 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
			Dark brown, dry to moist, stiff, silty CLAY (CL).						
					D1-1I	15	18.9	95	
	5		moist		D1-2I	17	20.3	90	UCC 3.0tsf
	10		brown, very stiff		D1-3I	21	23.0	101	
			Brown, moist, stiff, sandy CLAY (CL); fine sand.						
	15				D1-4I	11	24.8	99	
	20		wet		D1-5I	11	28.5	92	
			hard		D1-6I	67/9"			

Boring terminated at 24.5' below existing site grades.
Groundwater observed at 11' below existing site grades.

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FIGURE 8

Project: Solano Coummunity College - New Sicence Building **Project Location: Fairfield, California** **WKA Number: 10642.01P**	**LOG OF SOIL BORING D2** Sheet 1 of 1

Date(s) Drilled	**7/30/15**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Stem 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		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**140 lb hammer, 30 inch 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
			Brown to gray brown, dry, very stiff, clayey SILT (ML-Fill).		D2-1I	30	7.9	95	
			Dark brown and red brown, moist, very stiff, silty CLAY (CL-Fill).		D2-2I	23	16.7	103	
	5		Dark brown, moist, stiff to very stiff, silty CLAY (CL).						
	10				D2-3I	20	23.5	103	

Boring terminated at 10' below existing site grades.
Groundwater was not observed.

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FIGURE 9

Project: Solano Coummunity College - New Sicence Building
Project Location: Fairfield, California
WKA Number: 10642.01P

LOG OF SOIL BORING D3

Date(s) Drilled	**7/30/15**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Rotary Wash**	Drilling Contractor	**V & W Drilling, Inc.**	Total Depth of Drill Hole	**51.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**4"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**11.0**	Sampling Method(s)	**Standard Penetration Test**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk sample D3 (0' - 4')**			Driving Method and Drop	**140 lb hammer, 30 inch 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
			SAMPLE DATA				**TEST DATA**		
	0		Brown, dry, hard, silty CLAY (CL).		D3-1I	35	9.3		
	5		moist, very stiff		D3-2I	21	17.6		
	10		stiff		D3-3I	14	21.6		
	15		Brown, moist to wet, stiff, sandy CLAY (CL); fine sand.		D3-4I	10	27.1		73% fines
	18		Brown, wet, medium dense, silty SAND (SM); fine to medium sand.		D3-5I	11	30.9		20% fines
	23		Brown, wet, medium dense, poorly-graded SAND with silt and gravel (SP-SM); fine to coarse sand; fine gravel.		D3-6I	19	20.5		10% fines
	30		Brown, wet, very stiff, silty CLAY (CL).		D3-7I	29	21.5		PI 76% fines
	35		Brown, wet, very stiff, sandy CLAY (CL); fine sand.		D3-8I	23	25.8		PI 61% fines
	38		Brown, wet, hard, silty CLAY (CL).		D3-9I	40	23.2		
	45		very stiff		D3-10I	23	27.1		PI 79% fines
	50		blue gray		D3-11I	18	35.5		

Boring terminated at 51.5' below existing site grades.
Groundwater observed at 11' below existing site grades.

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FIGURE 10

Project: Solano Coummunity College - New Sicence Building
Project Location: Fairfield, California
WKA Number: 10642.01P

LOG OF SOIL BORING D4

Sheet 1 of 1

Date(s) Drilled	**7/30/15**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Stem 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		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil Cuttings**
Remarks	**Bulk sample D4 (0' - 4')**			Driving Method and Drop	**140 lb hammer, 30 inch drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, dry, very stiff, silty CLAY (CL).						
					D4-1I	27	11.9	91	
	5		moist, stiff		D4-2I	18	20.3	98	
	10		brown		D4-3I	16	21.6	105	
			Boring terminated at 10' below existing site grades. Groundwater was not observed.						

BORING LOG 10642.01P - SOLANO CC NEW SCIENCE BLDG.GPJ WKA.GDT 8/20/15 3:15 PM

Wallace Kuhl & ASSOCIATES

FIGURE 11

Project: Solano Coummunity College - New Sicence Building Project Location: Fairfield, California WKA Number: 10642.01P	**LOG OF SOIL BORING D5** Sheet 1 of 1

Date(s) Drilled	**7/30/15**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Stem 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		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**140 lb hammer, 30 inch 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
			Light brown, moist, medium dense, silty SAND (SM-Fill); fine to medium sand.						
			Dark brown, moist, stiff, silty CLAY (CL).		D5-1I	27	17.1	109	UCC 5.3tsf
	5		brown		D5-2I	18	18.7	105	UCC 2.7tsf
			Brown, moist, stiff to very stiff, sandy CLAY (CL; fine sand.						
	10				D5-3I	20	21.8	101	
			Boring terminated at 11.5' below existing site grades. Groundwater was not observed.						

BORING LOG 10642.01P - SOLANO CC NEW SCIENCE BLDG.GPJ WKA.GDT 8/20/15 3:15 PM

Wallace Kuhl & ASSOCIATES

FIGURE 12

Project: Solano Coummunity College - New Sicence Building
Project Location: Fairfield, California
WKA Number: 10642.01P

LOG OF SOIL BORING D6

Date(s) Drilled	**7/30/15**	Logged By	**ML**	Checked By	**DRG**
Drilling Method	**Solid Stem 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		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil Cuttings**
Remarks				Driving Method and Drop	**140 lb hammer, 30 inch 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
			Light brown, moist, medium dense, silty SAND (SM-Fill); fine to medium sand.		D6-1I	34	27.5	70	
	5		Dark brown, moist, very stiff, silty CLAY (CL). brown		D6-2I	22	27.2	99	
	10		Brown, moist, very stiff, sandy CLAY (CL; fine sand.		D6-3I	21	20.0	100	

Boring terminated at 10' below existing site grades.
Groundwater was not observed.

BORING LOG 10642.01P - SOLANO CC NEW SCIENCE BLDG.GPJ WKA.GDT 8/20/15 3:15 PM

WallaceKuhl & ASSOCIATES

FIGURE 13

UNIFIED SOIL CLASSIFICATION SYSTEM

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

OTHER SYMBOLS

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

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

PI = Plasticity Index

EI = Expansion Index

UCC = Unconfined Compression Test

TR = Triaxial Compression Test

GR = Gradational Analysis (Sieve)

K = Permeability Test

PP = Pocket Penetrometer

GRAIN SIZE CLASSIFICATION

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

WallaceKuhl & ASSOCIATES

UNIFIED SOIL CLASSIFICATION SYSTEM

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	14
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15

WKA NO. 10642.01P

Adapted from the Firm Flood Insurance Map for Solano County, Ca. dated May 4, 2009.
Projection: NAD 83, California State Plane, Zone II

Legend

ZONE A	Special flood hazard subject to inundation by the 1% annual chance flood.
ZONE AE	The floodway is the channel of a stream plus anyadjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increase in flood heights.
ZONE X	Areas of 0.2% annual chance of flood; areas of 1% annual chance flood with average depths of less the 1 foot or with drainage areas with less than 1 square mile; and areas protected by levees from 1% annual chance flood.

N

0 500 1,000
Feet

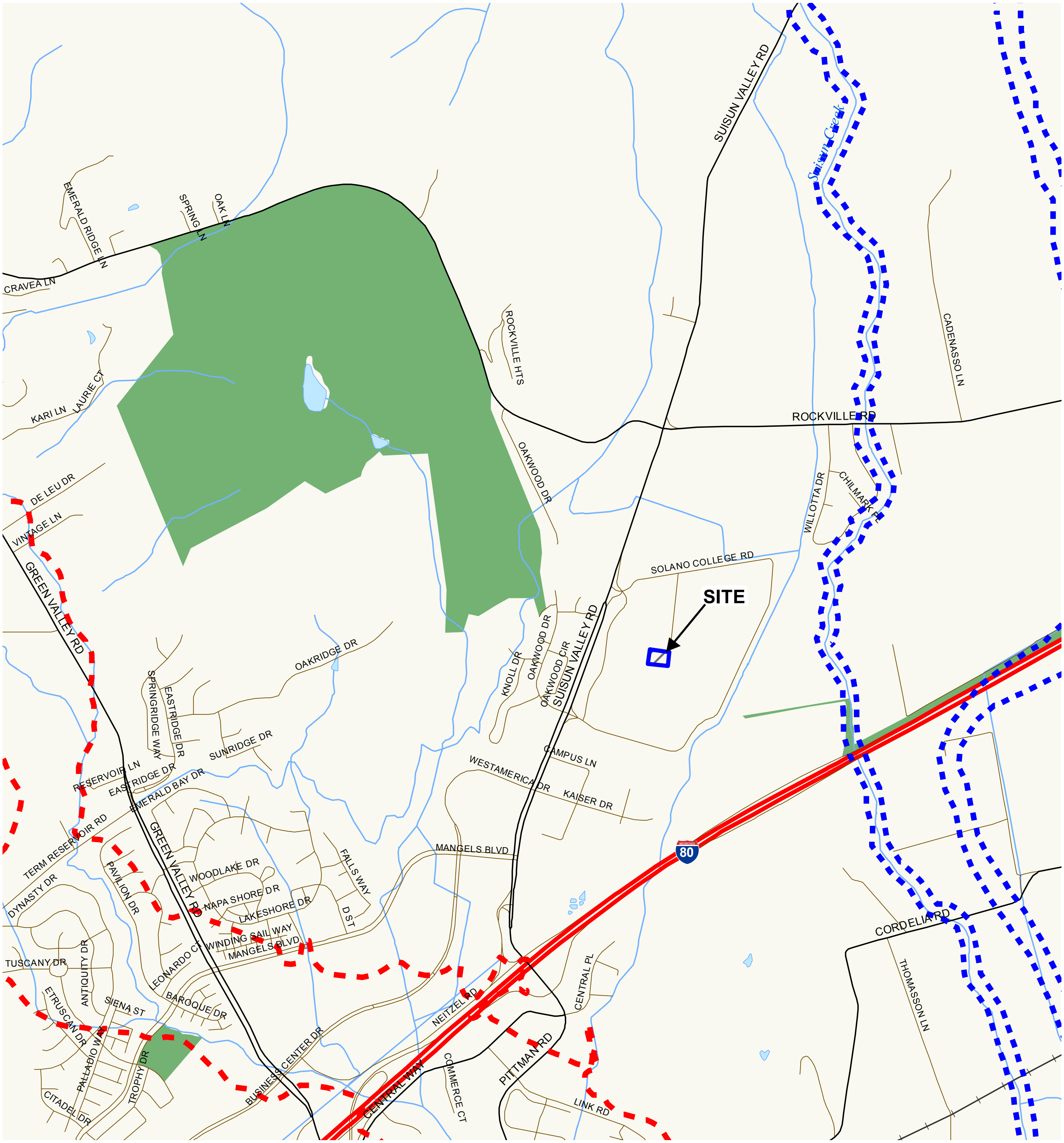
WallaceKuhl & ASSOCIATES

FEMA FLOOD MAP

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	15
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO. 10642.01P	



Adapted from the Firm Flood Insurance Map for Solano County, Ca. dated May 4, 2009.
Projection: NAD 83, California State Plane, Zone II

Legend

Lake Frey Inundation area

Lake Curry Inundation area

N

0 1,000 2,000
Feet

WallaceKuhl & ASSOCIATES

DAM INUNDATION MAP

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	16
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO. 10642.01P	

APPENDICES

APPENDIX A
General Project Information, Laboratory Testing and Results

APPENDIX A

A. GENERAL INFORMATION

The performance of a geotechnical engineering and geologic hazards investigation for the proposed science classroom building to be constructed at the central portion of the Solano Community College campus located at 4000 Suisun Valley Road in Fairfield, California, was authorized by Mr. Stan R. Arterberry on July 23, 2015. Authorization was for an investigation as described in our proposal letter dated June 11, 2015, sent to our client Solano Community College District c/o Kitchell (project construction manager), whose mailing address is 360 Campus Lane, Suite 203, in Fairfield, California 94534; telephone (707) 863-7847.

In performing this study, we made reference to a *Preliminary Floor Plan*, dated July 8, 2015, provide to us by Kitchell.

B. FIELD EXPLORATIONS

As part of our study for the proposed science classroom building, our field exploration included the drilling and sampling of six borings (D1 through D6) at the approximate locations shown in Figure 2.

The borings were drilled on July 30, 2015, utilizing a CME-75 truck-mounted, drill rig equipped with six-inch-diameter, solid helical-flight augers. Borings D1, D2 and D4 through D6 were drilled to depths ranging from 10 to 24½ feet below existing site grades. Boring D3 was initially drilled to a depth of approximately 15 feet below existing site grade using the solid helical-flight augers. After groundwater was encountered in Boring D3 and the water level was measured, the drilling method was switched to mud-rotary drilling. Boring D3 was then drilled with a 4-inch diameter drag bit to a maximum depth explored of 51½ feet below existing site grades.

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 split-spoon sampler and a 2-inch O.D., 1⅜-inch I.D., Standard Penetration Test (SPT) split-spoon sampler. Both split-spoon 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, or portion thereof, 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 engineer visually classified the soil recovered in the tubes and plastic bags. After the samples were classified, the ends of the tubes and plastic bags were sealed to preserve the natural moisture contents.

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

The Logs of Soil Borings containing descriptions of the soils encountered in each boring are presented as Figures 8 through 13. A Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained in Figure 14.

C. LABORATORY TESTING

Selected soil samples were tested to determine dry unit weight (ASTM D2937) and natural moisture content (ASTM D4643) and unconfined compression strength (ASTM D2166). The results of the moisture/density tests and compression strength tests are included on the boring logs at the depth each sample was obtained.

Three samples of fine-grained soil, considered to be representative of the near-surface soils, were subjected to Atterberg Limits testing (ASTM D4318). The test results are presented in Figure A1.

Six soil samples were tested for grain-size distribution (ASTM C136/D422) and percent passing the No. 200 sieve (ASTM D1140). The results of the gradation (grain-size) tests are contained in Figure A2. The percent passing the No. 200 sieve are included on the boring logs at the depth each sample was obtained.

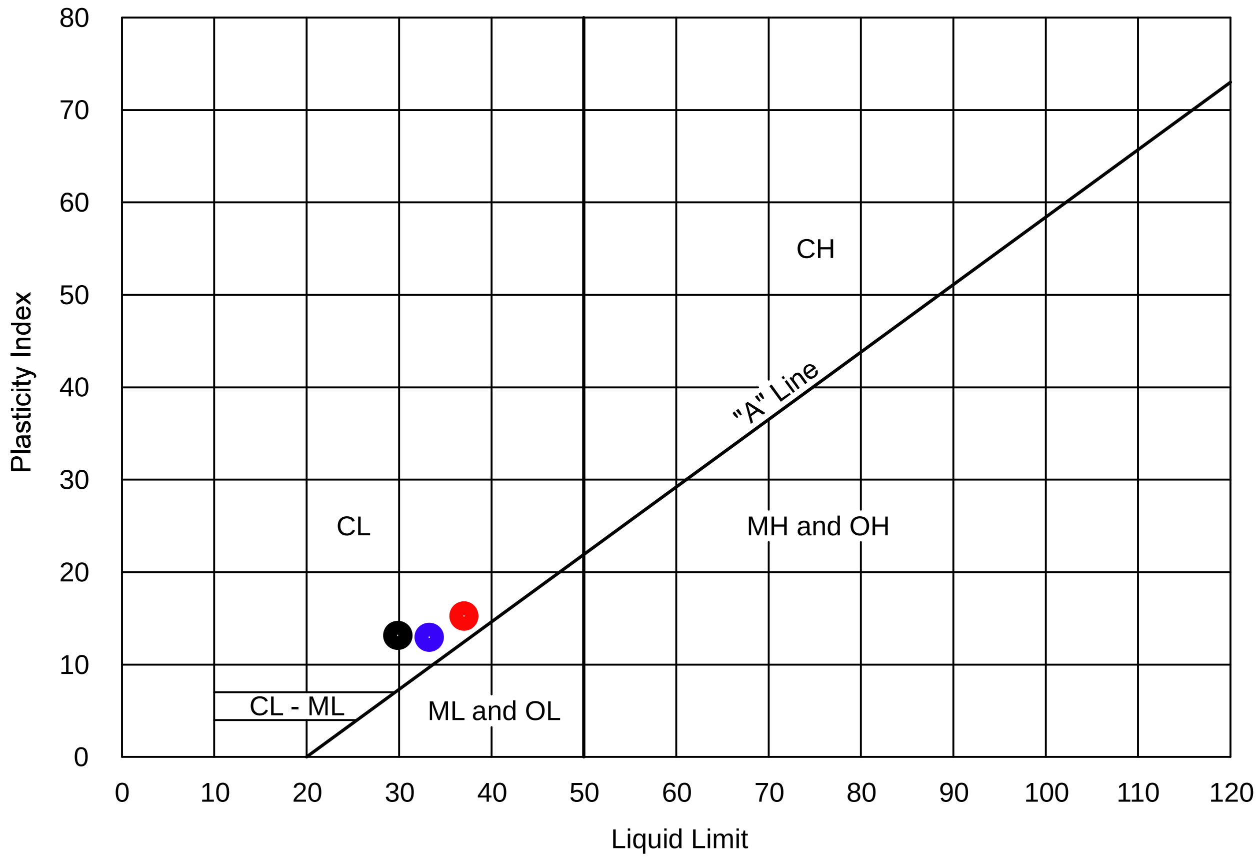
A representative bulk sample of near-surface soil was subjected to Expansion Index testing (ASTM D4829). The test results are presented in Figure A3.

One representative bulk sample of near-surface soil 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 A4.

One sample of representative near-surface soils was submitted to Sunland Analytical 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 test results are presented in Figures A5 and A6.

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 (%)		
●	D3-7I	30'-31.5'	21.5	30	13	76	CL
●	D3-8I	35'-36.5'	25.8	38	15	61	CL
●	D3-10I	45'-46.5'	27.1	34	13	79	CL

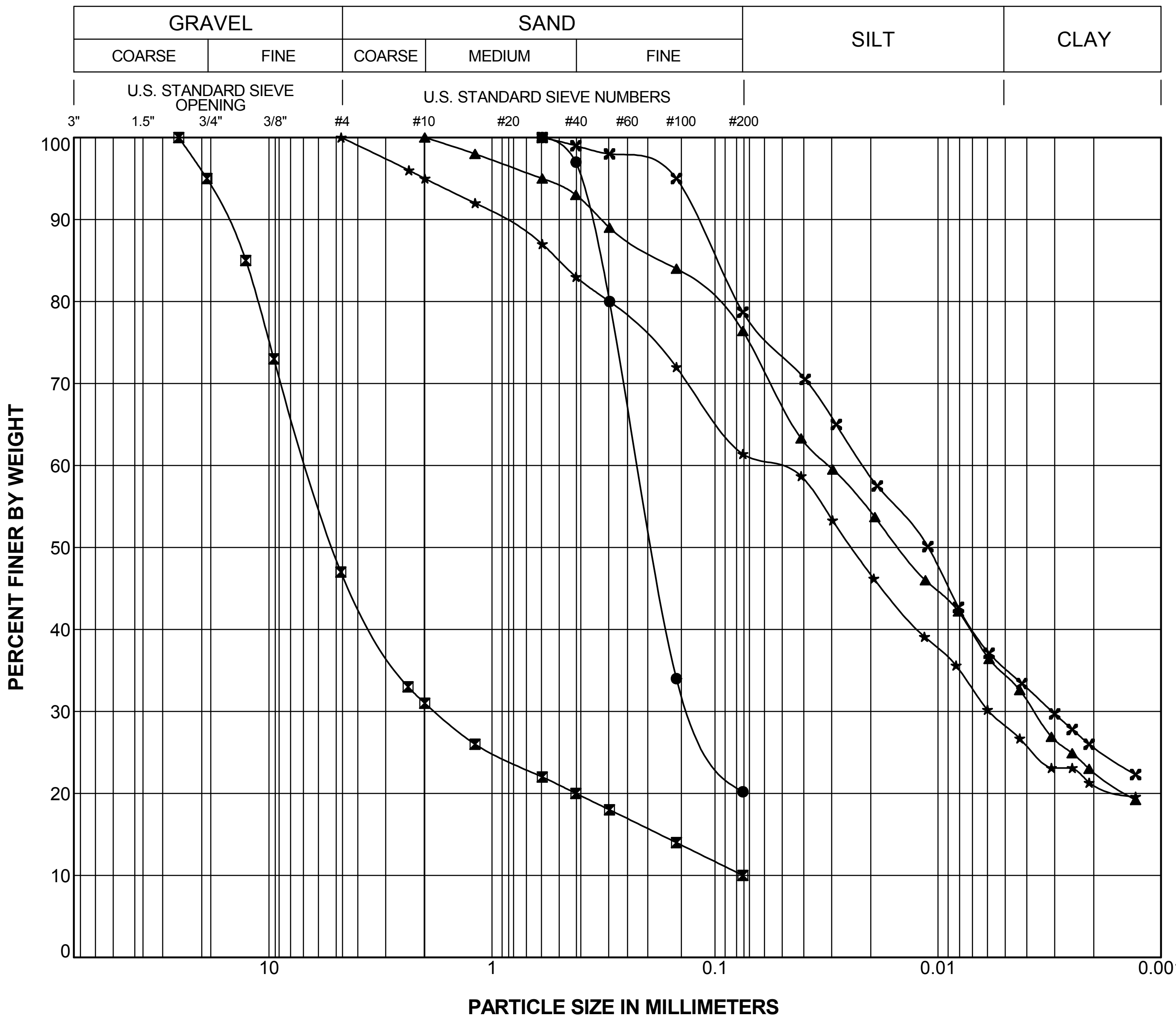
WallaceKuhl & ASSOCIATES

ATTERBERG LIMITS

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	9/15
WKA NO.	10642.01P



Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
D3	D3-5I	SM	20 to 21.5	●			silty SAND
D3	D3-6I	SP-SM	25 to 26.5	⧗			poorly-graded SAND with silt
D3	D3-7I	CL	30 to 31.5	▲	30	13	sandy CLAY
D3	D3-8I	CL	35 to 36.5	★	38	15	silty CLAY
D3	D3-10	CL	45 to 46.5	✖	34	13	silty CLAY

PARTICLE SIZE DISTRIBUTION

Project: Solano Coummunity College - New Sicence Building
WKA No. 10642.01P

FIGURE A2

WallaceKuhl & ASSOCIATES

GRAIN SIZE 10642.01P - SOLANO CC NEW SCIENCE BLDG .GPJ WKA.GDT 8/20/15 3:18 PM

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark brown, silty clay

LOCATION: D1

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 4'	31.3	28.3	100	**99**

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

WallaceKuhl & ASSOCIATES

EXPANSION INDEX

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	A3
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO.	10642.01P

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Dark brown, silty clay

LOCATION: D4 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (dial, inches x 1000)	Expansion Pressure (psf)	R Value
1	101	22.1	446	11	48	*

* Sample extruded, therefore R-Value = 5

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

Date Reported 08/07/2015
Date Submitted 08/03/2015

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 : 10642.01P SOLANO CC Site ID : D1 @ 0-4 FT.
Your purchase order number is 3729.
Thank you for your business.

* For future reference to this analysis please use SUN # 70164-146287.

EVALUATION FOR SOIL CORROSION

Soil pH	6.62	
Minimum Resistivity	1.31 ohm-cm (x1000)	
Chloride	37.5 ppm	00.00375 %
Sulfate	76.9 ppm	00.00769 %

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

WallaceKuhl & ASSOCIATES	**CORROSION TEST RESULTS** SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING Fairfield, California	FIGURE	A5
		DRAWN BY	RWO
		CHECKED BY	ML
		PROJECT MGR	DRG
		DATE	09/15
		WKA NO. 10642.01P	

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

Date Reported 08/07/2015
Date Submitted 08/03/2015

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 : 10642.01P SOLANO CC Site ID : D1 @ 0-4 FT.
Your purchase order number is 3729. Thank you for your business.

* For future reference to this analysis please use SUN # 70164-146288.

Extractable Sulfate in Water

TYPE OF TEST	RESULTS	UNITS
Sulfate-SO4	89.14	mg/kg

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

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

SOLANO COMMUNITY COLLEGE - NEW SCIENCE BUILDING

Fairfield, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	09/15
WKA NO.	10642.01P

APPENDIX B
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APPENDIX B - REFERENCES

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APPENDIX C

Liquefaction Analysis and Associated Data

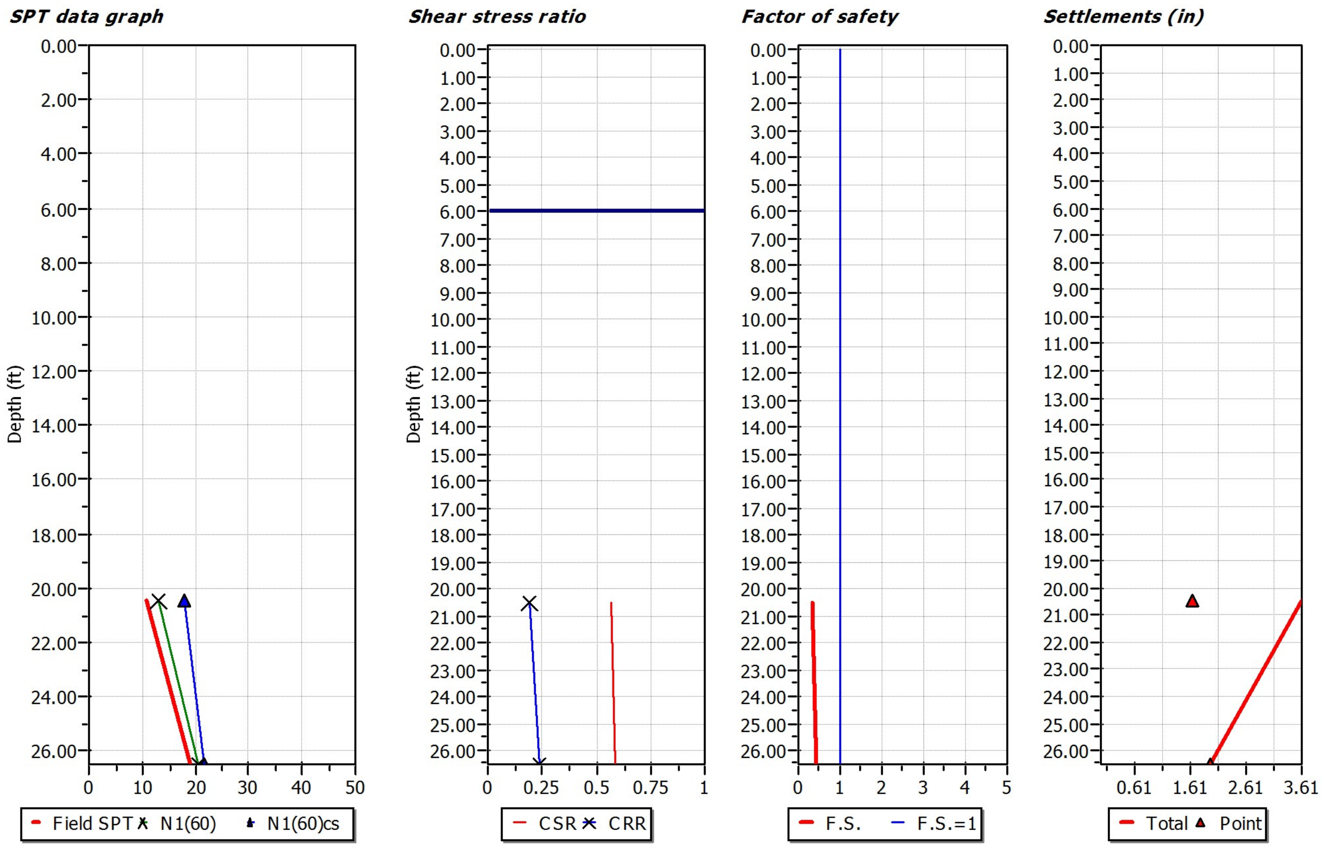
LIQUEFACTION ANALYSIS REPORT

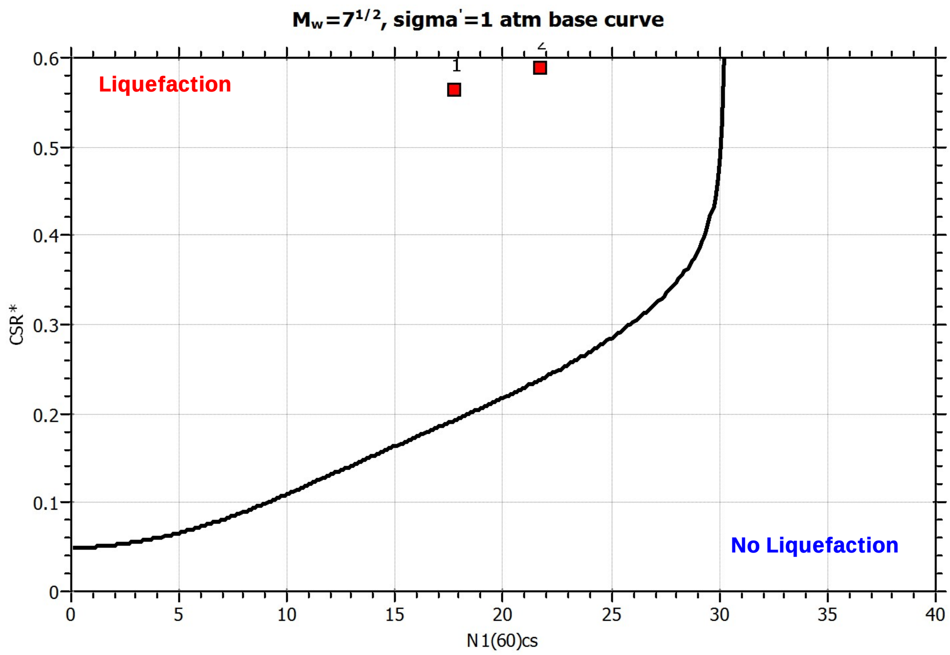
Project title : 10642.01P - SCC New Science Building

Project subtitle : D3

Input parameters and analysis data

In-situ data type:	Standard Penetration Test	Depth to water table:	6.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	6.60
Analysis method:	NCEER 1998	Peak ground accelaration:	0.80 g
Fines correction method:	Idriss & Seed	User defined F.S.:	1.00





:: Field input data ::

Point ID	Depth (ft)	Field N_{SPT} (blows/feet)	Unit weight (pcf)	Fines content (%)
1	20.50	11.00	120.00	20.00
2	26.50	19.00	120.00	10.00

Depth : Depth from free surface, at which SPT was performed (ft)
Field SPT : SPT blows measured at field (blows/feet)
Unit weight : Bulk unit weight of soil at test depth (pcf)
Fines content : Percentage of fines in soil (%)

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*
1	20.50	1.23	0.45	0.78	0.95	0.78	1.39	0.57	1.00	0.57
2	26.50	1.59	0.64	0.95	0.94	0.82	1.39	0.59	1.00	0.59

Depth : Depth from free surface, at which SPT was performed (ft)
Sigma : Total overburden pressure at test point, during earthquake (tsf)
u : Water pressure at test point, during earthquake (tsf)
Sigma' : Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$ CSR adjusted for M=7.5
K_{sigma} Effective overburden stress factor
CSR^* CSR fully adjusted

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
1	11.00	1.16	0.90	1.00	0.95	1.20	13.08	4.65	17.73	0.19
2	19.00	1.05	0.90	1.00	0.95	1.20	20.44	1.31	21.75	0.24

C_n : Overburden corretion factor
C_e : Energy correction factor
C_b : Borehole diameter correction factor
C_r : Rod length correction factor
C_s : Liner correction factor
$N_{1(60)}$: Corrected N_{SPT}
DeltaN : Addition to corrected N_{SPT} value due to the presence of fines
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines
$CRR_{7.5)}$: Cyclic resistance ratio for M=7.5

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
1	17.73	14.78	0.34	2.72	1.63
2	21.75	18.12	0.41	2.35	1.98

Total settlement : 3.61

$N_{1,(60)}$: Stress normalized and corrected SPT blow count
N_1: Japanese equivalent corrected value
FS_L: Calculated factor of safety
e_v: Post-liquefaction volumentric strain (%)
Settle.: Calculated settlement (in)

:: Liquefaction potential according to Iwasaki ::

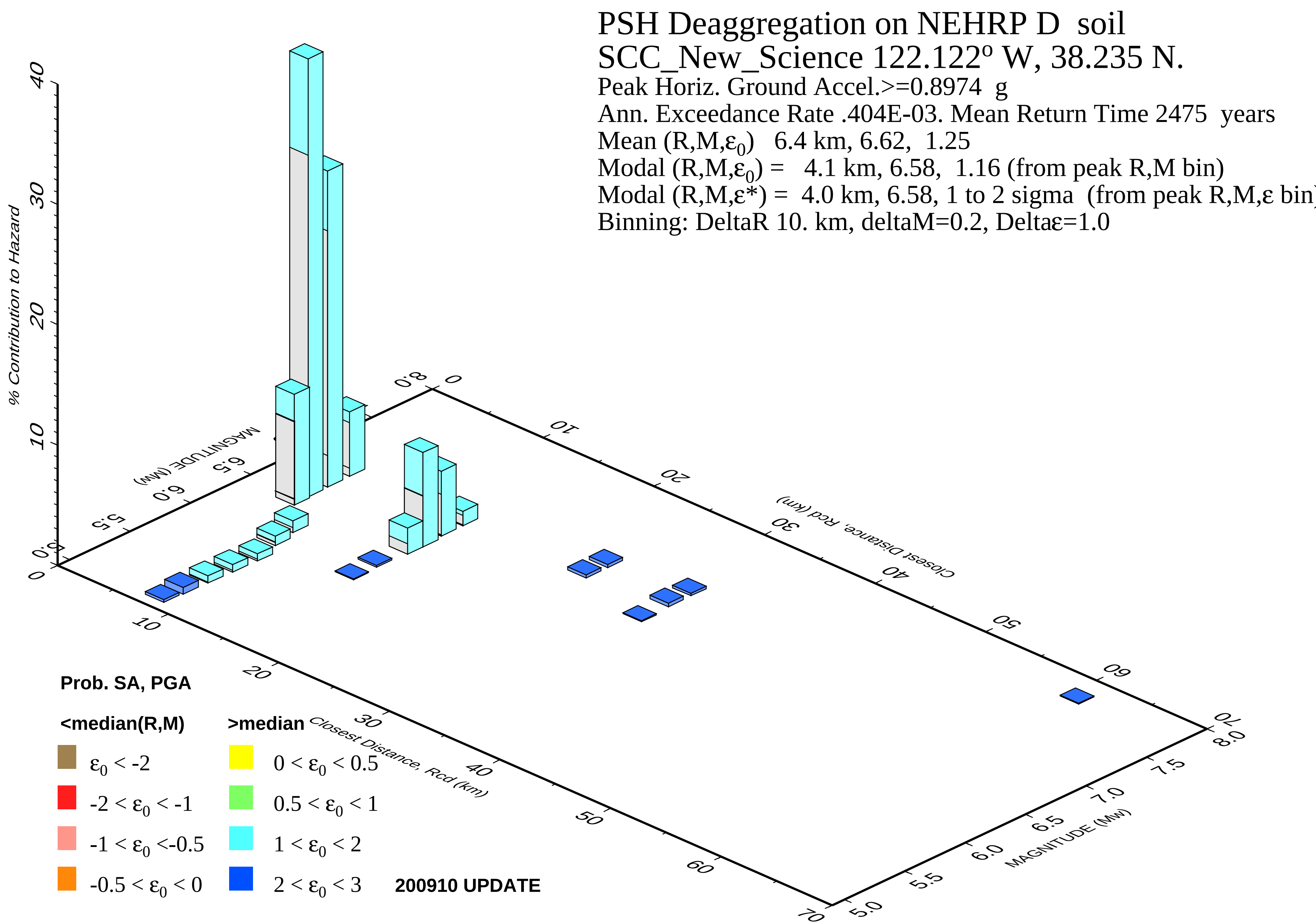
Point ID	F	w_z	I_L
1	0.66	6.88	28.32
2	0.59	5.96	6.48

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L

Overall potential I_L : 34.80

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain



GMT 2015 Aug 10 23:31:47 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 274. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

APPENDIX D
Guide Earthwork Specifications

APPENDIX D

GUIDE EARTHWORK SPECIFICATIONS

SOLANO COMMUNITY COLLGE – NEW SCIENCE BUILDING

Fairfield, California

WKA No. 10642.01P

PART 1: GENERAL

1.1 SCOPE

a. General Description

This item shall include all clearing of existing surface and subsurface structures, utilities, vegetation, rubbish, rubble, stockpiles 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 them or their 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 GEOTECHNICAL REPORT

a. A Geotechnical Engineering Report (WKA No. 10642.01P; dated September 4, 2015) 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 borings upon which to base conclusions drawn by the Contractor, all at no cost to the Owner.

1.4 EXISTING SITE CONDITIONS

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 SEASONAL LIMITS

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

PART 2: PRODUCTS

2.1 MATERIALS

a. All fill shall be of approved local materials from required excavations, supplemented by imported fill, if necessary. Approved local materials are defined as local soils 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. Imported fill materials shall be approved by the Geotechnical Engineer; they shall be compactable materials meeting 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 materials also shall be free of known contaminants and within acceptable corrosion limits, with appropriate documentation provided by the contractor.

c. Materials to be lime-treated shall be on-site clayey soils free from significant quantities of rubble, rubbish and vegetation and shall have been tested and approved by the Geotechnical Engineer.

d. 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. Lime used for stabilization shall be high-calcium or dolomitic quicklime conforming to the definitions in ASTM Designation C977.

1) When sampled by the Geotechnical Engineer from the lime spreader or during the spreading operations, the sample of lime shall conform to the following requirements:

Lime Quality

Property	ASTM Designation	Requirements
Available calcium and magnesium oxide [minimum percent (%)]	C25 or C1301 & C1271	High calcium quicklime: CaO > 90% Dolomitic quicklime: CaO > 55% & CaO + MgO > 90%
Loss on ignition [maximum percent (%)]	C25	7% (total loss) 5% (carbon dioxide) 2% (free moisture)
Slaking Rate [degrees Celsius (°C)]	C110	30°C rise in 8 minutes

2) When dry sieved in a mechanical sieve shaker for 10 minutes ±30 seconds, a 0.5 pound (lb) test sample of quicklime shall conform to the following grading requirements:

Lime Grading

Sieve Sizes	Percentage Passing
3/8-inch	98 - 100

f. The burden of proof as to quality and suitability of alternatives shall be upon the Contractor and/or Supplier and he shall furnish test data and all information necessary, as required by the Geotechnical Engineer. Written request for alternatives, accompanied by complete data as to the quality and suitability of the material shall be made in ample time to permit testing and approval without delaying the work. The Geotechnical Engineer shall be the sole judge as to the quality and suitability of alternatives and his decision shall be final. Documentation shall be provided to the Geotechnical Engineer no later than two weeks before the alternative material is imported to the site.

g. Lime from more than one source or of more than one type may be used on the same project but the different limes shall not be mixed.

h. The lime shall be protected from moisture until used and shall be sufficiently dry to flow freely when handled.

i. Water for use in subgrade stabilization shall be clean and potable and shall be added during mixing, remixing and compaction operations, and during the curing period to keep the cured material moist until covered.

j. 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 PAVEMENT AREAS

a. All surface and other sub-surface items associated with previous 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. Existing trees and any other vegetation 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. All demolition debris shall be hauled off site, or used as engineered fill, provided it is processed per the recommendations in Geotechnical 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. All structural areas (building pad, exterior flatwork, pavements etc.) shall be stripped of vegetation and organically laden topsoil. With prior approval of our office, stripping may be used in landscaped areas, provided they are kept at least five (5) from the buildings pad and other surface improvements, moisture conditioned and compacted.

d. Sub-excavation to remove clay soils from structural areas (building pad, exterior flatwork, etc.) shall be performed as recommended in the Geotechnical

Engineering Report, unless on-site clay subgrade soils are lime-treated as recommended in the Geotechnical Engineering Report.

e. The bottom of any required excavations, as well as areas to receive fill, achieved by excavation or remain at grade, should be scarified 12 inches (12”), uniformly moisture conditioned to at least two percent (2%) above the optimum moisture content and compacted to at least ninety percent (90%) of the maximum dry density as determined by ASTM D1557 Test Method. Compaction operations shall be undertaken with a heavy, self-propelled, sheepsfoot compactor capable of achieving the compaction requirements included in the Geotechnical Engineering Report.

f. When the moisture content of the fill material is less than two percent (2%) over the optimum moisture content as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

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

h. 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 UNTREATED 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. When the moisture content of the fill material is less than two percent (2%) over the optimum moisture content for clay soils or the optimum moisture content for granular soils (import fill materials), as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

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

d. After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted to at least ninety percent (90%) 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.

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

3.4 LIME-STABLIZED SUBGRADE CONSTRUCITON

a. On-site clay material to be treated shall be placed at a moisture content at least two percent (2%) over optimum moisture as defined by the ASTM D1557 Compaction Test.

b. Material to be treated shall be scarified and thoroughly broken up to the full depth and width to be stabilized. The material to be treated shall contain no rocks or solids larger than one and one-half inches (1½") in maximum dimension.

c. Mixing lime-treated material shall consist of the following:

1) Lime shall be added to the material to be treated at a rate of no less than four and a half pounds (4½ lbs.) of lime per cubic foot of compacted treated soil.

2) Lime shall be spread by equipment that will uniformly distribute the required amount of lime for the full width of the prepared material. The rate of spread per linear foot of blanket shall not vary more than five percent (5%) from the designated rate.

3) The spread lime shall be prevented from blowing by suitable means selected by the Contractor. Quicklime shall not be used to make lime slurry. The spreading operations shall be conducted in such a manner that a hazard is not present to construction personnel or the public. All lime spread shall be thoroughly ripped in, or mixed into, the soil the same day lime spreading operations are performed.

4) The distance which lime may be spread upon the prepared material ahead of the mixing operation will be determined by the Geotechnical Engineer.

5) No traffic other than the mixing equipment will be allowed to pass over the spread lime until after the completion of mixing.

6) Mixing equipment shall be equipped with a visual depth indicator showing mixing depth, an odometer or foot meter to indicate travel speed and a controllable water additive system for regulating water added to the mixture.

7) Mixing equipment shall be of the type that can mix the full depth of the treatment specified and leave a relatively smooth bottom of the treated section. Mixing and re-mixing, regardless of equipment used, will continue until the material is uniformly mixed (free of streaks or pockets of lime), moisture is at approximately two percent (2%) over optimum and the mixture complies with the following requirements:

Sieve Size	Minimum Percent Passing
1-1/2"	100
1"	95
No. 4	60

8) Non-uniformity of color reaction when the treated material, exclusive of one inch or larger clods, as tested with the standard phenolphthalein alcohol indicator, will be considered evidence of inadequate mixing.

9) Lime-treated material shall not be mixed or spread while the atmospheric temperature is below 35 degrees Fahrenheit (35°F).

10) Remixing of the treated soils shall be performed no sooner than twelve (12) hours after the initial mixing, and no later than seventy-two (72) hours after the initial mixing. The entire mixing operation shall be completed within seventy-two (72) hours of the initial spreading of lime, unless otherwise permitted by the Geotechnical Engineer.

d. Spreading and compacting of lime-treated material shall consist of the following:

1) The treated mixture shall be spread to the required width, grade and cross-section. The maximum compacted thickness of a single layer may be determined by the Contractor provided he can demonstrate to the Geotechnical Engineer that his equipment and method of operation will provide uniform distribution of the lime and the required compacted density throughout the layer. If the Contractor is unable to achieve uniformity and density throughout the thickness selected, he shall rework the affected area using thinner lifts until a satisfactory treated subgrade meeting the distribution and density requirements is attained, as determined by the Geotechnical Engineer, at no additional cost to the Owner.

2) The finished thickness of the lime-treated material shall not vary more than one-tenth foot (0.1') from the planned thickness at any point.

3) The lime-treated soils shall be compacted to a relative compaction of not less than ninety percent (90%) for structural areas (concrete foundation slabs, exterior flatwork, etc.) and ninety five percent (95%) for pavements as determined by the ASTM D1557 Compaction Test.

4) Initial compaction shall be performed by means of a sheepsfoot or segmented wheel roller. Final rolling shall be by means of steel-tired or pneumatic-tired rollers.

5) Areas inaccessible to rollers shall be compacted to meet the minimum compaction requirement by other means satisfactory to the Geotechnical Engineer.

6) Final compaction shall be completed within thirty-six (36) hours of initial mixing, and within four (4) hours of the final mixing. The surface of the finished

lime-treated material shall be the grading plane and at any point shall not vary more than eight one hundredths of a foot (0.08') foot above or below the grade established by the Civil Engineer except that when the lime-treated material is to be covered by material which is paid for by the cubic yard the surface of the finished lime-treated material shall not extend above the grade established by the Civil Engineer.

7) Before final compaction, if the treated material is above the grade tolerance specified in this section, uncompacted excess material may be removed and used in areas inaccessible to mixing equipment. After final compaction and trimming, excess material shall be removed and disposed of off site. The trimmed and completed surface shall be rolled with steel or pneumatic-tired rollers. Minor indentations may remain in the surface of the finished material so long as no loose material remains in the indentations.

8) At the end of each day's work, a construction joint shall be made in thoroughly compacted material and with a vertical face. After a partial-width section has been completed, the longitudinal joint against which additional material is to be placed shall be trimmed approximately three inches (3") into treated material, to the neat line of the section, with a vertical edge. The material so trimmed shall be incorporated into the adjacent material to be treated.

9) An acceptable alternate to the above construction joints, if the treatment is performed with cross shaft rotary mixers, is to actually mix three inches (3") into the previous day's work to assure a good bond to the adjacent work.

3.5 FINAL SUBGRADE PREPARATION USING UNTREATED SOILS

a. Final subgrade for the building pad and exterior flatwork shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. Clay soils should not be used in fills within the upper twelve inches (12") of the final building pad subgrade, unless the lime-treatment alternative include in the Geotechnical Engineering Report is selected. The upper twelve inches (12") of final building pad subgrade shall consist of compactable, granular soils, be brought to a uniform moisture content not less than the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test, unless the lime-treatment alternative include in the Geotechnical Engineering Report is selected.

b. The upper six inches (6") of any untreated final pavement subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety-five percent (95%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades, unless the lime-treatment alternative included in the Geotechnical Engineering Report is selected.

3.6 FINAL SUBGRADE PREPARATION USING TREATED SOILS

a. Final subgrade for the building pad and exterior flatwork using treated soils shall be constructed in accordance with Section 3.2 and Section 3.4 of these specifications. The upper twelve inches (12”) of treated final subgrades for the building pad and exterior flatwork shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

b. Final subgrade for pavements using treated soils shall be constructed in accordance with Section 3.2 and Section 3.4 of these specifications. The twelve inches (12”) of treated final pavement subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety-five percent (95%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

3.7 TESTING AND OBSERVATION

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

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

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

d. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, 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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