Geotechnical Engineering and Geologic Hazards Investigation

HIRAM JOHNSON HIGH SCHOOL
TRACK AND FOOTBALL FIELD

WKA No. 11843.01P

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Prepared for:

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INTRODUCTION

We have completed a geotechnical engineering and geologic hazards study for the design and construction of the Hiram Johnson High School Track and Football Field project in Sacramento, California (see Figure 1). The purposes of our study have been to explore the existing soil, geologic, and groundwater conditions at the site, and to provide geologic hazards and geotechnical engineering conclusions and recommendations for use by the other members of the design team for design and construction of the proposed project. This report presents the results of our study.

Scope of Work

Our scope of work included the following:

1. Site reconnaissance;
2. Review of United States Geological Survey (USGS) topographic maps, aerial photographs and available groundwater data;
3. Review of geologic maps and fault maps;
4. Review of seismic activity within 100 kilometers (62 miles) of the site;
5. Subsurface exploration, including the drilling and sampling of 12 borings to depths of approximately five to 22½ feet below the ground surface (BGS);
6. Bulk sampling of near-surface soils;
7. Laboratory testing of selected soil samples;
8. Engineering analyses; and,
9. Preparation of this report.

Figures and Attachments

The following figures are included with this report:

Table 1: Figures

Figure	Title	Figure	Title
1	Vicinity Map	6	Fault Map
2	Site Plan	7	Epicenter Map
3	USGS Topographic Map	8 - 20	Logs of Soil Borings
4	Geologic Map	21	Unified Soil Classification System
5	Geologic Cross Section	22	FEMA Flood 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).
- *Guide Earthwork Specifications* that may be used in the preparation of contract documents (Appendix C).

Proposed Development

Based on our communication with Verde Design, Inc. (Verde) and review of a site plan provided by Verde dated February 20, 2018, we understand the project will consist of the renovation of the existing track and field including the design and construction of a new synthetic turf football field, all-weather track, restroom building, scoreboard, home and visitor bleacher seating, press box, sports field lighting, and ticket booth. Associated development is anticipated to consist of underground utilities, exterior concrete flatwork, and asphalt concrete and/or concrete pavements.

A grading plan was not available when this report was prepared. However, based on existing site topography and our understanding of the proposed construction, we anticipate cuts and fills on the order of about one to two feet will be required to establish final subgrade levels across the site.

FINDINGS

Site Description

The existing Hiram Johnson High School site is located northeasterly of the intersection of 14th Avenue and 65th Street in Sacramento, California (Figure 1). The property encompasses a total area of approximately 38-acres and is identified by Sacramento County Assessor's Parcel Number (APN) 015-0301-001-0000. The site is bordered to the north by an existing residential housing development; to the east by Redding Avenue; to the south by 14th Avenue; and, to the west by 65th Street. The site is bound on all sides by existing residential housing developments. The site is located at approximately 38.5424º north latitude and 121.4270º west longitude.

The overall site is relatively flat, with the football field and track at a level approximately eight feet lower than the rest of the campus based on our observations. Review of the topographic map of the *Sacramento East Quadrangle*, published by the USGS, dated 1992, indicates the elevation of the site is approximately +40 feet relative to mean sea level (msl). Therefore, the surface elevation of the track is approximately +32 feet msl. A portion of the USGS topographic map containing the site is presented as Figure 3.

At the time of our explorations on March 23, 2018 and April 3, 2018, the project site was used as a track and field facility, with the athletic field covered in a light growth of turf grass. The existing track is surfaced decomposed granite.

Historical Aerial Photographic Review

We reviewed historical aerial photographs from the years 1947 through 2017. The site and surrounding vicinity are shown to be irrigated farmland with structures in the northwestern portion of the site in the 1947 photo. The photo from 1957 appears to show the campus in the early stages of construction. The photos from 1964 until 2017 appear to show the site in a similar condition as today.

General Site Geology

The site is located within the Great Valley geomorphic province of California, a 500-mile, northwest-trending structural trough, generally constrained to the west by the Coast Ranges and to the east by the foothills of the Sierra Nevada Range (Norris and Webb, 1990). The Great Valley consists of two valleys lying end-to-end, with the Sacramento Valley to the north and the San Joaquin Valley to the south.

The Sacramento Valley has been filled to its present elevations with thick sequences of sediment derived from both marine and terrestrial sources. The sedimentary deposits range in thickness from relatively thin deposits along the eastern valley edge to more than 25,000 feet in the south-central portion of the Great Valley (Norris and Webb, 1990). The sedimentary geologic formations of the Great Valley Province vary in age from Jurassic to Quaternary, with the older deposits being primarily marine in origin. Younger sediments are continentally derived and were typically deposited in lacustrine, fluvial, and alluvial environments with their primary source being the Sierra Nevada Range.

According to the USGS Geologic Map of the Sacramento Quadrangle (Wagner, 1981), the project site is underlain by the Quaternary-aged Riverbank Formation (Qr). The California Geological Survey's Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle (Gutierrez, 2011) identifies the area underlying the site as the middle unit of the Riverbank Formation (Qr2). Review of the Preliminary geologic map of Cenozoic deposits of the Davis, Knights Landing, Lincoln, and Fair Oaks quadrangles California identifies the site as underlain by the lower unit of the Riverbank Formation (Qr1) (Helley, 1979). The geologic materials that comprise the Riverbank Formation are primarily fine-grained sediments with horizontal stratification. The mapped geology was found to be consistent with the subsurface soil conditions encountered within our soil borings performed at the site, which indicate cemented alluvial deposits at approximately 22½ feet below site grade.

A copy of a portion of the 2011 Preliminary Sacramento Quadrangle Geologic Map is provided as Figure 4. Geologic cross sections are included in this report as Figure 5.

<u>Faulting</u>

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 *Sacramento County General Plan* (Sacramento County, 2011) indicates that the site is in an area of low-intensity ground motion. The site is <u>not</u> located within a designated Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 2007). The nearest Alquist-Priolo Earthquake Fault Zones have been established around the Green Valley Fault and the Cordelia Fault; the closest edge of these fault zones is located approximately 68.1 kilometers (42.3 miles) west-southwest of the site.

Using the *Revised 2002 California Probabilistic Seismic Maps* (Cao, 2003), we have prepared Table 2 containing faults and fault systems within 100 kilometers (62 miles) of the site that are considered capable of producing earthquakes with a moment magnitude (M_W) of 6.5 or greater. A fault map is presented as Figure 7. The nearest of these faults are associated with the

Foothills Fault System, which is used to model seismic sources along the west side of the Sierra Nevada foothills. Segment No. 1 of the Foothills Fault System is located about 37.5 kilometers (23.3 miles) east-northeast of the site.

Table 2: Fault Systems within 100 Kilometers of the Subject Site

Fault Name	Distance		Maximum Earthquake Magnitude (Mw)
	Miles	Kilometers	
Foothills Fault System	28.2	45.4	6.5
Great Valley 4a, Trout Creek	31.1	50.0	6.6
Great Valley 4b, Gordon Valley	32.9	52.9	6.6
Great Valley 5, Pittsburg Kirby Hills	34.8	55.9	6.5
Great Valley 3, Mysterious Ridge	35.2	56.6	6.9
Hunting Creek-Berryessa	42.2	68.0	7.1
Green Valley Connected	42.3	68.1	6.2
Greenville Connected	50.5	81.3	6.2
West Napa	51.2	82.5	6.5
Great Valley 7	56.0	90.1	6.7
Mount Diablo Thrust	56.7	91.3	6.6
Calaveras; CN+CC+CS	59.3	95.4	6.8
Calaveras; CN	59.3	95.4	6.8
Calaveras; CN+CC	59.3	95.4	6.8
Great Valley 2	60.0	96.7	6.4

Coseismic Ground Deformation

The California State Legislature passed the Seismic Hazards Mapping Act (SHMA) in 1990 (Public Resources Code Division 2, Chapter 7.8) following the 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 SP117). The Hiram Johnson High School site is not mapped within any seismic hazard zones and there are no published maps available on the CGS website that cover the Hiram Johnson High School site.

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 2016). 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) VII. According to the tabulated data, the most intense earthquake ground shaking within 100 kilometers of the site resulted from an MR 6.4 earthquake on April 19, 1892, with an epicenter located approximately 52.3 kilometers (32.5 miles) southwest of the site.

Subsurface Soil Conditions

Twelve (12) exploratory borings (D1 through D12) were performed on March 23, 2018 and April 3, 2018 at the approximate locations shown on the attached Site Plan (Figure 2).

Borings D1 through D5 were performed at grade consistent with the adjacent roads and major portion of the High School campus. Borings D6 through D12 were performed in the football field and track area which is approximately eight feet lower than the adjacent road and the rest of the campus. Borings D1 through D7, and D9 through D12, had approximately one to two inches of grass at the surface, and D8 had decomposed granite at the surface. The surface and near-surface soils at boring locations D1 through D12 consisted of stiff to very hard sandy clays or loose to medium dense silty sands, interbedded with medium dense to very dense sandy to clayey silts and silty to clayey sands extending to the explored depths of five to 22½ feet BGS. Loose sands were only encountered at the surface of boring D12. At borings D2 and D3, sandy gravel was encountered at depths of about 18 ½ to 20 feet below the ground surface and extended to the explored 20 to 22½ foot depths of those borings. At boring D2, the boring was terminated at a depth of 22½ feet BGS due to practical auger refusal when cemented alluvial deposits consistent with the Riverbank Formation were encountered.

For specific information regarding the soil conditions at a specific location, please refer to the Logs of Soil Borings (Figures 8 through 19).

Groundwater

Groundwater was not encountered within any of the borings performed on March 23, 2018 or April 3, 2018. To supplement the groundwater data, we reviewed available groundwater data published by the California Department of Water Resources (DWR, 2017) from a monitoring well (08N05E15E001M) located about ¼-mile north of the site. DWR has monitored water levels in the well from 1966 to at least 1987. Ground surface elevation at the well is indicated to be about +40 feet msl which is approximately the subject site's elevation. Groundwater measurements at the DWR well have fluctuated from a "high" of about -3 feet msl in March 1984 to a "low" of about -14 feet msl in September 1966. Based on this data, groundwater elevations at the site have fluctuated from about 35 to 46 feet below field site grades between 1966 and 1987.

CONCLUSIONS

Seismic Site Class

The soil conditions encountered at the boring locations indicates the site is underlain by the middle member of the Pleistocene- aged (130,000 to 450,000 years before present) Riverbank Formation. The Riverbank Formation has been identified as a material meeting Site Classification C (Wills, et al., 2001). Standard Penetration Test (SPT) blow counts obtained within the upper 20 feet at boring D2 varied from 17 to 31 blows per foot. To calculate the Site Classification based on Table 20.3-1 of *American Society of Civil Engineers (ASCE) Standard 7-10*, we have conservatively assumed the blow counts between 20 and 100 feet below the ground surface are at least 15 blows per foot based on the site geology and the blow counts obtained within the upper 20 feet of the soil profile.

Based on the soil conditions encountered at the boring locations, the SPT blow counts obtained within the upper 20 feet of the boring, conservatively assuming the blow counts are least 15 blows per foot from depths of 20 to 100 feet below the ground surface, and the documented site geology, it is our opinion that a Site Class D is applicable to the Hiram Johnson High School site, in accordance with Table 20.3-1 of *ASCE 7-10* and the 2016 *California Building Code* (*CBC)*.

2016 CBC/ASCE 7-10 Seismic Design Criteria

The 2016 edition of the CBC references the *ASCE 7-10* for seismic design. The following seismic parameters provided in Table 3 were determined based on the site latitude and longitude using the public domain computer program developed by the USGS. The seismic design parameters summarized below in Table 3 may be used for seismic design of the elementary school buildings.

Table 3: 2016 CBC/ASCE 7-10 Seismic Design Parameters

Latitude: 38.5424° N Longitude: 121.4270° W	ASCE 7-10 Table/Figure	2016 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2 seconds	Figure 22-1	Figure 1613.3.1(1)	S_S	0.640 g
1.0 second Period MCE	Figure 22-2	Figure 1613.3.1(2)	S_1	0.284 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	1.288
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.832
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-37	S_{MS}	0.825 g
	Equation 11.4-2	Equation 16-38	S_{M1}	0.520 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	0.550 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.347 g
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 exposed to 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 the subsurface exploration at the site, performed on March 23, and April 3, 2018 revealed the underlying soils generally consist interbedded clay, silt, and sand layers extending to the explored five to 22½ feet BGS. Practical auger refusal was encountered within cemented soils at a depth of 22½ feet below the ground surface at Boring D2 and a depth of 13 feet below the ground surface at Boring D12. Based upon the relatively thick layers of cohesive soils, the age of the geologic materials, and the lack of historic occurrence of liquefaction, it is our opinion that the potential for liquefaction of the soils beneath most of the site is relatively low.

Our review of available historical groundwater data for the area indicates groundwater levels have been as high as about 40 feet below the existing ground surface. The soil conditions encountered below the anticipated high groundwater are anticipated to consist of relatively dense/hard soils based on site geology.

Based on the soil, groundwater, and geology conditions at the site, in our opinion, significant deposits of saturated, loose, cohesionless soils do not exist beneath the site and the potential for liquefaction and significant seismic settlement of the soils beneath the site is very low.

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, 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. The site is not located within a seismic hazard zone pursuant to the Seismic Hazard Zone Mapping Act.

Volcanic Hazards

The school 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 the historic topographic maps. We noted that the approximately eight-foot high slopes surrounding the track and field are flatter than a 2:1 (horizontal: vertical) inclination, and no evidence of existing or past slope instability was noted. Review of the Health and Safety Element of the

Sacramento County General Plan (Sacramento County, 2011) revealed the site is not considered to be in an area of landslide potential. It is our opinion that the existing slopes surrounding the track and field are stable and will remain stable provided that the recommendations presented in this report are followed during construction of the planned track and field improvements.

Flood Hazards

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map for Sacramento County, California (Community-Panel Numbers 06067C0195H, August 16, 2012), the school area is located within OTHER AREAS - ZONE X defined as “Areas determined to be outside the 0.2% annual chance floodplain.” The FEMA flood map for the site vicinity is presented in Figure 21 of this report.

Dam Inundation

The Health and Safety Element of the Sacramento County General Plan (Sacramento County, 2011) identifies four major dams and two minor dams that have the potential for human injury or loss of life in the county if failure were to occur. The site lies approximately 19 miles southwest of the Folsom Dam which is within an area likely to be affected by a failure of Folsom Dam. The California Office of Emergency Services indicated that the Site would be affected by the failure of the Folsom, Nimbus, and Oroville Dams.

Tsunamis and Seiches

The publicly available “Tsunami Inundation” maps developed by the CGS do not cover the site. Given 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 ground water, oil, natural gas or oxidation of peat. Based on our subsurface exploration, the soil at the project site generally consists of silt and clay layers with interbedded sand layers to the explored depths of five to 22½ feet.

DWR has mapped the entire Central Valley of California as having potential (low to high) for future land subsidence; however, DWR indicates the mapping is intended to be advisory only to assist state and local agencies in defining areas of potential subsidence that may require additional study (DWR, 2014).

Based on the subsurface conditions encountered at the site, it is our opinion that settlement at the site due to subsidence is very unlikely, provided the recommendations of this report are followed.

Naturally Occurring Asbestos (NOA)

Pleistocene alluvial sediments directly underlie the site. 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) indicate the site is not underlain by ultramafic rocks likely to contain asbestos. This is consistent with our observations.

Radon-222 Gas

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

According to the Environmental Protection Agency's Map of Radon Zones, the 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

The results of our study indicate the undisturbed site soils can provide adequate support for the proposed structures and pavements provided the recommendations of this report are carefully followed. Clearing operations to remove vegetation and remnants of any existing improvements will disturb the underlying materials and create loose and variable soil conditions. Disturbed soils must be excavated to expose a firm base and the excavations widened, as necessary to provide equipment access, and backfilled with engineered fill to provide uniform support for the planned structures. Engineered fill that is properly placed and compacted as recommended in this report will be capable of supporting the proposed structures and pavements.

Soil Expansion Potential

Laboratory testing of soils collected from the upper five feet within the planned location of the proposed buildings revealed the near-surface clay soils possess a "medium" expansion potential (Expansion Index [EI]=57) when tested in accordance with the American Society for Testing and Materials (ASTM) D4829 test method (see Figure A4). Based on these test results, the near-surface soils at the site are considered capable of exerting moderate expansion pressures on building foundations, interior floor slabs, and exterior flatwork.

To reduce the impact of the near-surface expansive clay soils, at least 12 inches of imported, compactable, very low-expansive (Expansion Index < 20) granular soils will be required beneath interior and exterior 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. Specific recommendations for subgrade preparation and engineered fill construction are included in this report in this report to reduce the effect of expansive clay soils on the planned improvements.

Pavement Subgrade Quality

Based upon our laboratory testing of the anticipated subgrade soils, the near-surface soils are indicated to be poor subgrade quality materials (R-value of 5) for support of asphalt concrete pavements. Our experience indicates that chemical treatment of the soils can be used to improve the pavement subgrade support quality of the soil.

Chemical Treatment of Soil

Chemical treatment of the soil can be a very effective and economical method to increase the subgrade quality of poor to moderate quality subgrades to support pavements; to reduce the moisture content of near-saturated soils, enabling construction to proceed during or shortly after the rainy season; and, to reduce the expansive characteristics of clayey subgrades.

Based on the relatively poor pavement subgrade quality of the near-surface soils, we anticipate chemical treatment of the near-surface soils can significantly improve the support quality of the soils and reduce the required thickness of the base materials. Based upon laboratory testing and our experience with similar soil types, it is our opinion that pavements supported on native soils treated with at least 4½ pounds of lime per cubic foot of compacted soil can be improved to an R-value greater than 50.

The performance of chemically stabilized soils is critically dependent on uniform mixing of the lime into the subgrade and providing for a proper curing period following amendment with the lime. An experienced lime-stabilization contractor coupled with a comprehensive quality control program are generally required to achieve the best possible stabilized subgrade.

The major disadvantage of chemically treated subgrades supporting pavements results from shrinkage of the treated material, similar to shrinkage of structural concrete, and reflective cracking through the asphalt concrete surface. Proper curing techniques, as outlined in the attached *Guide Earthwork Specifications*, can minimize this effect.

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 not encountered in any of our 12 borings performed on March 23, 2018 and April 3, 2018. Review of available groundwater data revealed the groundwater elevation at nearby monitoring wells has ranged from 40 to 51 feet below the existing well ground surface. Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation, local pumping, the proximity to the American River, and other factors. Locally perched shallower groundwater may be encountered.

Based on current explorations performed at the site and historical groundwater data, we do not anticipate excavations to encounter permanent groundwater, although locally perched water could be encountered and require localized dewatering (depending on the time of year). If perched is encountered, the use of sumps or submersible pumps 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 likely that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soils, intended for use as engineered fill, will require a prolonged period of dry weather and aeration or chemical treatment to reach a moisture content suitable for proper 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.

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

Soil Corrosion Potential

Two soil samples were tested to determine resistivity, pH, 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 potential test results performed by Sunland Analytical are presented on Figures A6 through A9.

Table 4: Soil Corrosivity Testing Results

Analyte	Test Method	D2 (0-3')	D5 (0 – 3')
pH	CA DOT 643 Modified*	6.26	6.49
Minimum Resistivity	CA DOT 643 Modified*	3750 ϑ-cm	2680 ϑ-cm
Chloride	CA DOT 422	5.5 ppm	5.1 ppm
Sulfate	CA DOT 417	16.5 ppm	14.2 ppm

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

The California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, Corrosion Guidelines (Version 2.1 dated January 2015), considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or the pH is 5.5 or less. Based on this criterion, the on-site soils tested are not considered corrosive to steel reinforcement properly embedded within Portland cement concrete (PCC).

Table 19.3.1.1 – Exposure Categories and Classes, of American Concrete Institute (ACI) 318-14, Section 19.3 – Concrete Durability Requirements, as referenced in Section 1904.1 of the 2013 CBC, indicates the severity of sulfate exposure for one of the samples tested is Exposure Class S0. Exposure Class S0 is assigned for conditions where the water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern. The project structural engineer should review the requirements of ACI 318 and determine their applicability to the site.

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 in this report are based on assumed excavations and fills on the order of about one to two feet for the development of the site. We consider it essential that our office review grading and structural foundation plans to verify the applicability of the following recommendations, to verify that the intent of our recommendations has been incorporated into the construction documents, and to provide supplemental recommendations, if necessary.

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 is the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Clearing

The proposed improvement areas should be cleared of existing vegetation, decomposed granite track surface, debris, and other deleterious materials to expose undisturbed native soils. Where practical, the clearing should extend a minimum of five feet beyond the limits of the proposed structural areas of the site. Existing underground utilities to be abandoned should be completely removed, including existing trench backfill. All trees/large brush designated for removal should include the rootballs and roots ½ inch or larger in size.

Depressions resulting from removal of underground structures (e.g., foundations, utilities, etc.) should be cleaned of loose soil and properly backfilled in accordance with the recommendations of this report.

Existing pavements and flatwork (asphalt concrete and concrete), if any, that are not incorporated into the new design should be broken up and removed from the site. Alternatively, pulverized asphalt and Portland cement concrete rubble may be used as fill provided it is processed into fragments less than three inches in largest dimension, is mixed with soil to form a compactable mixture, and approved by the Owner.

The near-surface soils on the existing football field containing the grass and excessive organic soils should be removed and not used within the artificial turf, pavements, slabs, and building areas. For this project, the acceptable organic content is less than four percent (4%) organics by weight as determined by ASTM D2974 (Organic Content by Ignition Method). In our opinion, soils having excessive organic matter contents should be removed to expose undisturbed native soils with acceptable organic contents.

Soils containing organic material may be used in landscape areas. However, the landscape architect should have the final decision as to the placement of soils containing organic material in landscape areas.

Where encountered, any loose, soft or saturated soils should be cleaned out to firm native soil and backfilled with engineered fill in accordance with the recommendations in this report. It is important that the Geotechnical Engineer's representative be present for a sufficient time during clearing operations to verify adequate removal of the surface and subsurface items, as well as the proper backfilling of resulting excavations.

Subgrade Preparation

Following the site clearing operations, surfaces to receive fill and at-grade areas to receive improvements should be scarified to a depth of at least 12 inches, 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.

The upper 12 inches of final subgrade for the interior concrete 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 Lime Treatment Alternative section of this report. All soils supporting interior and exterior slab-on-grade concrete should be uniformly compacted to 90 percent of the ASTM D1557 maximum dry density.

The upper six inches of pavement and synthetic turf subgrades should be moisture conditioned to at least the optimum moisture content and compacted to no less than 95 percent relative compaction, regardless of whether final subgrade is achieved by excavation, filling or left at existing grade. Final pavement and synthetic turf subgrade processing and compaction should be performed **AFTER** completion of underground utilities and must be stable under construction traffic prior to aggregate base placement.

Loose, soft, or saturated soils encountered during the subgrade preparation operations should be properly prepared (i.e., scarified and compacted) or excavated to expose firm, undisturbed native materials.

Site clearing and compaction operations should be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of the subgrade under compactive load and identify loose or unstable soils that could require additional subgrade preparation.

If unstable soil conditions are encountered during subgrade preparation, stabilizing the subgrade soils may be required to achieve a stable pavement subgrade. Typical recommendations for stabilizing unstable soil subgrades include: cross-rip, blade, and aerate; removal and replacement; geogrid stabilization; and/or, chemical treatment. Stabilization recommendations will depend on the actual conditions encountered at the time of construction and should be determined by the project team, including the Geotechnical Engineer.

Compaction of the soil subgrade should be achieved using a heavy, self-propelled, sheepsfoot compactor and must be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of the subgrade under the compaction loads, and identify loose or unstable soil conditions that could require additional excavation. Loose, soft or saturated soil deposits encountered below the depth of scarification during compaction operations should be removed to expose firm undisturbed soils as identified by the Geotechnical Engineer's representative and backfilled with engineered fill as recommended in this report. Difficulty in achieving subgrade compaction or unusual soil instability may be indications of loose soils associated with past subsurface items. Should these conditions exist, the materials should be excavated to check for subsurface structures and the excavations backfilled with engineered fill. We recommend construction bid documents contain a unit price (price per cubic yard) for additional excavation due to unstable wet soil or the presence of unsuitable materials and replacement with engineered fill.

Engineered Fill Construction

Any fill placed within the construction areas should be an approved material, free of significant quantities of organics or other deleterious materials. The fill should be spread in level layers not exceeding six inches in compacted thickness and compacted to a minimum of 90 percent of the maximum dry density as determined by the ASTM D1557 test method. Engineered fill consisting of onsite native clays should be moisture conditioned to at least two percent above the optimum moisture content and imported granular soils should be moisture conditioned to at least the optimum moisture content. Subgrade soils should be maintained in a moist condition similar to the condition achieved during compaction.

Imported fill, including the upper 12 inches of the final synthetic turf, building pad and exterior flatwork subgrades, should be an approved compactable granular material, have an Expansion Index of 20 or less, and be free of particles larger than three inches in maximum dimension. The Geotechnical Engineer must approve import material *before* being transported to the project site. In addition, we recommend that the contractor supply a certification for any imported fill materials that designates the fill materials to be free of known contaminants per Department of Toxic Substances Control's (DTSC, 2001) guidelines for clean fill and have corrosion characteristics within acceptable limits.

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

<u>Lime-Treated Subgrade - Alternative</u>

Based on the clayey near-surface soil conditions at the site, consideration may be given to chemically treating the subgrade soils supporting the proposed synthetic turf, building pads, exterior flatwork, and pavement areas to provide a uniform bearing surface, increase the subgrade quality of poor to moderate quality subgrades to support pavements; to reduce the moisture content of near-saturated soils, enabling construction to proceed during or shortly after the rainy season; and, to reduce the expansive characteristics of the clayey soil subgrade.

Lime-treated subgrade soils should be treated with at least 4½ pounds of high-calcium or dolomitic quicklime per square foot at a depth sufficient to produce a finished compacted lime-treated layer 12 inches thick. Lime-stabilized soils should be compacted to at least 90 percent relative compaction within building pad and exterior flatwork areas and at least 95 percent within synthetic turf and pavement areas, at a moisture content at least two percent over optimum conditions.

If necessary, our firm can provide additional recommendations for subgrade stabilization based on the soil conditions at the time of earthwork construction.

If undisturbed native soils are to be lime-treated, the scarification and compaction procedures outlined in the Subgrade Preparation section of this report are not required within the upper 12 inches of the final subgrade, prior to lime-treatment.

Utility Trench Backfill

Utility trench backfill within structural areas (track, field, grandstands, buildings, slabs and pavements) should be mechanically compacted as engineered fill in accordance with the following recommendations. Bedding and initial backfill around and over the pipe should conform to the pipe manufacturers recommendations and applicable sections of the governing agency standards. Utility trench backfill should be placed in maximum 12-inch thick lifts (compacted thickness), moisture conditioned to at least two percent above the optimum moisture content and mechanically compacted to at least 90 percent of the ASTM D1557 maximum dry density. Utility trench backfill within the upper six inches of the final track, field, and pavement subgrade should be compacted to at least 95 percent of the maximum dry density. Utility trench backfill should be continuously observed and tested during construction.

Backfill for the upper 12 inches of trenches must match the quality of the adjacent materials. That is, if the upper 12 inches of subgrades for the building pad and exterior flatwork consists of lime-treated soils, the upper 12 inches of trench backfill should consist of controlled density fill (CDF) or aggregate base. Lime-treated soils removed from excavations are not suitable for use as replacement for the soil treated in place. However, the lime-treated soils removed from excavations may be used as general site engineered fill is approved by the Geotechnical Engineer.

We recommend that all underground utility trenches aligned nearly parallel with foundations be at least five feet from the outer edge of foundations, wherever possible. If this is not practical, the trenches should not encroach into a zone extending at a one horizontal to one vertical (1:1) inclination below the 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

Proposed buildings at the site may be supported upon a continuous perimeter foundation with isolated interior spread foundations. Goal posts, score boards, light standards and other ancillary structures that will proportionally support more lateral loading than axial loading may be supported upon a deep foundation system consisting of drilled, cast-in-place reinforced concrete piers (drilled piers).

Our recommendations for shallow spread foundations and drilled piers are provided in the following sections.

Shallow Foundations

The proposed buildings 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. Continuous foundations should maintain a minimum width of 12 inches; isolated spread foundations should be at least 18 inches wide. A continuous, reinforced foundation is recommended 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 buildings. All foundations should extend at least 18 inches below lowest surrounding soil grade.

Foundations constructed as described above may be sized utilizing a net allowable bearing capacity of 2000 pounds per square foot (psf) for dead plus live loads. This value may be increased by 1/3 to include wind or seismic forces. The weight of foundation concrete extending below the lowest adjacent soil grade may be disregarded in sizing computations.

The project structural engineer should determine the final dimensions and structural reinforcement of the foundations.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.25, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 250 psf per foot of depth. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

We estimate total static settlement for shallow footing foundations using the recommended maximum net allowable bearing pressure presented above, should be less than one inch. Static differential settlements are estimated to be about one-half (½) the total settlement. These settlement estimates are based on the available boring information, our experience with similar structures and soil conditions, and field verification of suitable bearing soils by our firm during foundation construction.

Drilled Piers

Drilled, cast-in-place piers (drilled piers) may be used to support the proposed structures. Drilled piers should be at least 18 inches in diameter and extend to at least five feet below the existing ground surface. Piers so established may be designed based on an allowable end bearing capacity of 3000 psf for dead plus live loads. We recommend that adjacent piers be constructed no closer than three pier diameters apart, as measured between centers of the piers. Drilled pier foundations should be structurally isolated from any adjacent concrete flatwork by a felt strip or similar material.

Uplift resistance of the pier foundations may be computed assuming the following resisting forces, where applicable: 1) the unit weight of foundation concrete (150 pound per cubic foot); and, 2) uplift resistance of 300 psf applied over the shaft area of the pier. Increased uplift resistance can be achieved by increasing the diameter of the pier or increasing the depth of the embedment depth.

Sizing of drilled piers to resist lateral loads can be evaluated using Section 1807A.3.2 of the 2016 CBC. An allowable value of 300 pcf for lateral bearing as defined in Table 1806A.2 of the CBC may be used for the coefficients S_1 and S_3 for the nonconstrained and constrained conditions, respectively. Per Section 1806A.1 of the 2016 CBC, an increase of 1/3 is permitted when using the alternate load combinations in Section 1605A.3.2 that include wind or earthquake loads. The upper 12 inches of the subgrade should be neglected unless the drilled pier is surrounded by at least three feet of concrete on all sides.

Lateral resistance of drilled piers can also be evaluated by determining the shear, moment and deflection of the pier using a computer model of the pier and soil (i.e. LPILE). Such an analysis is beyond the current scope of this evaluation and can be accomplished after the dimensions of the piers and loading conditions are known, if desired.

The bottom of the pier excavations should be free of loose or disturbed soils prior to placement of the concrete. Cleaning of the bearing surface may be done mechanically with the belling bucket, but should be verified by the geotechnical engineer prior to concrete placement.

Reinforcement and concrete should be placed in the pier excavations as soon as possible after excavation is completed to reduce the potential of sidewall caving into the excavations.

To reduce lateral movement of the drilled shafts, it is necessary to place the concrete for the drilled shafts in intimate contact with the surrounding soil. Any voids or enlargements in the shafts due to over-excavation or temporary casing installation shall be filled with concrete at the time the shaft concrete is placed.

If the drilled piers are constructed in the "dry" (with dry being less than two inches of water at the base of the excavation), the concrete may be placed by the free-fall method, using a short hopper or back-chute to direct the concrete flow out of the truck into a vertical stream of flowing concrete with a relatively small diameter. The stream should be directed to avoid hitting the sides of the excavation or any reinforcing cages. For the free-fall method of concrete placement, we recommend the concrete mix be designed with a slump of five to seven inches.

In general, we anticipate the drilled pier excavations will be relatively dry for pier excavations. However, perched groundwater may be encountered depending on the time of year when the piers are excavated. Where perched groundwater will not be controlled such that more than six inches of water accumulates at the bottom of the pier excavation and after it is confirmed that the excess water cannot be removed from the caisson excavation by bailing or with pumps, concrete should be placed using a tremie. For concrete placed using the tremie method, a slump of six to eight inches, and a maximum aggregate size of ¾-inch is recommended. The required slump should be obtained by using plasticizers or water-reducing agents. Addition of water on-site to establish the recommended slump should not be allowed.

When extracting temporary casings or tremie methods from the excavation, care should be taken to maintain a head of concrete to prevent infiltration of water and soil into the shaft area. The head of concrete should always be greater than the head of water trapped outside the pier or tremie, taking into account the differences in unit weights of concrete and water.

We estimate total settlement for drilled pier foundations using the recommended maximum net allowable bearing pressure and allowable capacities presented above, will be less than one inch. Differential settlements may be as much as the total settlement between individual pier elements. The settlement estimates are based on the available soil information, our experience with similar structures and soil conditions, and field verification of suitable bearing soils during foundation construction.

Interior Floor Slab Support

Interior concrete slab-on-grade floors can be supported upon the low-expansive soil subgrade (either non-expansive imported materials and/or lime treated native soils) prepared in accordance with the recommendations in this report and maintained in that condition (at least two percent above the optimum moisture content) and are protected from disturbance.

Interior concrete slab-on-grade floors should be at least five inches thick and be reinforced for crack control. Final slab thickness, reinforcement and joint spacing should be determined by the slab designer. Proper and consistent location of the reinforcement near mid-slab is essential to its performance. The risk of uncontrolled shrinkage cracking is increased if the reinforcement is not properly located within the slab. Temporary loads exerted during construction from vehicle traffic, cranes, construction equipment, storage of palletized construction materials, etc. should be considered in the design of the thickness and reinforcement of the interior slab.

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 crushed rock or gravel, serving as a deterrent to migration of capillary moisture. The gravel/crushed rock layer should be between four and six inches thick and graded such that 100 percent passes a one-inch sieve and no appreciable amount 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 over the past 30 years or more has included placement of a thin layer of sand over the vapor retarder membrane where capillary break gravel is used. 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. Therefore, we consider the use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

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

Floor Slab Moisture Penetration Resistance

It is considered likely that floor slab subgrade soils will become wet to near saturated at some time during the life of structures. This is a certainty when slabs are constructed during the wet seasons, or when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that interior slabs intended for moisture-sensitive floor coverings or materials, require protection against moisture or moisture vapor penetration. Standard practice includes the gravel/crushed rock and vapor retarder as suggested above. However, the gravel/crushed rock and plastic membrane offer only a limited, first line of defense against soil-related moisture; they do not moisture-proof the slab. 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 does it assure that slab moisture transmission levels will be low enough to prevent damage to floor coverings or other building components. If increased protection against moisture vapor penetration of slabs is desired, a concrete moisture protection specialist should be consulted. The design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

Exterior Flatwork Construction

Soil subgrade areas to support exterior concrete flatwork should be prepared in accordance with the Subgrade Preparation and Engineered Fill Construction recommendations included in this report (i.e., at least 12 inches of non-expansive fill and/or lime treated subgrade). Exterior flatwork subgrade soils should be maintained in a moist condition (at least two percent above the optimum moisture content) and protected from disturbance. If this is not the case and subgrade soils become dry and/or disturbed, the exterior flatwork subgrade will require additional scarification, moisture conditioning and compaction prior to construction of the exterior flatwork. Exterior flatwork should be underlain by at least four inches of aggregate base compacted to at least 90 percent relative compaction. The aggregate base should be placed over the 12 inches of low-expansion potential imported soil, or lime treated soil. The additional four inches of aggregate base is not required if the low-expansion imported fill below the flatwork consists of aggregate base.

Flatwork should be at least four inches thick and reinforced for crack control. The architect or civil engineer should determine the final thickness, strength, reinforcement, and joint spacing of exterior slab-on-grade concrete. Exterior flatwork next to landscaped areas should be thickened to twice the slab thickness for a width of at least 12 inches to help support lawn mowing equipment and other maintenance equipment. Accurate and consistent location of the reinforcement at mid-slab is essential to its performance and the risk of uncontrolled drying shrinkage slab cracking is increased if the reinforcement is not properly located within the slab.

Subgrade soils in areas to receive exterior concrete flatwork (e.g., sidewalks) should be uniformly moisture conditioned to at least two percent above the optimum moisture content, and compacted to at least 90 percent relative compaction prior to the placement of the concrete. *Proper moisture conditioning of the subgrade soils is considered essential to the performance of exterior flatwork.* Uniform moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks.

Uniform moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks. Practices recommended by the Portland Cement Association (PCA) for proper placement and curing of concrete should be followed during exterior concrete flatwork construction. Flatwork should be independent of the building foundations and felt strips should be used to separate concrete slabs from building foundations.

Exterior flatwork that will be traversed by vehicles or heavy equipment should be designed in accordance with the recommendations provided in the Pavement Design section of this report.

Pavement Design

We are providing several alternative pavement alternatives based on the soil conditions encountered at the site, the results of laboratory testing previously obtained at the site, and our experience.

The procedures used to design the pavement sections are in general conformance with the California Highway Design Manual, Sixth Edition. Laboratory testing of the on-site soils indicates R-values of 5 were obtained on the near-surface soils at the site. An R-value of 5 was therefore used in our pavement design. If the site is chemically amended as noted above, an R-value of 50 is considered appropriate for design of on-site pavements.

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 5			Lime-Treated Subgrades Soils (a) R-value = 50		
		Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)	Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Track Surface / Light Automobile Parking	2½*	10	--	2½*	4	--
		--	6	4	--	4	4
6.5	Emergency Vehicle Traffic	3	16	--	3	6	--
		4*	13	--	4*	4	--
		--	6	6	--	6	4

* = Asphalt concrete thickness contains the Caltrans safety factor.

Pavement design alternates for chemically amended soil are based upon at least 12 inches of the compacted pavement subgrade soils being chemically amended with high-calcium or dolomitic quicklime. Based on our experience, the native soils should be mixed with at least 4½ pounds per compacted cubic foot of lime mixed to a depth sufficient to produce a 12-inch thick compacted layer. Lime treated soils should be compacted at least two percent over the optimum moisture content to at least 95 percent relative compaction.

We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, including utility trench backfill within the limits of the pavements. The upper six inches of untreated pavement subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should be compacted to at least 95 percent of the maximum dry density. Class 2 aggregate base should conform to Section 26 of the Caltrans Standard Specifications.

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

We suggest that concrete slabs be constructed with thickened edges at least two inches plus the slab thickness and 36 inches wide in accordance with ACI design standards and reinforced for crack control, if desired. Reinforcement must be located at mid-slab depth to be effective. Portland cement concrete should achieve a minimum compressive strength of 3500 pounds per square inch (psi) at 28 days. Concrete curing and joint spacing and details should conform to current PCA and ACI guidelines.

We suggest considering the use of full depth curbs where pavements abut landscaping and along the outer edge of the track. The curbs should extend to at least six inches into the surface of the soil subgrade or lime treated soils, if used. Weep holes also could be provided at storm drain drop inlets, located at the subgrade-base interface, to allow water to drain from beneath the pavements.

Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from the track and buildings and prevent ponding of water adjacent to foundations or slabs.

Subgrades adjacent to buildings 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 PVC pipes which are connected to available drainage features to convey water away from the structures, or discharging the drains onto paved, or hard surfaces that slope away from the foundations. Discharging or ponding of surface water should not be allowed adjacent to buildings, exterior flatwork or onto slope surfaces. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward buildings.

Geotechnical Engineering Observation and Testing During Construction

Site preparation should be accomplished in accordance with the recommendations of this report and the *Guide Earthwork Specifications* provided in Appendix C. Geotechnical testing and observation during construction is considered a continuation of our geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site clearing, 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 2016 CBC requires that the geotechnical engineering report provide a number and frequency of field compaction tests to determine compliance with the recommended minimum compaction. Many factors can affect the number of tests that should be performed during 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.

If 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

We recommend that our firm be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents. We would be pleased to submit a proposal to provide these services upon request.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed construction, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used prudent engineering and geologic judgment based upon the information provided and the data generated from our investigation. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

If the proposed construction is modified or 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

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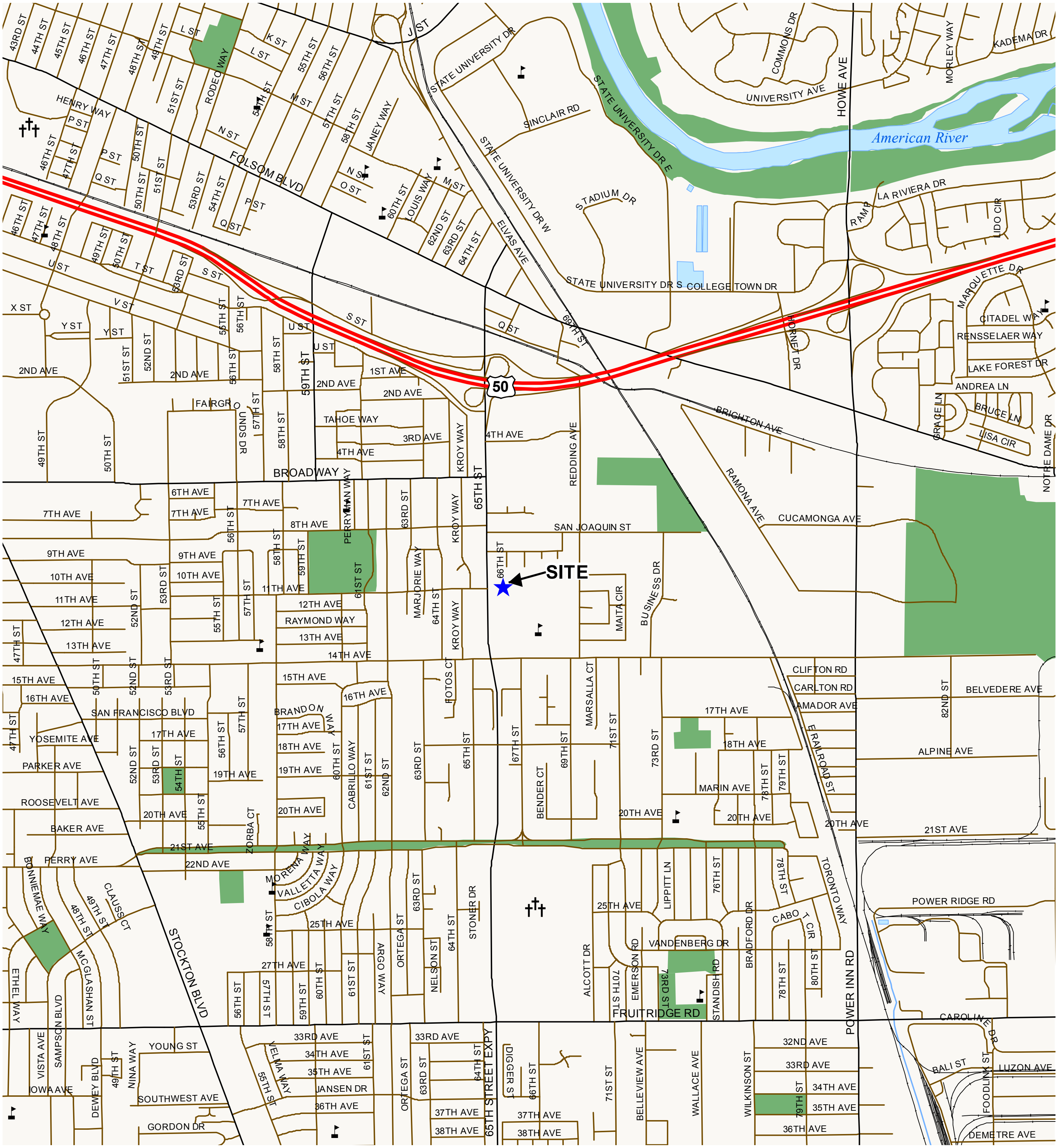


Street data courtesy of Sacramento County.
Hydrography courtesy of the U.S. Geological Survey
acquired from the GIS Data Depot, December, 2007.
Projection: NAD 83, California State Plane, Zone II

N

0 1,000 2,000
Feet

WallaceKuhl & ASSOCIATES

VICINITY MAP

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	

Aerial provided by ESRI.
Projection: NAD 83, California State Plane, Zone II

Legend

Approximate Site Boundary
Approximate Soil Boring Location
Cross-section Line

N
0 50 100
Feet

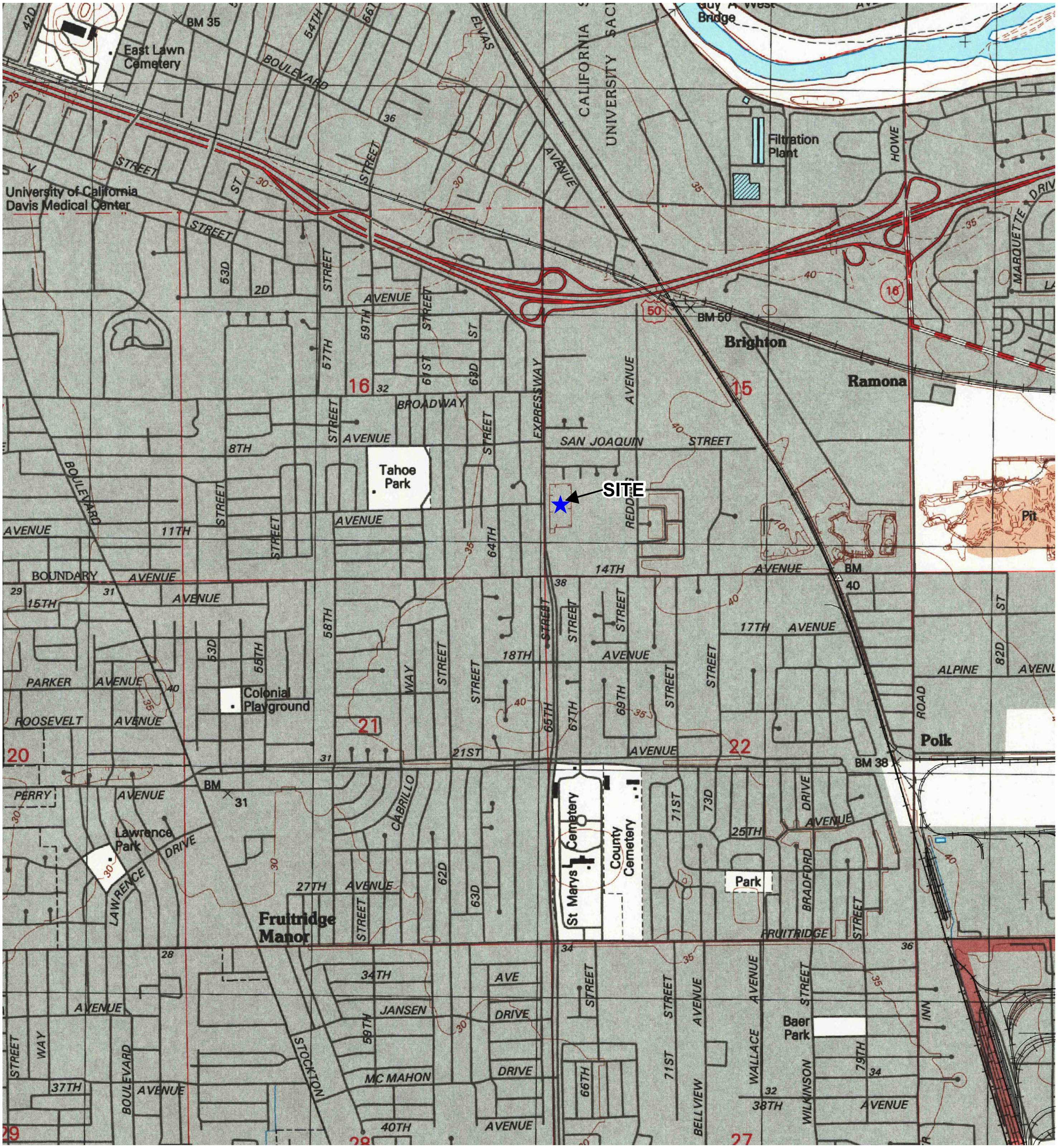
WallaceKuhl & ASSOCIATES

SITE PLAN

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	



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

N

0 1,000 2,000
Feet

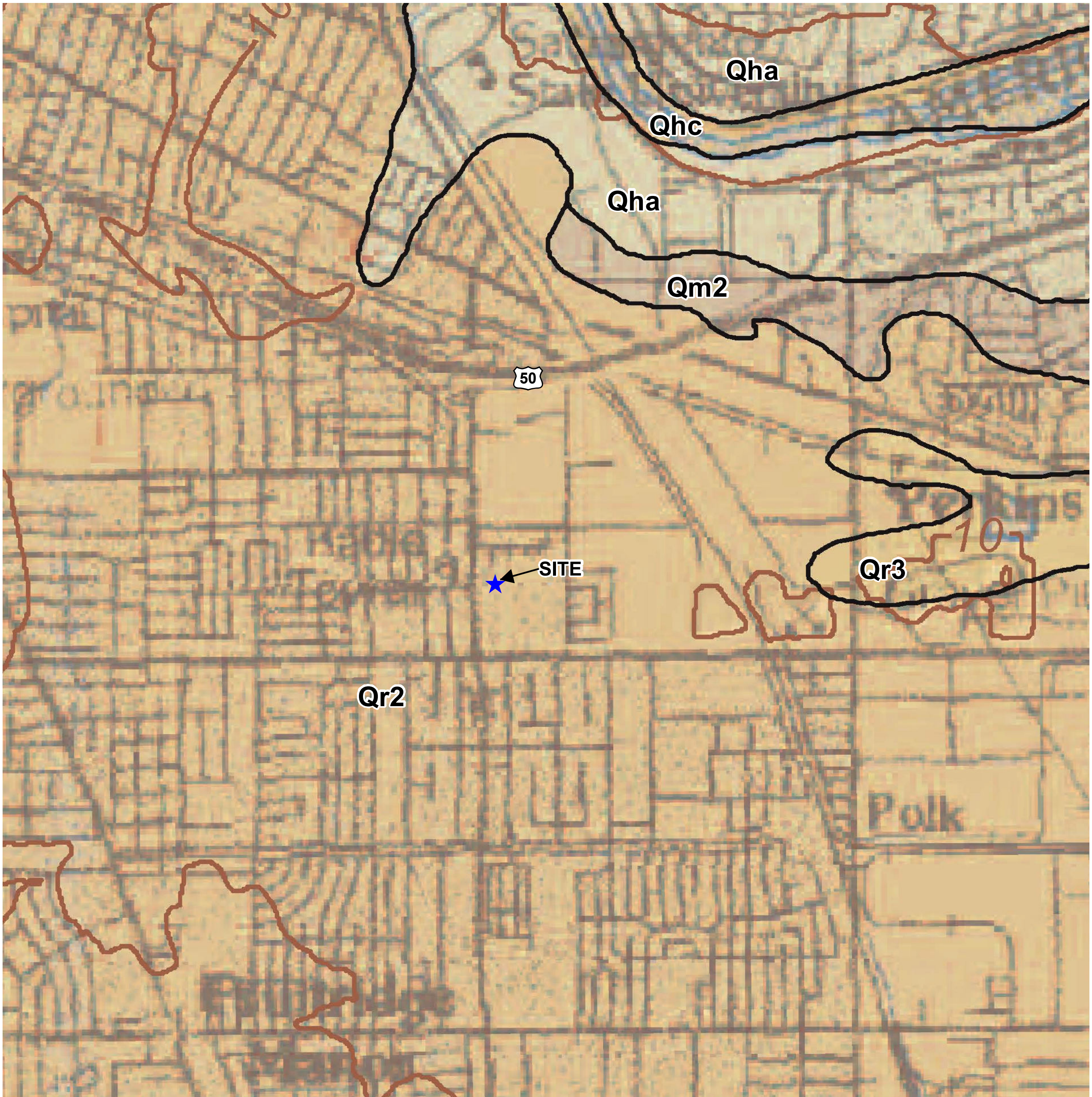
WallaceKuhl & ASSOCIATES

USGS TOPOGRAPHIC MAP

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	3
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	



Geologic Map adapted from the Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle, California, dated 2011. Compilation by Carlos I. Gutierrez. Projection: NAD 83, California State Plane, Zone II

Legend

- Geologic Contact
- **Qhc** Latest Holocene stream channel deposits
- **Qm2** Upper member of the ModestoFormation
- **Qr3** Upper unit of the Riverbank Formation
- **Qr2** Middle unit of the Riverbank Formation
- **Qha** Holocene alluvium

N

0 1,000 2,000 Feet

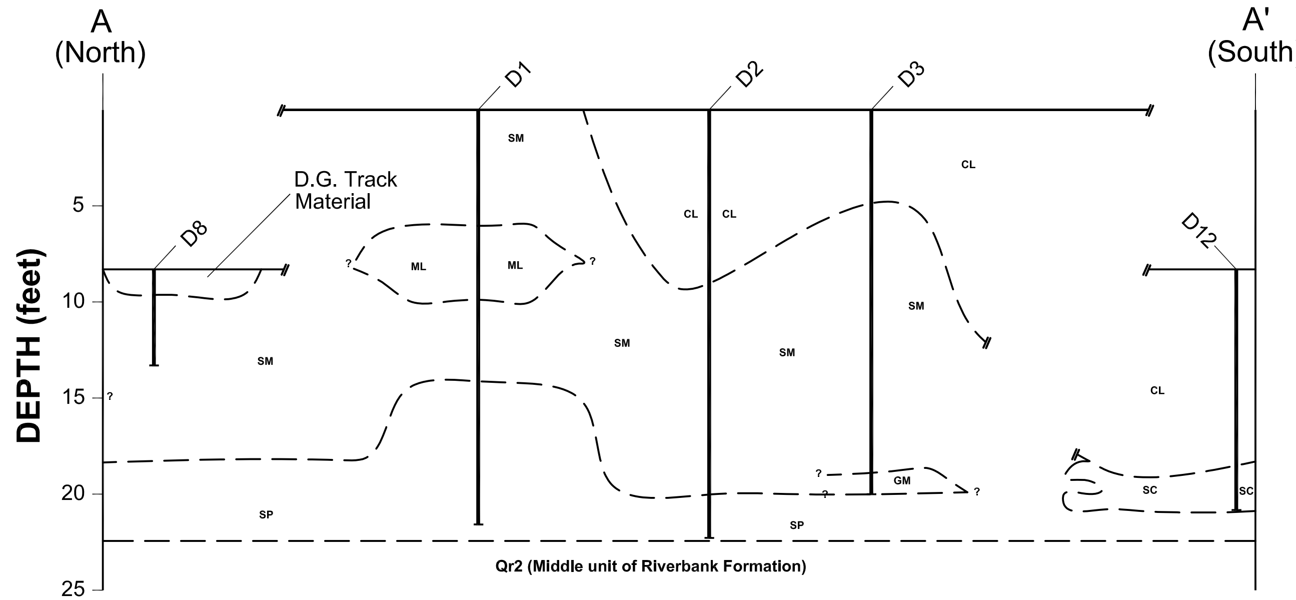
WallaceKuhl & ASSOCIATES

GEOLOGIC MAP

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	4
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	



Key:

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

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

(XX) * Detailed soil logs and descriptions available in Boring Logs, Figures 8 through 19.

* Elevations, Depths, and Locations are approximate.

Note:

Borings D8 and D12 were performed at an elevation of approximately 8 feet below borings D1, D2, and D3.

Scale
Horizontal: 1" = 100'
Vertical: 1" = 5'

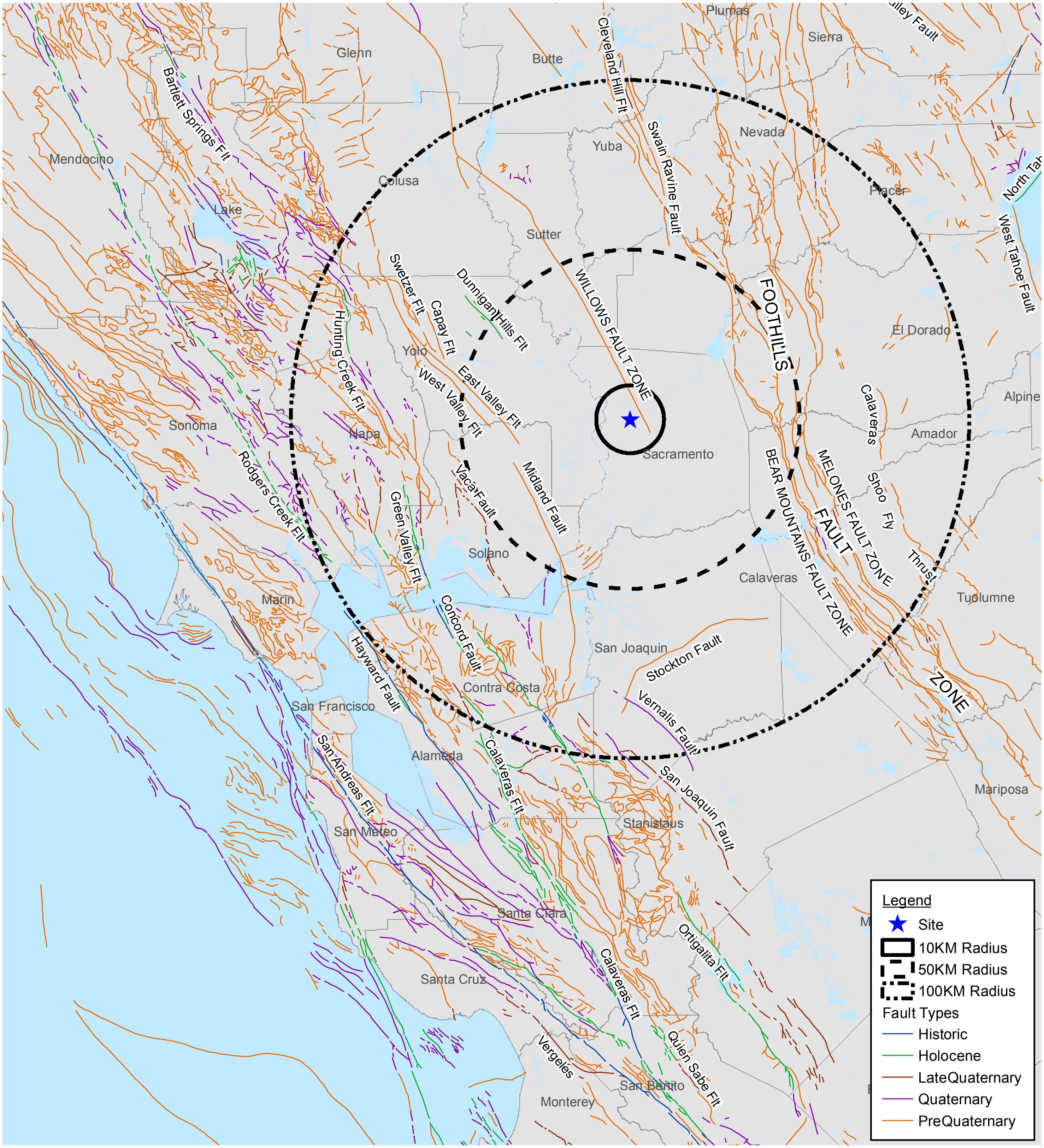
WallaceKuhl & ASSOCIATES

GEOLOGIC CROSS SECTION A-A'

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	5
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO.11843.01P	



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

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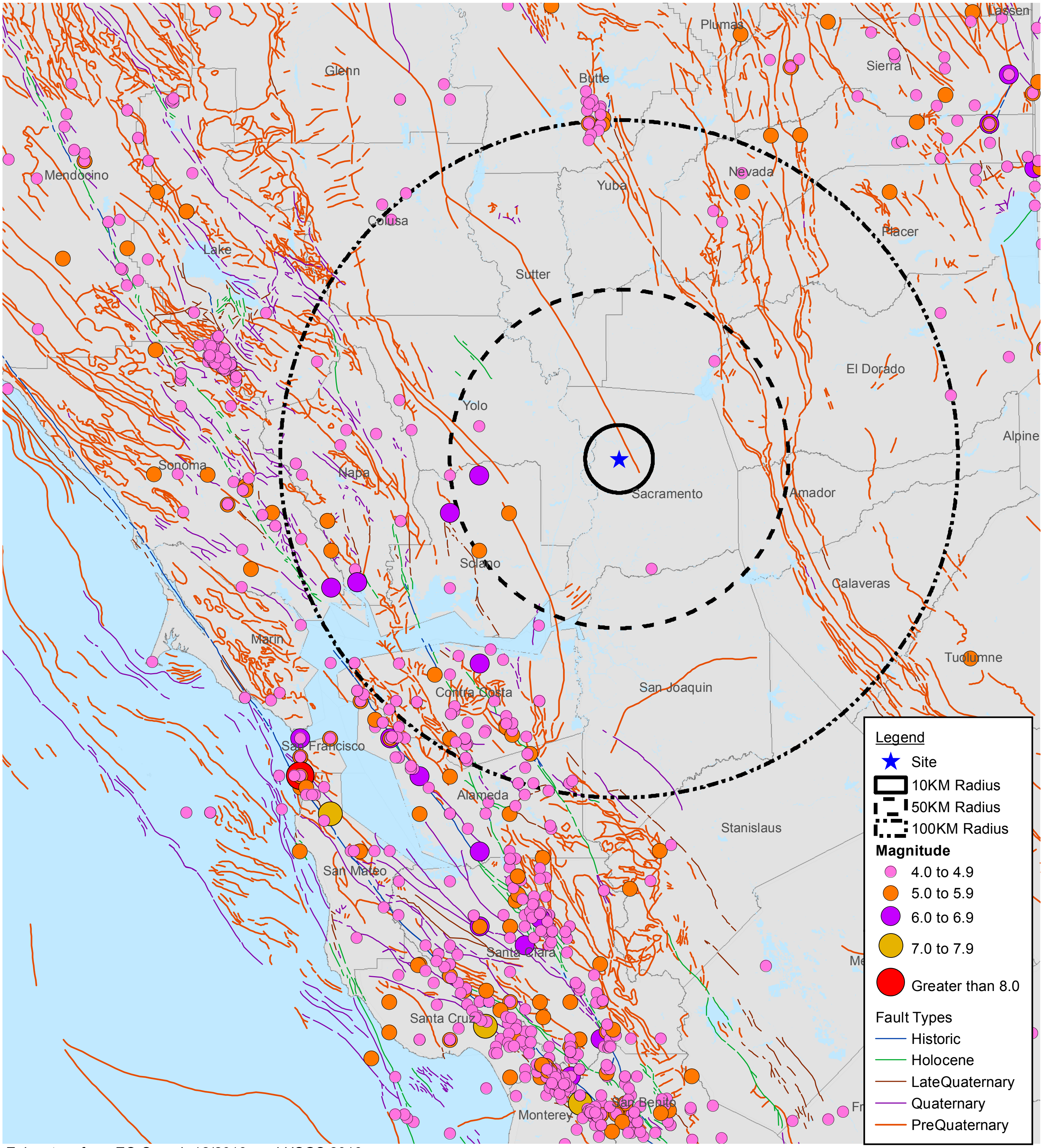
FAULT MAP

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	6
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	



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

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EPICENTER MAP

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	7
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18

WKA NO. 11843.01P

Project: Hiram Johnson High School Track and Football Field Project Location: Sacramento, California WKA Number: 11843.01P	**LOG OF SOIL BORING D1** Sheet 1 of 1

Date(s) Drilled	**4/3/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**21.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks				Driving Method and Drop	**140-lb automatic 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
	0		Dark brown to brown, moist, loose, silty SAND (SM)						
	1.5			■	D1-1I	5	18.1	112	TR
	5		Dark brown to brown, moist, very dense, variably cemented sandy SILT (ML)	■	D1-2I	81	15.2	107	
	10		Dark brown to brown, moist, medium dense, silty fine SAND (SM)	■	D1-3I	21	18.9	91	
	15		Light brown, moist, dense, SAND (SP)	■	D1-4I	34	8.4	92	
	20			■	D1-4I	43			
			Boring terminated at 21.5 feet below existing site grade Groundwater was not encountered						

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FIGURE 8

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D2

Date(s) Drilled	**4/3/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**22.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks	**Bulk (0-3)**			Driving Method and Drop	**140-lb automatic 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
	0		Brown, moist, very stiff, slightly sandy lean CLAY (CL)						
					D2-1I	19			
	5				D2-2I	31	17.9		
	10		Light brown, moist, medium dense, silty fine SAND (SM)		D2-3I	22	21.1		
	15				D2-4I	20	23.2		
	20		Gray to light brown, moist, medium dense, sandy GRAVEL (GP)		D2-5I	17			
			Gray to light brown, moist, medium dense, silty SAND (SM) with gravel (Qr2, middle unit of Riverbank formation)						

Boring terminated due to practical refusal at 22.5 feet below existing site grade
Groundwater was not encountered

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FIGURE 9

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D3

Sheet 1 of 1

Date(s) Drilled	**4/3/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**20.0 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**
Remarks	**Bulk (0-3)**			Driving Method and Drop	**140-lb automatic 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
	0		Dark brown to brown, moist, very stiff, sandy lean CLAY (CL)						
	4.5			■	D3-1I	22	17.7	108	PI
	5		Dark brown to brown, moist, medium dense, clayey silty SAND (SM)						
	9.5			■	D3-2I	17	23.6	93	
	14		no clay						
	14.5			■	D3-3I	18	17.6	104	
	18.5		Gray to brown, moist, dense, silty sandy GRAVEL (GM)	■	D3-4I	63	5.6	118	
	20		Boring terminated at 20 feet below existing site grade Groundwater was not encountered						

BORING LOG 11843.01P - HIRAM JOHNSON HS.GPJ WKA.GDT 4/16/18 2:57 PM

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FIGURE 10

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D4

Date(s) Drilled	**4/3/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**20.0 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**
Remarks	**Bulk (0-3)**			Driving Method and Drop	**140-lb automatic 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
	0		Dark brown to brown, moist, hard, sandy lean CLAY (CL)						
	5			■	D4-1I	49	13.6	112	
	8.5		Dark brown to brown, moist, medium dense, silty SAND (SM)						
	10			■	D4-2I	14	18.9	90	
	15			■	D4-3I	16	23.1	81	
	20		gray to brown	■	D4-4I	27	8.5	97	

Boring terminated at 20 feet below existing site grade
Groundwater was not encountered

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FIGURE 11

Project: Hiram Johnson High School Track and Football Field **Project Location: Sacramento, California** **WKA Number: 11843.01P**	**LOG OF SOIL BORING D5** Sheet 1 of 1

Date(s) Drilled	**4/3/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**21.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks	**Bulk (0-3)**			Driving Method and Drop	**140-lb automatic 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
	0		Dark brown to brown, moist, stiff, silty CLAY (CL)						
				■	D5-1I	6	17.9	107	PI
	5		Dark brown to brown, moist, medium dense, clayey silty SAND (SM)	■	D5-2I	20	23.9	95	
	10		loose, no clay	■	D5-3I	10	19.6	86	
	15		medium dense	■	D5-4I	20	21.1	96	
	20		variably cemented	■	D5-5I	26			
			Boring terminated at 21.5 feet below existing site grade Groundwater was not encountered						

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FIGURE 12

Project: Hiram Johnson High School Track and Football Field **Project Location: Sacramento, California** **WKA Number: 11843.01P**	**LOG OF SOIL BORING D6** Sheet 1 of 1

Date(s) Drilled	**3/23/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**WKA**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**Giddings**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks	**Bulk (1-3)**			Driving Method and Drop	**70-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
	0		Dark brown to brown, moist, stiff, clayey SILT (ML)						
	1			■	D6-1I	10	24.9	94	PI
	5		Dark brown to brown, moist, medium dense, clayey silty fine SAND (SM)	■	D6-2I	22	18.4	79	
	10		light brown, no clay	■	D6-3I	30	17.8	77	
	15			■	D6-4I	33	16.8	90	
	16.5		Boring terminated at 16.5 feet below existing site grade Groundwater was not encountered						

BORING LOG 11843.01P - HIRAM JOHNSON HS.GPJ WKA.GDT 4/16/18 2:57 PM

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FIGURE 13

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D7

Sheet 1 of 1

Date(s) Drilled	**3/23/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**WKA**	Total Depth of Drill Hole	**5.0 feet**
Drill Rig Type	**Giddings**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks	**Bulk (1-3)**	Driving Method and Drop	**70-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
			Light brown to brown, moist, sandy CLAY (CL)						
	5		Boring terminated at 5.0 feet below existing site grade Groundwater was not encountered						

BORING LOG 11843.01P - HIRAM JOHNSON HS.GPJ WKA.GDT 4/16/18 2:57 PM

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FIGURE 14

Project: Hiram Johnson High School Track and Football Field	LOG OF SOIL BORING D8
Project Location: Sacramento, California	
WKA Number: 11843.01P	Sheet 1 of 1

Date(s) Drilled	3/23/18	Logged By	JRY	Checked By	MSM
Drilling Method	Solid Stem Auger	Drilling Contractor	WKA	Total Depth of Drill Hole	5.0 feet
Drill Rig Type	Giddings	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	Modified California	Drill Hole Backfill	Soil
Remarks	Bulk (0-3)	Driving Method and Drop	70-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
			D.G. track material						
			Brown, moist, silty SAND (SM)						
	5								
			Boring terminated at 5.0 feet below existing site grade Groundwater was not encountered						

BORING LOG 11843.01P - HIRAM JOHNSON HS.GPJ WKA.GDT 4/16/18 2:57 PM

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FIGURE 15

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D9

Date(s) Drilled	3/23/18	Logged By	JRY	Checked By	MSM
Drilling Method	Solid Stem Auger	Drilling Contractor	WKA	Total Depth of Drill Hole	5.0 feet
Drill Rig Type	Giddings	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	Modified California	Drill Hole Backfill	Soil
Remarks	Bulk (0-3)	Driving Method and Drop	70-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
	0–5		Dark brown to brown, moist, slightly sandy CLAY (CL)						
	5		Boring terminated at 5.0 feet below existing site grade Groundwater was not encountered						

BORING LOG 11843.01P - HIRAM JOHNSON HS.GPJ WKA.GDT 4/16/18 2:57 PM

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FIGURE 16

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D10

Date(s) Drilled	3/23/18	Logged By	JRY	Checked By	MSM
Drilling Method	Solid Stem Auger	Drilling Contractor	WKA	Total Depth of Drill Hole	5.0 feet
Drill Rig Type	Giddings	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	Modified California	Drill Hole Backfill	Soil
Remarks	Bulk (0-3)	Driving Method and Drop	70-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 to brown, moist, clayey SAND (SC)						
			Brown, moist, silty SAND (SM)						
	5								
			Boring terminated at 5.0 feet below existing site grade Groundwater was not encountered						

BORING LOG 11843.01P - HIRAM JOHNSON HS.GPJ WKA.GDT 4/16/18 2:57 PM

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FIGURE 17

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D11

Date(s) Drilled	**3/23/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**WKA**	Total Depth of Drill Hole	**5.0 feet**
Drill Rig Type	**Giddings**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks	**Bulk (0-3)**			Driving Method and Drop	**70-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 to brown, moist, clayey silty SAND (SM)						
	5		Boring terminated at 5.0 feet below existing site grade Groundwater was not encountered						

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FIGURE 18

Project: Hiram Johnson High School Track and Football Field
Project Location: Sacramento, California
WKA Number: 11843.01P

LOG OF SOIL BORING D12

Date(s) Drilled	**3/23/18**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Stem Auger**	Drilling Contractor	**WKA**	Total Depth of Drill Hole	**13.0 feet**
Drill Rig Type	**Giddings**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet		Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Soil**
Remarks	**Bulk (0-3), RV (0-3'), EI (0-3')**	Driving Method and Drop	**70-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
	0		Dark brown to brown, moist, stiff, sandy lean CLAY (CL)						
	1			■	D12-1I	20	19.6	100	
	5			■	D12-2I	20	24.5	94	
	10		Brown, moist, medium dense, clayey SAND (SC)	■	D12-3I	24	21.3	97	
	12		Brown, moist, medium dense, silty SAND (SM) with gravel (Qr2, middle unit of Riverbank formation)						

Boring terminated due to practical refusal at 13 feet below existing site grade
Groundwater was not encountered

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FIGURE 19

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISIONS		USCS[4]	CODE	CHARACTERISTICS
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS[1] (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well-graded gravels or gravel - sand mixtures, trace or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, trace or no fines
		GM		Silty gravels, gravel - sand - silt mixtures, containing little to some fines[2]
		GC		Clayey gravels, gravel - sand - clay mixtures, containing little to some fines[2]
	SANDS[1] (50% or more of coarse fraction < no. 4 sieve size)	SW		Well-graded sands or sand - gravel mixtures, trace or no fines
		SP		Poorly graded sands or sand - gravel mixtures, trace or no fines
		SM		Silty sands, sand - gravel - silt mixtures, containing little to some fines[2]
		SC		Clayey sands, sand - gravel - clay mixtures, containing little to some fines[2]
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts, gravely silts, and sandy silts that are non-plastic or with low plasticity
		CL		Inorganic lean clays, gravelly lean clays, sandy lean clays of low to medium plasticity[3]
		OL		Organic silts, organic lean clays, and organic silty clays
	SILTS & CLAYS LL ≥ 50	MH		Inorganic elastic silts, gravelly elastic silts, and sandy elastic silts
		CH		Inorganic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
		OH		Organic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT		Peat
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

CR = Corrosion
PI = Plasticity Index
EI = Expansion Index
UCC = Unconfined Compression Test (TSF)
TR = Triaxial Compression Test
GR = Gradational Analysis (Sieve/Hydro)
FC = Wash (Fines Content)
PP = Pocket Penetrometer Test (TSF)
PID = Photo Ionization Detector Test (PPM)
RV = Resistance ("R") Value

REF = Refusal (>50 blows in 6 inches)

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS (b)	Above 12"	Above 300
COBBLES (c)	12" to 3"	300 to 75
GRAVEL (g) coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	75 to 4.75 75 to 19 19 to 4.75
SAND coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.75 to 0.075 4.75 to 2.00 2.00 to 0.425 0.425 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

Trace - Less than 5 percent
Few - 5 to 10 percent
Little - 15 to 25 percent
Some - 35 to 45 percent
Mostly - 50 to 100 percent

* Percents as given in ASTM D2488

NOTES:

1. Coarse grained soils containing 5% to 12% fines, use dual classification symbol (ex. SP-SM).
2. If fines classify as CL-ML (4<PI<7), use dual symbol (ex. SC-SM).
3. Silty Clays, use dual symbol (CL-ML).
4. Borderline soils with uncertain classification list both classifications (ex. CL/ML).

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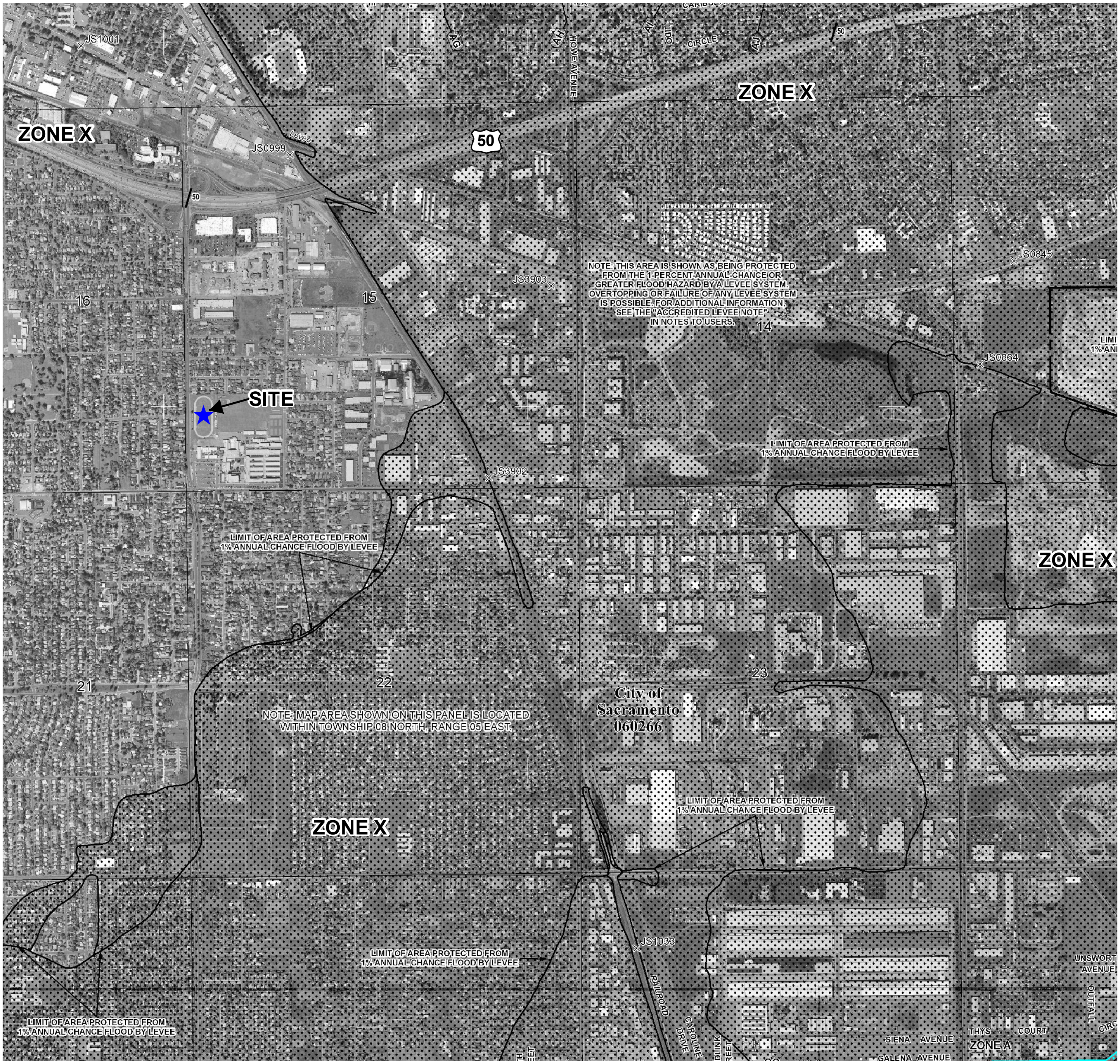
UNIFIED SOIL CLASSIFICATION SYSTEM

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	20
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	



Flood Maps provided by FEMA, panel 195 of 705.
map number 06067C0195H dated August, 2012..
Projection: NAD 83, California State Plane, Zone II

Legend

ZONE X Area of minimum flood hazard; area with reduced flood risk due to levee

N

0 1,000 2,000
Feet

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FEMA FLOOD MAP

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	21
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	

APPENDICES

APPENDIX A
General Project Information, Field and Laboratory Test Results

APPENDIX A
WKA No. 11843.01P

A. GENERAL INFORMATION

The performance of a geotechnical engineering and geologic hazards study for the proposed Hiram Johnson High School Track and Football Field Project located northwesterly of the intersection of 14th Avenue and 65th Street in Sacramento, California, was verbally authorized by Mr. Mark Baganski on March 6, 2018. Authorization was for a geotechnical study as described in our proposal letter dated November 22, 2017 and sent to our client, Verde Design, Inc., whose mailing address is 1024 Iron Point Road, Suite 100 #1241 in Folsom, California 95630; telephone (916) 415-6550.

B. FIELD EXPLORATIONS

As part of our study for the proposed improvements, our field exploration included the drilling and sampling of 12 borings (D1 through D12) at the approximate locations shown on Figure 2.

The borings were drilled on March 23, 2018 and April 3, 2018. On March 23, 2018, seven borings (D6 through D12) were drilled at the site utilizing a John Deere 6x6 Gator-mounted drilling equipment equipped with six-inch-diameter, solid-stem helical augers to depths of five feet below existing site grades at the locations shown in Figure 2.

On April 3, 2018, five borings (D1 through D5) were drilled at the site using a CME-75 truck-mounted drill rig equipped with six-inch-diameter, solid flight augers to depths ranging from about 15 to 22½ feet below existing site grades at the locations shown in Figure 2. 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 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 six-inch interval were recorded. The sum of the blows required to drive the sampler the lower 12-inch interval is designated the penetration resistance or "blow count" for that particular drive. The modified California samples were retained in 2-inch diameter by 6-inch long, thin walled brass tubes contained within the sampler. The SPT samples were retained in plastic zip-lock bags. After recovery, the field representative visually classified the soil recovered in the tubes and plastic bags. After the samples were classified, the 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 soil were collected and retained in plastic bags at the locations, shown in Figure 2. 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 19. A Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained in Figure 20.

C. LABORATORY TESTING

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

One sample of the near-surface soil was tested for triaxial shear strength (ASTM D4767) with results presented in Figure A1.

Three samples of near-surface clay soil were tested to determine the Atterberg Limits (ASTM D4318). The test results are presented in Figure A2.

Two representative samples of near-surface soils were tested for grain-size distribution (ASTM C136) and hydrometer analysis (ASTM D422). The results of the gradation tests are contained in Figure A3.

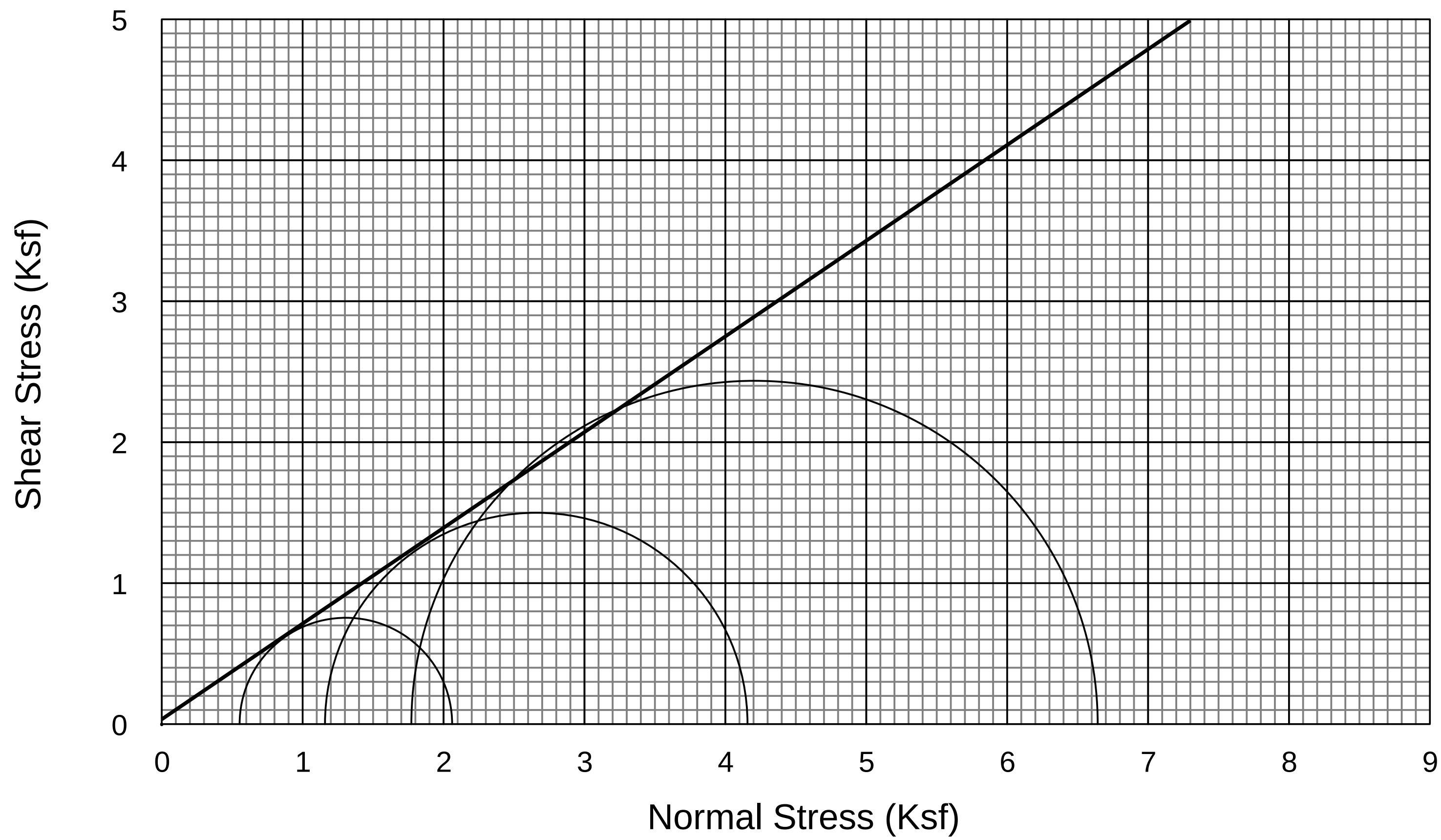
One sample of the near-surface clay was tested for Expansion Index (ASTM D4829). The test results are presented in Figures A4.

One bulk samples of anticipated pavement subgrade soil was subjected to Resistance-value ("R-value") testing in accordance with California Test 301. The results of the R-value test, which was used in the pavement design, is presented in Figure A5.

Two samples of near-surface soil were submitted to Sunland Analytical for corrosivity testing in accordance with California Test (CT) No. 643 (Modified Small Cell), CT 417, CT 422, and ASTM D-516. Copies of the analytical results are presented in Figures A6 through A9.

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. : D1-1I

SAMPLE CONDITION : Undisturbed

SAMPLE DESCRIPTION : Brown, silty sand

DRY DENSITY (PCF) : 112
INITIAL MOISTURE (%) : 17.0
FINAL MOISTURE (%) : 18.1

ANGLE OF INTERNAL FRICTION (ø) : 34°
COHESION (PSF) : 27

WallaceKuhl & ASSOCIATES

TRIAXIAL COMPRESSION TEST RESULTS

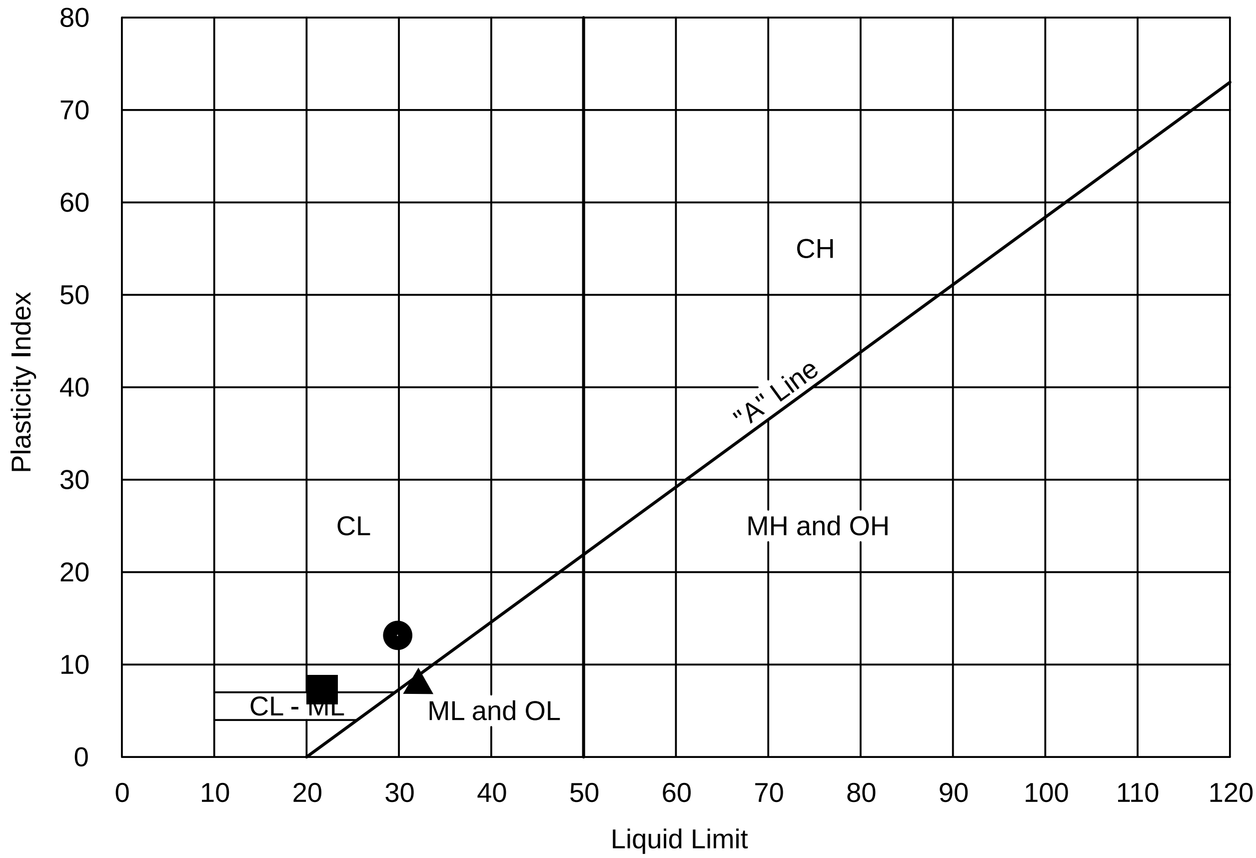
HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	

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 (%)		
●	D3-1I	3.5'-4.0'	---	30	13	---	CL
■	D5-1I	1.0'-1.5'	---	23	7	---	CL
▲	D6-1I	1.0'-1.5'	---	32	8	---	ML

Wallace Kuhl & ASSOCIATES

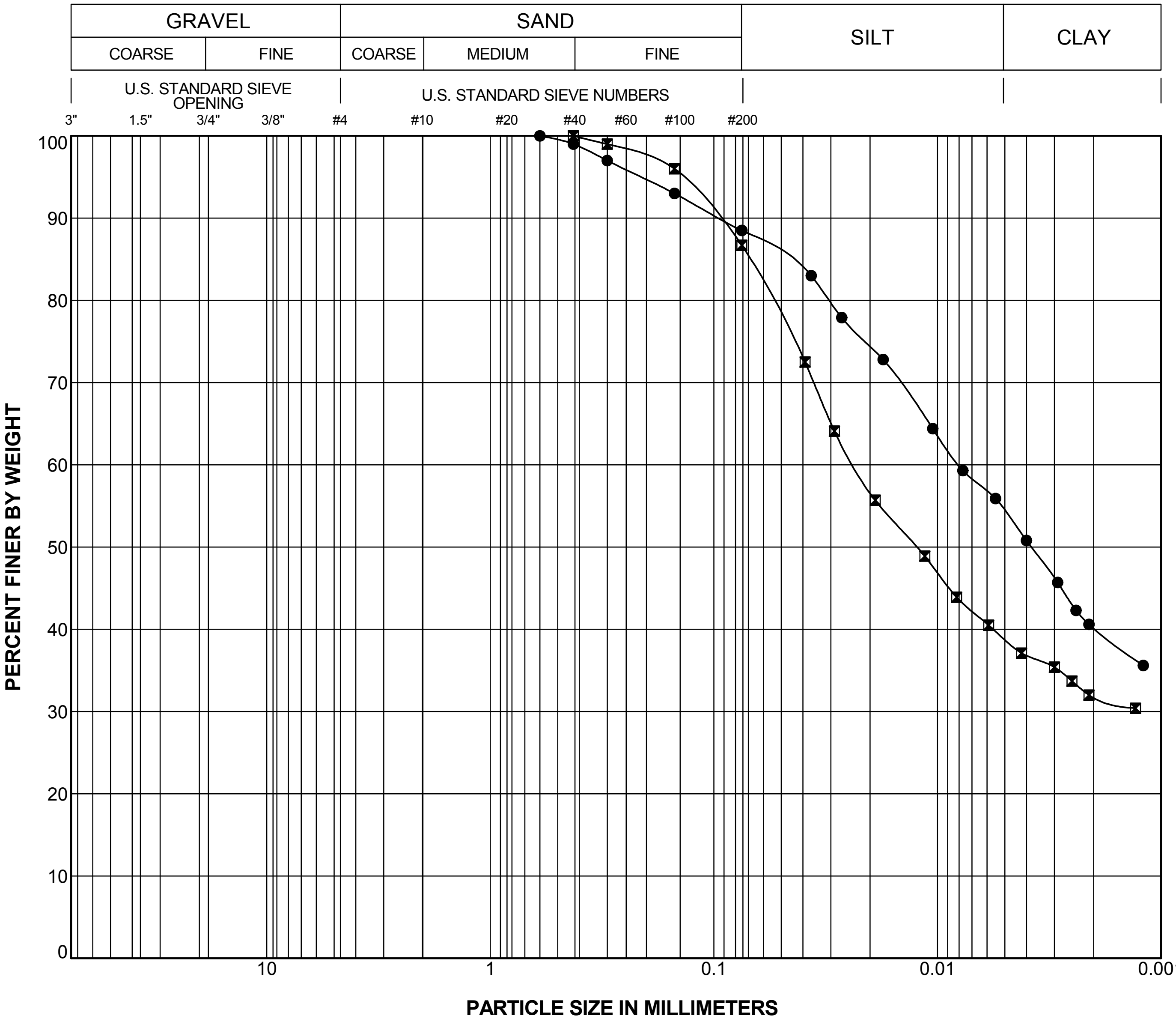
ATTERBERG LIMITS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18

WKA NO.11843.01P



Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
D2	D2-1I	CL	2.0-2.5	●			Brown, sandy lean CLAY
D4	D4-1I	CL	3.5-4.0	⧗			Dark brown to brown, sandy lean CLAY

PARTICLE SIZE DISTRIBUTION

Project: Hiram Johnson High School Track and Football Field
WKA No. 11843.01P

FIGURE A3

WallaceKuhl & ASSOCIATES

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark brown, sandy lean clay (CL)

LOCATION: D12

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 3'	12.9	26.2	101	**57**

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

Wallace Kuhl & ASSOCIATES

EXPANSION INDEX

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO.11843.01P	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Dark brown, sandy lean clay (CL)

LOCATION: D12 (0' - 3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	118	14.2	513	22	95	*

*Sample extruded, therefore R-Value = 5

WallaceKuhl & ASSOCIATES

RESISTANCE VALUE TEST RESULTS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	

Sunland Analytical

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

Date Reported 04/06/2018
Date Submitted 04/03/2018

To: Joey Ybarra
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 : 11843.01P Site ID : D2@0-3.
Thank you for your business.

* For future reference to this analysis please use SUN # 76565-159563.

EVALUATION FOR SOIL CORROSION

Soil pH	6.23		
Minimum Resistivity	3.75	ohm-cm (x1000)	
Chloride	5.5 ppm	00.00055	%
Sulfate	16.5 ppm	00.00165	%

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

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18

WKA NO. 11843.01P

Sunland Analytical

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

Date Reported 04/06/2018
Date Submitted 04/03/2018

To: Joey Ybarra
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 : 11843.01P Site ID : D2@0-3.
Thank you for your business.

* For future reference to this analysis please use SUN # 76565-159564.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	16.5	mg/kg

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

CORROSION TEST RESULTS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

WallaceKuhl & ASSOCIATES

FIGURE	A7
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	

Sunland Analytical

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

Date Reported 04/06/2018
Date Submitted 04/03/2018

To: Joey Ybarra
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 : 11843.01P Site ID : D5@0-3.
Thank you for your business.

* For future reference to this analysis please use SUN # 76565-159565.

EVALUATION FOR SOIL CORROSION

Soil pH	6.49		
Minimum Resistivity	2.68	ohm-cm (x1000)	
Chloride	5.1 ppm	00.00051	%
Sulfate	14.2 ppm	00.00142	%

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

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A8
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18

WKA NO. 11843.01P

Sunland Analytical

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

Date Reported 04/06/2018
Date Submitted 04/03/2018

To: Joey Ybarra
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 : 11843.01P Site ID : D5@0-3.
Thank you for your business.

* For future reference to this analysis please use SUN # 76565-159566.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	13.7	mg/kg

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

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

FIGURE	A9
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	MSM
DATE	04/18
WKA NO. 11843.01P	

APPENDIX B
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APPENDIX B – REFERENCES

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APPENDIX C

Guide Earthwork Specifications

APPENDIX C

GUIDE EARTHWORK SPECIFICATIONS

HIRAM JOHNSON HIGH SCHOOL TRACK AND FOOTBALL FIELD

Sacramento, California

WKA No. 11843.01P

PART I: GENERAL

1.1 SCOPE

a. General Description

This item shall include all clearing and grubbing, preparation of land to be filled, filling, soil treatment, spreading, compaction, observation and testing of the fill, and all subsidiary work necessary to complete the grading of the building and pavement areas to conform with the lines, grades and slopes as shown on the accepted Drawings.

b. Related Work Specified Elsewhere

(1) Trenching and backfilling for sanitary sewer system: Section ____.

(2) Trenching and backfilling for storm sewer system: Section ____.

(3) Trenching and backfilling for underground water, natural gas, and electrical supplies: Section ____.

c. Geotechnical Engineer

Where specific reference is made to "Geotechnical Engineer," this designation shall be understood to include both him and his representative.

1.2 PROTECTION

a. Adequate protection measures shall be provided to protect workmen 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, sidewalks and properties shall be kept free of mud, dirt or similar nuisances resulting from earthwork operations.

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

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

1.3 GEOTECHNICAL REPORT

a. A Geotechnical Engineering Report (WKA No. 11843.01P, April 17, 2018) has been prepared for this site by Wallace - Kuhl & Associates of West Sacramento, California [(916) 372-1434]. A copy is available for review at the Architect's office or 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 he may draw from this report; should he prefer not to assume such risk, he should employ his own experts to analyze available information and/or to make additional borings upon which to base his conclusions, all at no cost to the Owner.

1.4 EXISTING SITE CONDITIONS

The Contractor shall acquaint himself 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 his 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 indicated that the moisture contents of the subgrade and fill materials are satisfactory.

PART II: PRODUCTS

2.1 MATERIALS

a. Imported Select Non-Expansive Fill Materials (Select Fill)

Imported fill materials shall be approved by the Geotechnical Engineer; shall be compactable soils having an Expansion Index less than 20; shall be of three-inch (3") maximum particle size; and, shall have less than five percent (5%) of the material greater than one-inch (1") in maximum dimension.

b. Treated Soils

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

c. Capillary Barrier Material (Crushed Rock)

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 no appreciable material passing a number four (#4) sieve.

d. Lime

1) Lime shall be high-calcium or dolomitic quicklime conforming to the definitions in ASTM Designation C51. 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:

1a) High-calcium quicklime shall contain not less than 113 percent (113%) calcium hydroxide Ca(OH), as determined by California Test Method 414.

1b) Dolomitic quicklime shall contain not less than fifty-seven percent (57%) calcium oxide, Ca0, and not less than ninety-five percent (95%) combined calcium oxide, Ca0, and magnesium oxide, Mg0, as determined by California Test 404.

1c.) When dry sieved in a mechanical sieve shaker for 10 minutes ±30 seconds, a 250 gram test sample of quicklime shall conform to the following grading requirements:

Sieve Size	Percentage Passing
3/8"	98 - 100
No. 100	0 - 25
No. 200	0 - 15

2) 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.

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

4) A Certificate of Compliance in accordance with Caltrans Specification 6-2.03C shall be furnished with each delivery of lime and shall be submitted to the Engineer with a certified copy of the weight of each delivery.

e. Class II aggregate base

Class II aggregate base shall conform to the current requirements of the 2015 Caltrans Specifications, Section 26-1.02B.

e. Controlled Low Strength Material (CLSM)

Controlled low strength material shall consist of a workable mixture of aggregate, cementitious materials, and water; conforming to the provision for slurry cement backfill in section 19-3.02G of the 2015 Caltrans Specifications.

f. Water

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.

g. Other Products

Aggregate base, asphalt concrete and related asphaltic seal coats, tack coat, etc., shall comply with the appropriate provisions of the 2015 State of California (Caltrans) Standard Specifications.

PART III: EXECUTION

3.1 LAYOUT AND PREPARATION

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

3.2 CLEARING, GRUBBING, AND PREPARING SYNTHETIC TRACK FIELD, BUILDING PAD, AND PAVEMENT AREAS

a. All rubble and rubbish; irrigation pipes and underground utilities, associated trench backfill, 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. Excavations and depressions resulting from the removal of such items, as well as existing excavations and loose soil deposits, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soil and backfilled with suitable materials in accordance with these specifications.

b. The surfaces receiving fill shall be stripped of vegetation; or, they shall be thoroughly disced provided that a compactable mixture of soil containing minor amounts of vegetation can be attained which is free of clumps, layers or pockets of vegetation. If proper compaction of the disturbed surface soils cannot be achieved, those materials shall be excavated, to a depth satisfactory to the Geotechnical Engineer, so that a firm base for support of engineered fill can be attained.

c. All fill to be constructed that will be below the depth of lime treatment shall be constructed in accordance with Section 3.3 of these specifications and the surfaces receiving fill shall be prepared in accordance with the following paragraphs in this section: Section 3.2.

d. All loose fill soils and/or saturated materials shall be over-excavated to firm soil, as determined by the Geotechnical Engineer, and the resulting excavations shall be backfilled with suitable materials in accordance with these specifications; or, where saturated surface soils are located over native undisturbed soils, the subgrades may be stabilized with high-calcium or dolomitic quicklime to depths and with compactive effort meeting the satisfaction of the Geotechnical Engineer.

e. The surfaces upon which fill is to be placed shall be plowed or scarified to a depth of at least twelve inches (12"), until the surface is free from ruts, hummocks or other uneven features which would tend to prevent uniform compaction by the selected equipment.

f. When the moisture content of the subgrade is less than two percent (2%) above 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. After the foundations for fill have been cleared, moisture conditioned, and plowed or scarified, they shall be recompacted in place to a depth of at least twelve inches (12") to a minimum of ninety percent (90%) of the ASTM D1557 maximum dry density.

i. The building pad areas shall be defined as extending at least five feet (5') beyond the proposed building lines. The pavement areas shall be defined as extending at least two feet (2') beyond the edges of pavement.

3.3 CONSTRUCTION OF UNTREATED SUBGRADES

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

b. When the moisture content of 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.

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 not less than ninety percent (90%) of maximum dry density 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 fill operations shall be continued until the fills have been brought to the slopes and grades shown on the accepted Drawings.

3.4 LIME-STABILIZED SUBGRADE CONSTRUCTION

a. Placing Material

The 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. Preparing Material

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

1) Lime shall be added to the material to be treated at a rate of at least four and a half pounds (4.5 lbs) per square foot, to a depth sufficient to produce a 12-inch-thick compacted layer of 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 footmeter 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°F. 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.

10) All lime-treatment in pavement areas should be in general conformance with Section 24 of the 2015 Caltrans Standard Specifications.

d. Spreading and Compacting

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%) within building pad and exterior flatwork areas, and ninety-five percent (95%) within synthetic turf and pavement areas, 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 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. 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 part-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 <u>FINAL SUBGRADE PREPARATION USING UNTREATED SOILS</u>

a. All original ground preparation and engineered fill placed within building pads and slab-on-grade concrete areas shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. The upper twelve inches (12") of final building pad subgrades shall be composed of approved, compactable, granular, low expansion potential fill at a uniform moisture content not less than the optimum moisture (per ASTM D1557) and shall be uniformly compacted to not less than ninety percent (90%), as defined by that test.

b. The upper six inches (6") of any untreated final synthetic turf and pavement subgrades shall be uniformly compacted to at least ninety-five (95%) percent of the ASTM D1557 maximum dry density, at a moisture content at least the optimum moisture content regardless of whether the grade is achieved by filling, by excavation, or is left at or near original site grade.

3.6 Utility Trench Backfill

a. Bedding and initial backfill shall conform to the pipe manufactures recommendations and the applicable governing agencies standards.

b. If trench foundations are unstable or are composed of deleterious materials, the trench foundation shall be over excavated a minimum of six inches (6”) and replaced with crushed rock wrapped in filter fabric.

c. Trench zone backfill shall use native soils or select fill and shall extend from the top of the initial backfill to a point twelve inches (12”) below finished grade.

d. Trench zone and final zone backfill shall be compacted to not less than ninety percent (90%) of the maximum dry density as determined by ASTM D1557.

e. Final zone backfill in the upper twelve inches (12”) shall conform to the standards for final subgrade preparation shown in sections 3.3 to 3.5 above.

f. Where select fill or lime treated soils are removed from trenches, the material shall be replaced with compacted aggregate base.

g. CLSM may be used in place of native soil or select fill if approved by the Geotechnical Engineer.

h. Utility trenches aligned nearly parallel with foundations shall be a minimum of three feet (3’) from the outer edge of foundations, wherever possible; shall not encroach on a zone extending outward at one horizontal to one vertical (1H:1V); and, shall not remain open longer than 72 hours.

3.7 CONSTRUCTION OF PAVEMENT BASE COARSE

a. Materials supporting pavements shall be Class II aggregate base and shall be moisture conditioned to the optimum moisture content and compacted to not less than ninety-five percent (95%) of the maximum dry density as defined by ASTM D1557.

3.8 FOUNDATIONS

a. Foundation excavations shall be sized consistent with the project plans; and shall extend to firm undisturbed soil or engineered fill capable of bearing required loads. If loose or soft material is encountered, the material shall be over excavated and replaced with fill placed and compacted as required for foundation subgrade soils.

b. Foundation excavations shall be free of loose or deleterious material. Stockpiling of loose material shall not be permitted near open foundation excavations.

3.9 TESTING AND OBSERVATION

a. All grading operations, including lime-treatment of the subgrades, shall be tested and observed by the Geotechnical Engineer, serving as the representative of the Owner.

b. Field density tests and other tests shall be made by the Geotechnical Engineer after compaction or placement 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. Prior to placement of steel or concrete, foundation excavations shall be observed by the Geotechnical Engineer for conformance with the recommendations in the Geotechnical Report and the project plans.

d. 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.

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