Geotechnical Engineering
and Geologic Hazards Investigation

WESTMORE OAKS MODERNIZATION

WKA No. 12277.01P

April 2, 2019

Prepared for:

Washington Unified School District
930 Westacre Road
West Sacramento, California 95691

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Wallace Kuhl & ASSOCIATES

CORPORATE OFFICE
3050 Industrial Boulevard
West Sacramento, CA 95691
916.372.1434 phone
916.372.2565 fax

STOCKTON OFFICE
3422 West Hammer Lane, Suite D
Stockton, CA 95219
209.234.7722 phone
209.234.7727 fax

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WESTMORE OAKS MODERNIZATION
1504 Fallbrook Street
West Sacramento, California
WKA No. 12277.01P
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INTRODUCTION

We have completed a geotechnical engineering and geologic hazards study for the planned Westmore Oaks Modernization project in West 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.

Work Scope

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 of the site;
5. review of previous geotechnical engineering and geologic hazards reports prepared for the site;
6. subsurface exploration, including completion of two Cone Penetration Tests (CPTs) to depths of 50 feet below the existing ground surface BGS;
7. subsurface exploration, including the drilling and sampling of five borings to depths of approximately 20 to 21½ feet BGS;
8. evaluation of geologic hazards influential to the site in accordance with California Geological Survey Note 48 based on review of the preceding information;
9. Bulk sampling of near-surface soils;
10. Laboratory testing of selected soil samples;
11. Engineering analyses; and,
12. 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 - 12	Logs of Soil Borings
4	Geologic Map	13	Unified Soil Classification System
5	Geologic Cross Section	14	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).
- CPT Logs and Liquefaction analysis results (Appendix C).
- Previous subsurface exploration and laboratory test results obtained at the site (Appendix D)

Proposed Development

We understand the project will consist of the modernization of the existing campus, including the design and construction of two new single-story, slab-on-grade buildings and a slab-on-grade building addition to the existing kitchen building. We understand the new buildings will have a total building footprint of about 23,310 square feet in plan area. Associated improvements will consist of modernization of several existing buildings, new asphalt concrete and concrete parking and drive areas, exterior concrete flatwork, and underground utilities.

Previous Studies at the Site

Wallace-Kuhl & Associates (WKA), Inc previously prepared a *Geologic & Geotechnical Engineering Report* (WKA, Inc. No 7874.01) dated December 18, 2007 for the Westmore Oaks Elementary School project. The previous report included the performing six borings and one Cone Penetration Test (CPT) to depths of approximately 20 to 50 feet below the ground surface. The borings and CPT were performed within the proposed building footprints for the project, none of which are located within the proposed building footprints evaluated for this study.

The subsurface exploration logs and laboratory test results previously performed at the site are included in Appendix D.

FINDINGS

Site Description

The Westmore Oaks School site is located at 1504 Fallbrook Street in West Sacramento, California (Figure 1). The campus encompasses an approximately 9-acre parcel identified by Yolo County Assessor's Parcel Number (APN) 058-220-043-000. The site is bordered to the north by Grande Vista Avenue, beyond which is a residential subdivision; to the northeast by Fallbrook Street, beyond which is a residential subdivision; and to the east, south and west by a residential subdivision.

At the time of our explorations on March 8, 2019, and March 12, 2019, the areas of proposed construction consisted of asphalt concrete pavements, concrete flatwork and landscaped lawn areas. The area of the new buildings at the south end of the site also were covered in existing portable classroom buildings, which were enclosed by a chain-link fence. Relatively mature trees were observed south and west of the proposed building locations. An existing solar array is located at the east end of the site, just east of the proposed new building locations. Irrigation lines and utility pipes were observed throughout the area.

The overall site is relatively flat. Review of the topographic map of the *Sacramento West Quadrangle*, published by the USGS, dated 2018, indicates the elevation of the site is approximately +20 feet relative to mean sea level (msl). A portion of the USGS topographic map containing the site is presented as Figure 3.

Site History

We reviewed historical aerial photographs from the years 1947, 1957, 1964, 1966, 1993, 1998, and 2002 through 2018. The site and surrounding vicinity are shown to be fallow undeveloped land in the 1947 photo. In the photos from 1957 to 1966 the site is shown to have the school developed. The photos from 1993 and 1998 appear to show the site in a similar condition with more buildings added on the southern portion of the site. Also, there were photoelectric solar arrays added between July of 2015 and July of 2016 according to the photos.

General Site Geology

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

Surface elevations within the Great Valley generally range from several feet below mean sea level to more than 1000 feet above sea level. The major topographical feature in the Great

Valley is the Sutter Buttes (a volcanic remnant) that rise approximately 1980 feet above the surrounding valley floor.

According to the USGS Geologic Map of the Sacramento Quadrangle (Wagner, 1981), the project site is underlain by the Quaternary-aged Basin Deposits (Qb). The California Geological Survey's Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle (Gutierrez, 2011) identifies the area underlying the site as Holocene Basin Deposits (Qhb). The geologic materials that comprise both the Basin Deposits are primarily sands, silts, and gravels. The mapped geology was found to be consistent with the subsurface soil conditions encountered within our borings and CPTs performed at the site, which indicate similar deposits, to the approximate depth explored of 50 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.

Faulting

Based on our review of available geologic and seismic references, the site for the proposed modernization project at Westmore Oaks is not located across a mapped trace of any fault and we observed no surface evidence of faulting during our site reconnaissance. The site is *not* located within an Alquist-Priolo (AP) Earthquake Fault Zone (Parrish, 2018).

Using the *Revised 2002 California Probabilistic Seismic Maps* (Cao, et al, 2003), we have prepared Table 3 containing faults and fault systems within about 100 kilometers of the site that are considered capable of producing earthquakes with moment magnitude (M_W) of 6.5 or greater. A fault location map is presented on Figure 6.

Table 2
Faults Influential to Westmore Oaks

Fault Name	Distance		Maximum Magnitude (M_w)
	miles	kilometers	
GREAT VALLEY 4A, TROUT CREEK	22.1	35.6	6.6
GREAT VALLEY 4B, GORDON VALLEY	24.7	39.8	6.8
GREAT VALLEY 3, MYSTERIOUS RIDGE	26.0	41.8	7.1
GREAT VALLEY 5, PITTSBURG KIRBY HILLS	28.6	46.1	6.7
FOOTHILL FAULT SYSTEM	31.6	50.9	6.5
HUNTING CREEK-BERRYESSA	33.7	54.2	7.1
GREEN VALLEY, CONNECTED	34.3	55.2	6.8
CONCORD/GV; CON+GVS+GVN)	37.2	59.8	6.7
CONCORD/GV; GVS+GVN)	37.2	59.8	6.5
CONCORD/GV; CON+GVS)	39.5	63.5	6.6
WEST NAPA	43.3	69.7	6.7
GREENVILLE, CONNECTED	49.1	79.0	7.0
GREAT VALLEY 2	52.0	83.7	6.5
MOUNT DIABLO THRUST	53.6	86.2	6.7
BARTLETT SPRINGS	55.2	88.9	7.3
HAYWARD-RODGERS CREEK;RC+HN+HS	56.2	90.4	7.3
HAYWARD-RODGERS CREEK;RC	56.2	90.4	7.1
HAYWARD-RODGERS CREEK;RC+HN	56.2	90.4	7.2
HAYWARD-RODGERS CREEK;HN+HS	56.5	91.0	7.0
HAYWARD-RODGERS CREEK;HN	56.5	91.0	6.6
GREENVILLE; GS+GN	61.5	98.9	6.9
GREENVILLE; GS	61.5	98.9	6.6
CALAVERAS;CN	56.6	91.2	6.9
CALAVERAS;CN+CC	56.6	91.2	7.0
CALAVERAS;CN+CC+CS	56.6	91.2	7.0
GREAT VALLEY 7	57.8	92.9	6.9
MAACAMA-GARBERVILLE	59.6	95.8	7.4
COLLAYOMI	60.6	97.5	6.7
HAYWARD-RODGERS CREEK;HS	61.3	98.7	6.8
GREAT VALLEY 1	61.7	99.3	6.8

The Bear Mountain fault zone is the westerly-most fault within the Foothills Fault System, which consists of numerous northwesterly trending faults along the western edge of the Sierra Nevada range. The Foothills Fault System is generally bounded by the Bear Mountain and the Melones fault zones (Wagner, 1981). The closest segment of the Bear Mountain fault zone is approximately 15 kilometers east of the site. The closest segment of the Melones fault zone is approximately 55 kilometers east of the site. Most of the faults within Foothills Fault System are mapped as pre-Quaternary displacement with some segments mapped as having late-Quaternary displacement (Jennings, 1994). The faults are defined as CGS Class C fault sources.

According to the *Fault Activity Map of California and Adjacent Areas*, prepared by the CGS (Jennings, 1996), the closest Holocene-aged fault to the site is indicated to be the south end of the northwest-southeast trending Dunnigan Hills Fault, located approximately 34 kilometers northwest of the site. The closest faults of any age are shown as the Willows Fault located approximately 11 kilometers east, the Capay Fault, located about 23 kilometers west, and the Midland Fault located about 26 kilometers west of the site. These faults are all indicated to be of pre-Quaternary age, with no activity in the last 1.6 million years.

<u>Coseismic Ground Deformation</u>

The California State Legislature passed the Seismic Hazards Mapping Act (SHMA) in 1990 (Public Resources Code Division 2, Chapter 7.8) as a result of earthquake damage caused by the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes. The purpose of the SHMA is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, and other hazards caused by earthquakes (CGS SP117). We are not aware of any mapping of geologic hazards for this area based on review of currently published maps available on the California Geological Survey (CGS) website.

<u>Historic Seismicity</u>

Seismological data regarding significant historical earthquakes affecting the site was obtained using the commercially available software program EQSEARCH (Blake, 2000; database updated to August 2018). The EQSEARCH database was developed by extracting records of events greater than magnitude 4.0 from the Division of Mine and Geology Comprehensive Computerized Earthquake Catalog and supplemented by records from the USGS; University of California, Berkeley; the California Institute of Technology; and, the University of Nevada at Reno. A search radius of 100 kilometers (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 M_R 6.4 earthquake on April 19, 1892, with an epicenter located approximately 44.7 kilometers (27.8 miles) southwest of the site.

Subsurface Soil Conditions

Five borings were drilled and sampled to depths of approximately 20 to 21½ feet below existing grades, and two CPTs were performed to depths of about 50 feet below existing grades at the approximate locations as shown in Figure 2. The soils encountered at the boring locations generally consisted of yellowish brown to brown, loose silty sand to depths of about six to 10 feet BGS, underlain by very soft to medium stiff fat clay to the explored 20 to 21½ foot depths of the borings. Interbedded sand and lean clay layers were encountered within the fat clay layers at borings D1 and D5. The soil conditions at the CPT locations generally consisted of silty clays to depths of about 24 to 26 feet BGS underlain by sands to the explored 50 foot depths of the CPTs.

For soil conditions at a particular location, please refer to the Logs of Borings on Figures 8 through 12. A Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 13.

For more detail regarding the soil conditions at a CPT location, please refer to the interpretation logs presented in Appendix C.

The soil conditions encountered at the borings and CPTs performed for this study are generally consistent with the soil conditions encountered in the 2007 borings and CPT. The 2007 subsurface exploration logs and laboratory test results are included in Appendix D.

Groundwater

On March 8, 2019, groundwater was calculated to be about three feet and seven feet below the ground surface within the CPT soundings. On March 12, 2019, groundwater was encountered between about seven feet and 13 feet below the ground surface within the borings. Groundwater was encountered in two of the borings performed in 2007 at depths of approximately 14 feet below the ground surface

To supplement our groundwater data, we reviewed available groundwater information at the California Department of Water Resources (DWR) website. The DWR periodically monitors groundwater levels in wells across the state. Their website shows a well located approximately one mile northwest of the site. The well is identified as Well No. 09N04E32R001M with a ground surface elevation of about +15 feet (NAVD 88), similar to the project site. Groundwater data for this well was recorded from October 5, 1972 to at least February 24, 1992. Data shows the highest recorded groundwater elevation was about +4 feet (NAVD 88) at the well on March 26, 1991 (about 11 feet below the ground surface at the well location). The lowest recorded groundwater elevation was about -28 feet (NAVD 88) at the well on July 29, 1977 (about 42 feet below the ground surface at the well location). Further review of the data indicates that between the period of 1972 and 1992, groundwater at the well location fluctuated between approximate elevations of +4 to -42 feet (NAVD 88).

CONCLUSIONS

Seismic Hazards

No active or potential active faults are known to underlie the site of the existing school campus based on the published geologic maps or aerial photographs that we reviewed. The school 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.

Liquefaction

Liquefaction is a soil strength and stiffness loss phenomenon that typically occurs in loose, saturated cohesionless soils as a result of strong ground shaking during earthquakes. The potential for liquefaction at a site is usually determined based on the results of a subsurface geotechnical investigation and the groundwater conditions beneath the site. Hazards to buildings associated with liquefaction include bearing capacity failure, lateral spreading, and differential settlement of soils below foundations, which can contribute to structural damage or collapse.

The results of the borings and CPTs performed at the site revealed the underlying soils generally consists of relatively loose sands, underlain by soft to stiff fat clays, with interbedded sand and clay layers, over sands to the explored depths of 50 feet below existing site grades. A high groundwater level of about three feet below the existing ground surface was estimated based on the site and soil conditions. Based on the soil conditions encountered at the exploration locations and the anticipated high groundwater level at the site, an evaluation of the liquefaction potential is required at the site in accordance with the 2016 CBC.

A liquefaction analysis to determine factors of safety against liquefaction was performed for the soil and groundwater conditions encountered at CPT-1 and CPT-2.

Liquefaction Analysis and Results

In performing our analysis we used the soil liquefaction assessment software CLiq (Version 2.2.1.4) developed by GeoLogismiki that utilizes data collected from CPT soundings to determine factors of safety against liquefaction for varying earthquake input energies. The program uses the results of the National Center for Earthquake Engineering Research (NCEER) liquefaction evaluation methods summarized by Youd, et al (2001). Input values were obtained using the results of CPT-1 and CPT-2. Based on our review of historical groundwater levels at the site, a design groundwater level of three feet below the ground surface was used during a design earthquake in our liquefaction analysis. A peak ground acceleration (PGAm) of 0.32 g was used in our liquefaction analysis based on Equation 11.8-1 of ASCE Standard 7-10. A mode magnitude earthquake of 6.4 was used for our analysis using the 2014 USGS National Seismic Hazard Mapping Project (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation web site.

The results of the liquefaction analysis indicate isolated soils layers within the soil profile at various depths and thickness have a factor of safety against liquefaction below 1.3. A factor of safety below 1.3 requires a liquefaction-induced settlement analysis. The analysis revealed liquefaction would occur at CPT-1 from about five to seven feet and 27to 43 feet, and at CPT-2 from six to 11 feet and 24 to 42 feet.

Seismically Induced Settlement

The results of the liquefaction analysis at the CPT locations indicate calculated seismic settlements of about 4.2 and 3.32 inches at CPT-1 and CPT-2, respectively. Given the results of our analysis performed for this investigation, the worst-case estimate of total post-liquefaction settlement is calculated to be about 4¼ inches. Differential seismic induced settlement across 50 feet, or the shortest dimension of the structure, whichever is less, is estimated to be about one inch. These estimates of post-liquefaction seismic settlements represent free-field ground settlement, not settlement of the proposed structure. The presence of non-liquefiable clayey soil layers overlying and interbedded within the liquefiable layers will likely mitigate the impact of seismically induced settlement at the ground surface.

In our opinion, new structures should be designed to comply with California Administrative Code, Title-24, Section 4-301 to repairable architectural and structural damage from “worst-case scenario” total seismic settlement of 4¼ inches and differential settlements of one inch across 50 feet, or the shortest dimension of the structure, whichever is less.

Liquefaction potential at the site was also evaluated based on the Liquefaction Potential Index (LPI). The LPI is a measure of the liquefaction potential based on an analysis of the entire vertical soil profile not just discrete layers (Iwasaki, 1986; Toprak and Holzer, 2003). Factors taken into consideration for the LPI calculations include: thickness of the liquefied layer; proximity of the liquefied layer to the surface; and, the factor of safety. The LPI ranges from 0 to 100 with the value zero representing no liquefaction potential. Surface manifestations of liquefaction occur at LPI ≥ 5. The LPI for the soil conditions at CPT-1 and CPT-2 was calculated to be 8.29 and 6.80, respectively, indicating liquefaction risk is classified as high during the design seismic event (mode magnitude earthquake of 6.4 and a PGA of 0.32 g).

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

Copies of the output files for the liquefaction analysis are provided in Appendix C.

Seismic Site Class

Based on Table 20.3-1 of ASCE 7-10, a seismic Site Class D applies to sites with average Standard Penetration Test (SPT) blow counts between 15 and 50 for the upper 100 feet of the ground surface. Review of the CPT data indicates SPT blow counts correlated from the data (Lunne, et al., 1997) average at least 15 blows per foot to the explored 50 foot depths of the CPTs. Conservatively assuming that the deeper materials from depths of 50 to 100 feet at CPT-1 and CPT-2 have SPT blow counts of 15 or higher , the average SPT blow count for the upper 100 feet at the CPTs will be at least 15 blows per foot. Therefore, according to the information obtained from the CPT measurements, the soils at this site can be designated as site Class D in determining seismic design forces for this project in accordance with Table 20.3-1 of ASCE 7-10 and the 2016 CBC.

However, the results of our liquefaction analysis (see the Liquefaction Potential and Liquefaction Analysis and Results sections of this report) indicate that at least some of the soils encountered at the CPTs are “vulnerable to potential failure or collapse under seismic loading”, which would classify the site as Site Class F in accordance with Section 20.3.1 of *ASCE 7-10*. Further review of Section 20.3.1 of *ASCE 7-10* indicates that an exception can be made for structures having fundamental periods of vibration equal or less than 0.5 seconds and that a site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a Site Class is permitted to be determined in accordance with Section 20.3 of ASCE 7-

10 and the corresponding Fa and Fv values, as determined from Tables 11.4-1 and 11.4-2 of *ASCE 7-10*.

Due to the planned height (one-story) of the classroom buildings, we assume that the fundamental period of vibration for the planned buildings are less than or equal to 0.5 seconds, and therefore meets the requirement for the Site Class F exemption. Therefore, based on the data collected from CPTs, it is our opinion that a Site Class D is applicable to the proposed buildings, in accordance with Table 20.3-1 of *ASCE 7-10* and the 2016 *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 proposed school buildings.

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

Latitude: 38.5696° N Longitude: 121.5340° 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	0.715 g
1.0 second Period MCE	Figure 22-2	Figure 1613.3.1(2)	S_1	0.303 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	1.228
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.794
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-37	S_{MS}	0.878 g
	Equation 11.4-2	Equation 16-38	S_{M1}	0.544 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	0.585 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.363 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

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

Landslides

The topography across the site is relatively flat based on visual observations and review of topographic maps. The USGS Topographic Map of the Sacramento West Quadrangle, California indicates the surface elevation at the site is approximately +20 feet msl. Based on the fact that the site topography is flat and there are no slopes near the site, it is our opinion that the potential for landslides is nonexistent.

Naturally Occurring Asbestos (NOA) and Radon Gas

Review of *A General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos, California Geological Survey Open-File Report 2000-19* (Churchill and Hill, 2000) indicates the site is not underlain by ultramafic rocks likely to contain asbestos.

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, WKA does not consider the presence of naturally occurring radon gas to be likely.

Flood Hazards

According to the Flood Insurance Rate Map for Yolo County, Panel Number 060728 0010B, dated January 19, 1995), the Westmore Oaks campus is located within ZONE X defined as "Areas determined to be outside 500-year flood plain." (Figure 8).

Dam Inundation

The Health and Safety Element of the Yolo County General Plan (COY, 2009) identifies six major dams that have the potential for human injury or loss of life in or near the county if failure were to occur. The site lies approximately 23 miles southwest of the Folsom Dam. The California Office of Emergency Services indicated that the school site would be affected by the failure of the Folsom Dam.

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 groundwater, oil, natural gas or oxidation of peat. Based on our subsurface sampling, the soil at the project site is predominately silty clays with interbedded sand layers overlying moderately dense to dense sands to the explored 50-foot depths of the CPTs performed on site. These materials are not susceptible to hydrocollapse or land subsidence.

Review of the Health and Safety Element of the County of Yolo *2030 Countywide General Plan* (COY, 2009) reveals that the site does not lie in an area of known subsidence.

Based on the subsurface conditions encountered at the proposed school site, it is our opinion that settlement at the site due to hydrocollapse and/or subsidence is very unlikely.

Bearing Capacity

Relatively loose near-surface soils were encountered within the upper 10 feet at the borings performed at the site. The loose soils are not considered capable of providing adequate or uniform support for the planned buildings in their current condition without experiencing significant total and/or differential settlements, which can potentially result in structural damage.

Therefore, it is our opinion the planned buildings will need to be supported on an improved subgrade or a deep foundation system.

It is also our opinion that an improved subgrade consisting of over-excavation, processing, and re-compaction of the over-excavated soils beneath foundations, or an improved subgrade consisting of rammed aggregate piers (RAP), will be necessary to adequately support the buildings on conventional shallow foundations.

Several deep foundation systems also were considered for support of the buildings, including drilled piers and driven and auger-cast piles. However, we anticipate a deep foundation system will not be as cost effective as shallow foundations on an improved subgrade.

Specific recommendations for shallow foundations supported on an over-excavated/re-compacted and/or an improved subgrade consisting of a RAP system are provided in this report.

Effect of New Construction on Existing Development

There are existing buildings and other improvements (e.g. pavements, exterior flatwork, underground utilities, etc.) adjacent to the planned buildings. We assume that the buildings are supported on conventional shallow foundations (isolated spread and/or continuous footings). It is our opinion that excavations associated with the proposed development of the site should not affect the foundations of the existing buildings and other improvements, provided the new excavations are at least 10 feet from the existing improvements or do not encroach within a one horizontal to one vertical (1H:1V) projection from the bottom of the existing building foundations or improvements. If excavations will encroach within the zone described above, stabilizing the existing buildings and/or other improvements using an underpinning system that supports the existing foundations should be evaluated by the Geotechnical Engineer in coordination with the design team.

Soil Expansion Potential

The near surface soil encountered at the boring locations generally consist of granular soils, which are not considered expansive. Laboratory test results on the near-surface soils indicates these materials possess low plasticity when tested in accordance with American Society of Testing and Materials (ASTM) D4318 test method (see Figure A2). In addition, laboratory testing of soils collected from the upper three feet within the planned location of the proposed buildings revealed the near-surface soils possess a “low” expansion potential (Expansion Index [EI]=35) when tested in accordance with the ASTM D4829 test method (see Figure A3). Based on the soil conditions encountered at the borings and the results of the laboratory testing, special site preparation or foundation designs to mitigate expansive soils are not required for development of this site.

Pavement Subgrade Quality

Based upon laboratory testing of near-surface soils at the site the anticipated pavement subgrade soils indicate poor to moderate quality materials for support of asphalt concrete pavements. A Resistance ("R") value of 25 was obtained on near-surface soil samples tested in accordance with California Test 301 (Figure A4). Based on the results of the R-value test and our experience in the area, an R-value of 25 is considered appropriate for design pavements at the site.

Excavation Conditions

The surface and near-surface soils at the site should be readily excavatable with conventional earthmoving and trenching equipment. Subsurface remnants from existing development of the site may be encountered and can be slow to excavate with a standard, rubber-tired backhoe; however, experience has shown that excavators can remove these materials with moderate effort.

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 and Seasonal Water

Groundwater was observed in boring D3 at a depth of approximately seven feet below existing site grades on March 12, 2019. This boring was left open for several hours; however, the boring may not have been left open long enough for groundwater to reach static equilibrium. The CPT revealed water could be as shallow as three feet below existing site grades on March 8, 2019.

Based on explorations performed at the proposed building sites and available groundwater data, we anticipate excavations extending about five feet below existing site grades may encounter groundwater and require dewatering (depending on time of year time). For planning purposes, groundwater should be anticipated to be about five feet below existing site grades (elevation of about +15 feet msl). Groundwater monitoring wells may be installed near the improvement areas prior to construction to evaluate actual groundwater levels before and during construction. If groundwater is encountered, the use of sumps, submersible pumps, deep wells or a well point system could be used as methods to lower the groundwater level. The dewatering method used will depend on the soil conditions, depth of the excavation and amount of groundwater present within the excavation. Dewatering, if required, should be the contractor's responsibility. The dewatering system should be designed and constructed by a dewatering contractor with local experience. We recommend the selected dewatering system lower the groundwater level to at least two feet below the bottom of the proposed excavations.

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

On-site Soil Suitability for Use in Fill Construction

The on-site soils encountered in our borings are considered suitable for use in engineered fill construction, provided these materials do not contain rubble, rubbish, significant organic concentrations, and are at a workable moisture content appropriate for compaction. Imported materials, if necessary, should be compactable granular soils and be approved by our office prior to importing the materials to the site.

<u>Soil Corrosion Potential</u>

A soil sample was 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 test is summarized in Table 4. Copies of the corrosion potential test results performed by Sunland Analytical are presented on Figures A5 and A6.

Table 4: Soil Corrosivity Testing Results

Analyte	Test Method	D4 (0-3')
pH	CA DOT 643 Modified*	6.82
Minimum Resistivity	CA DOT 643 Modified*	6,770 ϑ-cm
Chloride	CA DOT 422	2.0 ppm
Sulfate	CA DOT 417	4.4 ppm
Sulfate – SO4	ASTM D-516	5.7 mg/kg

* = 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 two to ten 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

Prior to site grading, construction areas should be cleared of rubble, deleterious debris, if any, and any other surface and subsurface items designated for removal to expose undisturbed firm and stable 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, if encountered, to be abandoned should be completely removed, including existing trench backfill.

All trees/large brush designated for removal, if any, should include the rootballs and roots ½ inch or larger in size. Adequate removal of debris and tree roots may require handpicking by laborers to clear the subgrade soils to the satisfaction of our on-site representative.

Soils containing excessive organic soils should be removed and not used within the 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.

Existing underground utilities within the proposed building pads should be completely removed and/or rerouted as necessary. Any existing underground utilities designated to be removed or relocated should include all trench backfill and be replaced with engineered fill. Utilities located outside the building areas should be properly abandoned (i.e., fully grouted provided the abandoned utility is situated at least 2½ feet below the final subgrade level to reduce the potential for localized “hard spots”).

Existing pavements and flatwork (asphalt concrete and concrete) 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.

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

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

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.

<u>Subgrade Preparation</u>

Based on the soil conditions encountered at the borings performed at the site, we conclude the existing near-surface soils at the site are not considered suitable for shallow foundation support of the planned buildings unless the subgrade soils are improved (i.e. over-excavated and recompacted or improved with a RAP system). Therefore, subgrade preparation for development of the site will depend on the specific ground improvement alternative chosen (i.e. subgrade over-excavation or subgrade improvement with a RAP system). A discussion of the subgrade preparation required for the subgrade over-excavation and RAP system ground improvement alternatives is provided below. The intent of these subgrade improvement alternatives is to provide adequate and uniform support for the planned buildings.

Over-excavation of Building Pad Areas

The following grading recommendations should be used for support of the planned buildings if shallow conventional foundation systems are supported on over-excavated and re-compacted subgrade soils (i.e. without a RAP system). Following site clearing activities, the building pad areas should be over-excavated to a depth of at least five feet below existing grades or at least three feet below the bottom of the foundations, whichever is deeper. The over-excavation should extend at least five feet beyond the edge of exterior foundations or the building footprints, whichever is greater. Any debris exposed by the required over-excavation should be removed and the resulting excavations should be restored to grade with engineered fill placed and compacted in accordance with the recommendations in this report. The lateral extents of the required over-excavation should be clearly marked on the final grading plans. The Geotechnical Engineer should be given the opportunity to review the final grading plan to determine if the intent of the over-excavation recommendation has been properly implemented.

Following over-excavation operations, the exposed subgrade soils should be statically rolled to smooth out the bottom of the excavation. Following the rolling operations, a layer of geogrid reinforcement (Tensar BX1100, Tensar TX140, Mirafi 5XT, or equivalent) should be placed directly on the exposed subgrade. Overlap of the geogrid reinforcement should be performed in accordance with the manufacturer's recommendations. The geogrid should be covered with at least a six inch thick lift of Caltrans Class 2 aggregate base and the aggregate base should be uniformly compacted to at least 90 percent relative compaction at no less than the optimum moisture content. Relative compaction should be based on the maximum dry density as determined in accordance with the ASTM D1557 Test Method. Recycled aggregate base is acceptable for use. The resulting over-excavations should be restored with engineered fill placed and compacted in accordance with Engineered Fill Construction section of this report.

Rammed Aggregate Pier (RAP) Alternative

If a RAP system will be used to improve the subgrade beneath the footprint of the buildings, over-excavation of the building pads would not be necessary. The RAP system uses a drilled shaft backfilled with compacted aggregate base to improve subgrade stability and reduce settlements within the treated areas. The RAP system should be designed by a professional engineer in the State of California that is qualified and experienced in RAP design.

Although over-excavation of the building pads would not be required if a RAP system is used for support of the buildings, the floor slab subgrade should be scarified and compacted to provide adequate and uniform floor slab support across the building footprints. Specifically, areas to receive fill and at-grade areas should be scarified to a depth of at least 12 inches, thoroughly moisture conditioned to at least the optimum moisture content, and uniformly compacted to at least 90 percent relative compaction.

Pavement and Exterior Flatwork

Please note that the ground improvement recommendations provided above are not necessary within areas designated for exterior flatwork or pavements (outside of the building pad areas). Any other surfaces to receive fill outside of the building pad areas, achieved by excavation or remain at grade, should be scarified to a depth of at least 12 inches, thoroughly moisture conditioned to at least the optimum moisture content and uniformly compacted to at least 90 percent relative compaction.

The upper six inches of pavement subgrades should be uniformly compacted to at least 95 percent relative compaction at a moisture content of at least the optimum moisture content, regardless of whether final grade is established by excavation, engineered fill or left at grade. Additional recommendations regarding pavement subgrades are provided in the Pavement Design section of this report.

General

Compaction of all subgrade soils should be performed using a heavy, self-propelled, sheepsfoot compactor capable of achieving the required compaction and must be performed in the presence of the Geotechnical Engineer’s representative who will evaluate the performance of subgrade under compactive load. Difficulty in achieving subgrade compaction may be an indication of loose, soft or unstable soil conditions that could require additional excavation. If these conditions exist, additional subgrade stabilization recommendations may be required at the time of construction.

Engineered Fill Construction

On-site soils are considered suitable for use in engineered fill construction, if they do not contain significant concentrations of organic materials, rubble debris, or particles greater than three inches in maximum dimension. Imported fill materials, if required, should be granular, compactable materials with a Plasticity Index of 15 or less when tested in accordance with ASTM D4318; an Expansion Index of 20 or less when tested in accordance with ASTM D4829; an organic content less than four percent; do not contain particles greater than three inches in maximum dimension, and be within a compactable moisture content. Additionally, import fill materials that will be used within pavement areas should be non-expansive and have a minimum Resistance value equal to or greater than the on-site soils when tested in accordance with California Test 301. Imported fill should be observed and approved by the Geotechnical Engineer at least three business days prior to being transported to the site. Also, if import fills are required (other than aggregate base), the contractor must provide appropriate documentation that the import is clean of known contamination and within acceptable corrosion limits.

Engineered fill should be placed in lifts not exceeding six inches in compacted thickness with each lift being uniformly moisture conditioned to at least the optimum moisture content and compacted to not less than 90 percent of the maximum dry density per ASTM D1557.

The upper six inches of final pavement subgrade should be uniformly compacted to at least 95 percent of the ASTM D1557 maximum dry density at a moisture content of at least the optimum moisture and must be stable under construction traffic prior to placement of aggregate base. Final pavement subgrade processing and compaction should be performed just prior to placement of aggregate base, after construction of underground utilities is complete. The moisture content of the subgrade soils must be maintained until covered by aggregate base, or the subgrade soils re-moisture conditioned just prior to base placement.

To help identify unstable pavement subgrades, a proof-roll should be performed with a fully-loaded water truck on the exposed subgrades prior to placement of aggregate base. The proof-roll should be observed by a representative of the Geotechnical Engineer.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2H:1V) and should be vegetated as soon as practical following grading to minimize erosion. As a minimum, the following erosion control measures should be considered: placement of straw bale sediment barriers or construction of silt filter fences in areas where surface run-off may be concentrated. Slopes should be over-built and cutback to design grades and inclinations. The final decision of erosion control measures should be made by the Project Stormwater Pollution Prevention Plan Engineer.

All earthwork operations should be accomplished in accordance with the recommendations contained within this report. We recommend the Geotechnical Engineer's representative be present on a regular basis during all earthwork operations to observe and test the engineered fill and to verify compliance with the recommendations of this report and the project plans and specifications.

<u>Utility Trench Backfill</u>

Utility trench backfill 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 for the pipe materials selected and applicable sections of the governing agency standards.

We recommend that native, on-site soil be used as trench backfill. Utility trench backfill should be placed in thin lifts, thoroughly moisture conditioned to at least the optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. The lift thickness will depend on the type of compaction equipment used to backfill utility trenches.

We recommend that all underground utility trenches aligned nearly parallel with new foundations be at least three feet from the outer edge of foundations, wherever possible. Trenches should not encroach into the zone extending outward at a one horizontal to one vertical (1H:1V) inclination below the bottom of foundations. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

<u>Foundations</u>

As noted previously, we anticipate seismically induced settlements of about 3½ to 4½ inches of total settlement and one inch of differential settlement over 50 feet should be anticipated. The foundation system chosen to support the proposed improvements should be designed to accommodate the calculated settlements. Based on the subsurface conditions encountered at the boring locations, the foundations may consist of shallow spread foundations with stiffened support elements on an improved subgrade, mat foundations on an improved subgrade, or a deep foundation system. In our experience, we anticipate shallow spread foundation with stiffened support elements will be the most cost-effective foundation system. Therefore, our recommendations for shallow spread foundations on an improved subgrade are provided below.

The buildings may be supported on a conventional shallow foundation system with an interior slab-on-grade lower floor, provided the building pad areas are over-excavated and constructed in accordance with the recommendations included in the Subgrade Preparation section of this report. Below we have provided recommendations for conventional shallow foundations supported on an over-excavated building pad. We have also provided preliminary recommendations for shallow foundations supported on a RAP improved subgrade. The Geotechnical Engineer should be given the opportunity to review final grading plans and foundation plans to determine if the intent of our recommendations has been properly implemented into those documents.

Conventional Shallow Foundations on Over-Excavated Building Pads

The planned 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, provided the subgrade has been prepared in accordance with the Subgrade Preparation and Engineered Fill Construction sections of this report. Lowest soil grade is defined as either the adjacent exterior soil grade or the soil subgrade beneath the building, whichever is lower. Continuous foundations should maintain a minimum width of 12 inches and isolated spread foundations should be at least 24 inches in plan dimension.

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

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

Resistance to lateral foundation displacement for conventional shallow foundations may be computed using an allowable friction factor of 0.30, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure equivalent to a fluid pressure of 300 psf per foot of depth, acting against the vertical projection of the foundation. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since full mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

We recommend that all foundation excavations be observed by the Geotechnical Engineer's representative prior to placement of reinforcement and concrete to verify firm bearing materials are exposed.

Conventional Shallow Foundations on Rammer Aggregate Piers (RAPs)

The planned buildings may also be supported on continuous and/or isolated spread foundations, or a mat foundation, supported on a RAP system extending below the bottom of foundations. The RAP system is considered capable of densifying the subsurface soils at the site and provide adequate and uniform support for the planned buildings. This will result in an increased ultimate bearing capacity and mitigation of some of the effects of total and differential settlement. A qualified RAP contractor licensed in the State of California should be contacted directly to provide final recommendations for the RAP system, including RAP depths, allowable capacities, and post-construction settlements. Upon request, we can recommend qualified contractors familiar with the local area.

Continuous and/or isolated spread foundations or a mat foundation bearing on a RAP improved subgrade should extend at least 18 inches below the lowest adjacent soil grade, provided the subgrade has been prepared in accordance with the Subgrade Preparation section of this addendum. Lowest soil grade is defined as either the adjacent exterior soil grade or the soil subgrade beneath the structure, whichever is lower. Continuous foundations should maintain a minimum width of 12 inches and isolated spread foundations should be at least 24 inches in plan dimension.

Our previous experience with RAP systems and similar soil conditions indicates the allowable bearing capacity of conventional shallow foundations constructed over a RAP system would be on the order of about 4,000 to 6,000 psf for dead plus live load condition assuming a properly installed RAP system. The RAP system layout, final bearing pressures, cell capacities and anticipated settlement will depend on the actual loading conditions for the buildings and should be determined by the RAP system designer. The final bearing pressures and cell capacities should include an appropriate factor of safety. The weight of foundation concrete extending below adjacent soil grade may be disregarded in sizing computations.

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

Preliminary resistance to lateral foundation displacement for conventional foundations supported on a RAP system may be computed using an allowable friction factor of 0.30 for soil subgrade and 0.40 for aggregate base (RAPs), 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, acting against vertical projections of the foundations. 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, effectively reducing the frictional resistance.

Interior Floor Slab Support

Interior concrete slab-on-grade floors for the proposed buildings can be supported upon the soil subgrade prepared in accordance with the recommendations in this report, provided the subgrade soils are maintained in a moist condition and protected from disturbance.

Interior concrete slab-on-grade floors for the planned buildings should be at least four inches thick. We recommend that interior floor slabs be reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The structural engineer should determine final floor slab reinforcing requirements. Temporary loads exerted during construction from vehicle traffic, construction equipment, storage of palletized construction materials, etc. should be considered in the design of the thickness and reinforcement of the interior slab-on-grade floor.

Interior floor slabs should be underlain by a layer of free-draining gravel/crushed rock, serving as a deterrent to migration of capillary moisture. The gravel/crushed rock layer should be between four and six inches thick and graded such that 100 percent passes a one-inch sieve and less than five percent passes a No. 4 sieve. Additional moisture protection may be provided by placing a plastic, water vapor retarder (at least 10-mils thick) directly over the gravel/crushed rock. The water vapor retarder should meet or exceed the minimum specifications for plastic water vapor retarders as outlined in ASTM E1745 and be installed in strict conformance with the manufacturer's recommendations.

Floor slab construction practice over the past 30 years or more has included placement of a thin layer of sand or pea gravel over the vapor retarder membrane. The intent of the sand/ pea gravel is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern of water trapped within the sand/pea gravel. As a consequence, we consider use of the sand/pea gravel layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

The recommendations presented above are intended to reduce significant soils-related cracking of slab-on-grade floors. Also important to the performance and appearance of a PCC 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 subgrades supporting exterior concrete flatwork (i.e., sidewalks, courtyards, etc.) should be brought to at least the optimum moisture content and uniformly compacted to at least 90 percent of the ASTM D1557 maximum dry density prior to the placement of the aggregate base. Exterior concrete flatwork should be at least four inches thick in pedestrian traffic areas and underlain by at least four inches of aggregate base compacted to at least 95 percent of the ASTM D1557 maximum dry density.

Proper moisture conditioning of the subgrade soils is considered important to the performance of exterior flatwork. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of the perimeter building foundation and isolated column foundations by the placement of a layer of felt material between the flatwork and the foundation.

Consideration should be given to thickening the edges of exterior flatwork to at least twice the slab thickness. Flatwork reinforcement for crack control, if desired, should be determined by the structural engineer.

Our recommendations are intended to reduce the effects of variable soil subgrade conditions in exterior concrete flatwork areas. However, some seasonal movement of exterior flatwork should be anticipated where flatwork is adjacent to landscape areas.

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

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

<u>Pavement Design</u>

The following pavement sections have been calculated based on the results of R-value testing. The procedures used for pavement design are in general conformance with Chapters 600 to 670 of the *California Highway Design Manual*, dated November 20, 2017. An R-value of 20 was used for the design of on-site pavements. The project civil engineer should determine the appropriate traffic index based on anticipated traffic conditions. We can provide alternate pavement sections based on different traffic indices, upon request.

Table 5
Pavement Design Alternatives (R-value = 25)

Traffic Index (TI)	Traffic Condition/Street Classification	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Light Automobile Parking	2½*	7	--
		--	4	4
6.5	Emergency Vehicle Traffic	2½	11	--
		3½*	9	
		--	6	5

* = Asphalt thickness includes Caltrans Factor of Safety.

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using the PCC pavements in areas subjected to concentrated heavy wheel loading, such as truck turning areas and in front of trash enclosures. These PCC pavements should be at least four inches thick, supported on at least four inches of compacted Class 2 aggregate base as noted in Table 5 above.

We emphasize that the performance of pavements is critically dependent upon uniform and adequate compaction of the soil subgrade, as well as all engineered fill and utility trench backfill within the limits of the pavements. We recommend that pavement subgrade preparation (i.e. scarification, moisture conditioning and compaction) be performed after underground utility construction is completed and just prior to aggregate base placement. The upper six inches of pavement subgrade soils should be compacted to at least 95 percent relative compaction at the optimum moisture content. All aggregate base should be compacted to at least 95 percent of the ASTM D1557 maximum dry density.

We suggest the concrete slabs be constructed with thickened edges in accordance with ACI design standards. Reinforcing for crack control, if desired, should consist of No. 4 reinforcing bars placed on maximum 24-inch centers each way throughout the slab. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform with the current PCA or ACI guidelines. Portland cement concrete should achieve a minimum compressive strength of 3500 pounds per square inch at 28 days.

Pavement subgrades must be stable and unyielding under heavy wheel loads of construction equipment. A proof-roll test using a fully loaded water truck should be performed prior to placement of aggregate base to help identify areas that are unstable, as observed by our representative. Areas that are found to be unstable should be excavated to firm, undisturbed materials and restored to grade with compacted aggregate base.

Materials quality and construction within the structural section of the pavement should conform to the applicable provisions of the latest edition of the Caltrans Standard Specifications.

<u>Site Drainage</u>

Final site grading should be accomplished to provide positive drainage of surface water away from the planned improvements and prevent ponding of water adjacent to foundations, slabs or pavements. The subgrade adjacent to the planned buildings should be sloped away from foundations at a minimum two percent gradient for at least five feet, where possible.

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

Geotechnical Engineering Observation and Testing During Construction

Site preparation should be accomplished in accordance with the recommendations of this report. 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

REGISTERED PROFESSIONAL ENGINEER
MATTHEW S. MOYNEUR
NO. 2920
Exp. 6/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

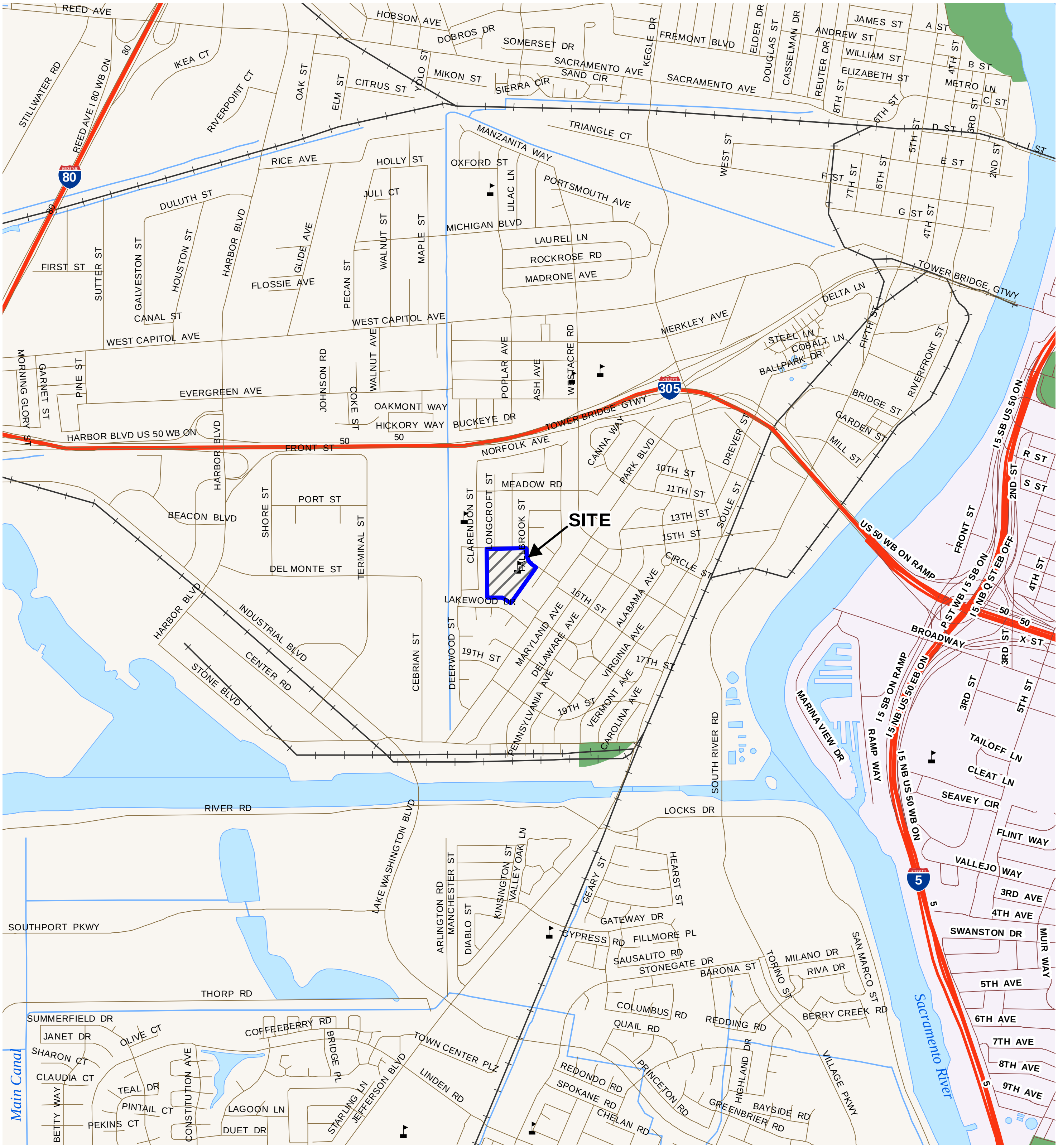
Matthew S. Moyneur
Senior Engineer

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

REGISTERED PROFESSIONAL ENGINEER
DAVID R. GIUS, JR.
NO. 2318
Exp. 9/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

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

Joseph R. Ybarra
Staff Geologist



Street data courtesy of Yolo County, 2017.
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

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VICINITY MAP

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

Aerial provided by ESRI.
Site Plan adapted from a drawing provided by BCA Architects, dated 02/11/19.
Projection: NAD 83, California State Plane, Zone II

Legend

- Approximate Site Boundary
- ✦ Approximate Soil Boring Location
- ⊕ Approximate CPT Location
- Approximate Cross Section Line
- ● Approximate Previous Soil Boring Location (WKA No. 7874.01, 12/07)
- ■ Approximate Previous CPT Location (WKA No. 7874.01, 12/07)

N

0 100 200
Feet

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SITE PLAN

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

Topographic Map adapted from the Sacramento West Quadrangle, California, dated 2018.
Projection: NAD 83, California State Plane, Zone II

Contour intervals at 10 feet.

Legend

—10— Contour Line

N

0 1,000 2,000
Feet

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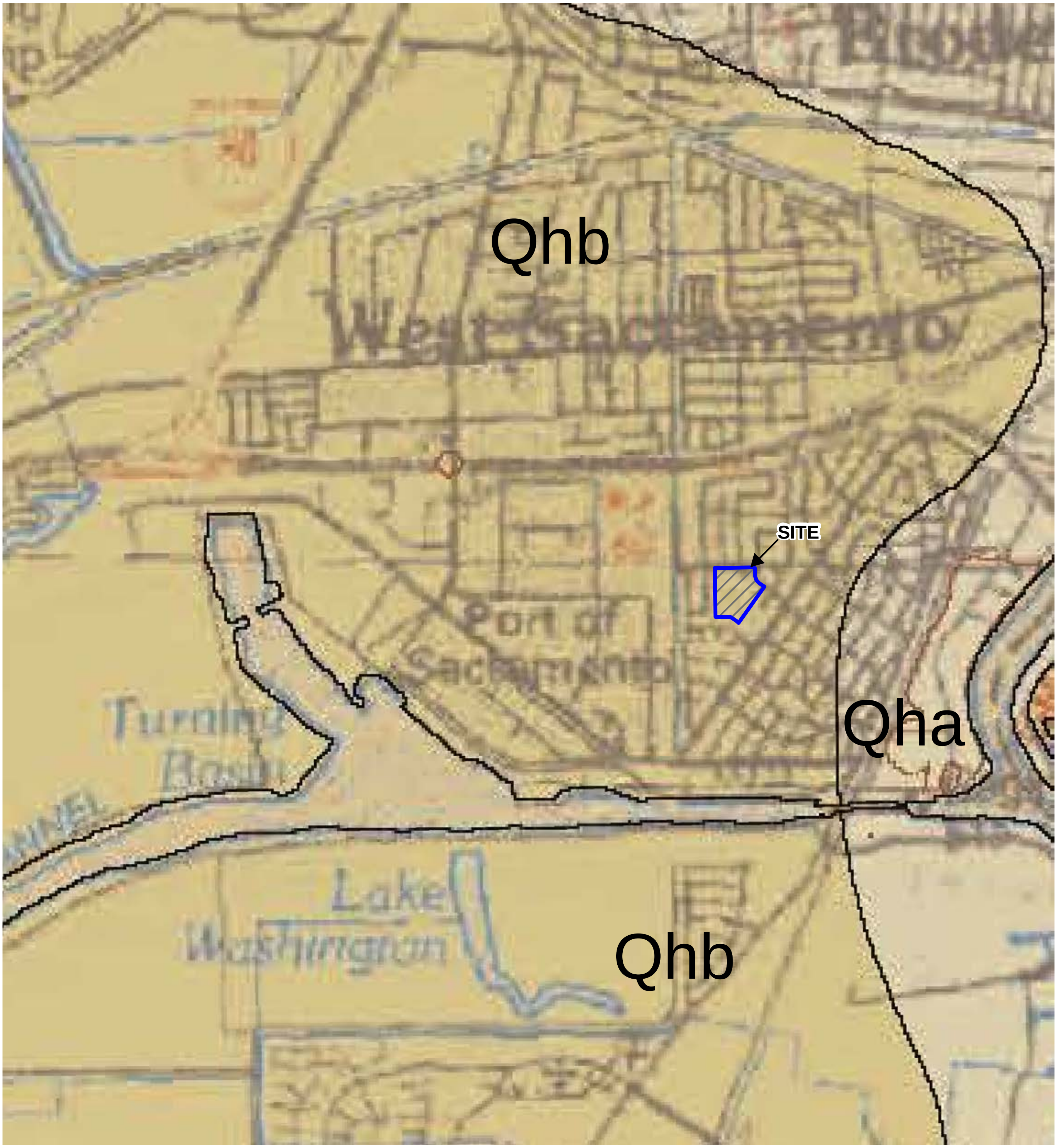
TOPOGRAPHIC MAP

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	3
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	



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

Legend

Qha	Holocene alluvium
Qhb	Holocene basin deposits

N

0 1,000 2,000
Feet

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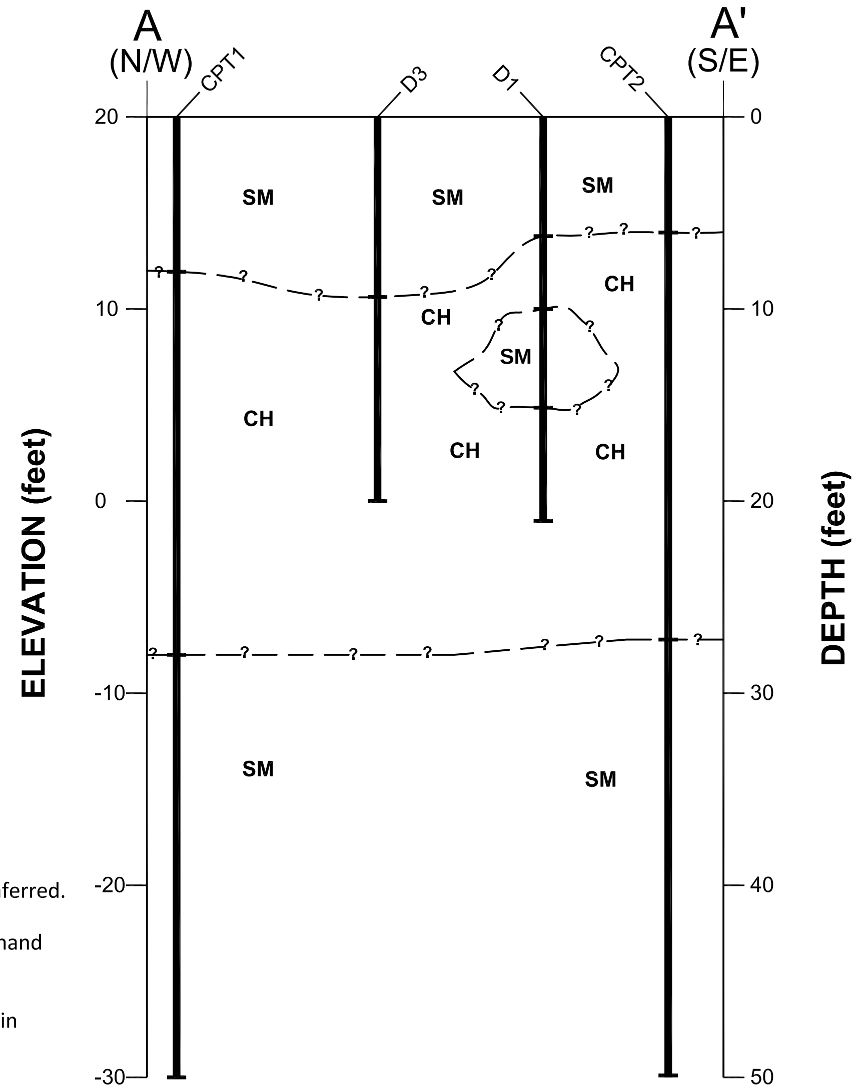
GEOLOGIC MAP

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	4
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19

WKA NO. 12277.01P



Key:

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

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

* Detailed soil logs and descriptions available in Boring Logs, Figures 8 through 13.

(XX) * Elevations, Depths, and Locations are approximate.

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

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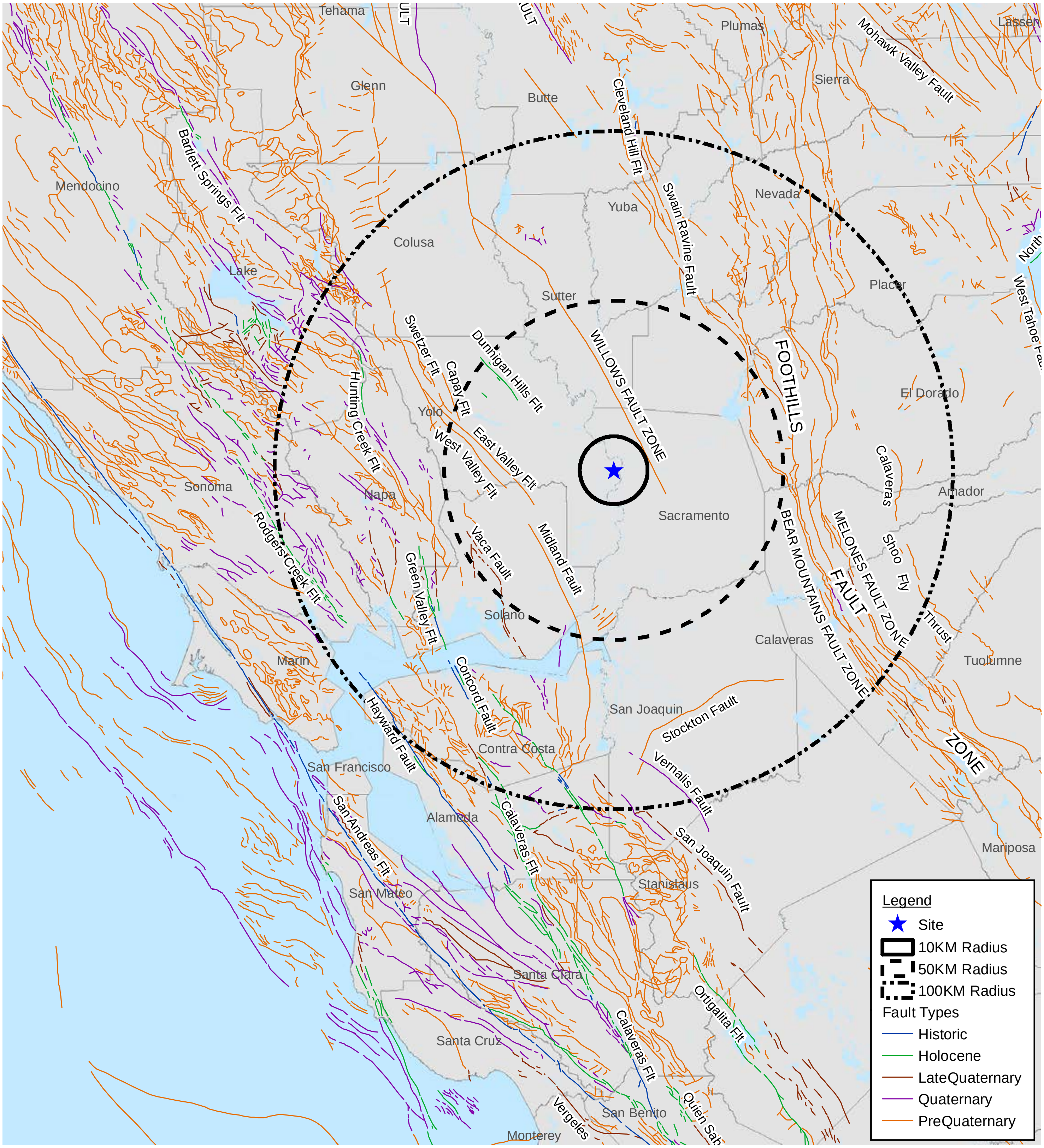
GEOLOGIC CROSS SECTION A-A'

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	5
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19

WKA NO.12277.01P



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

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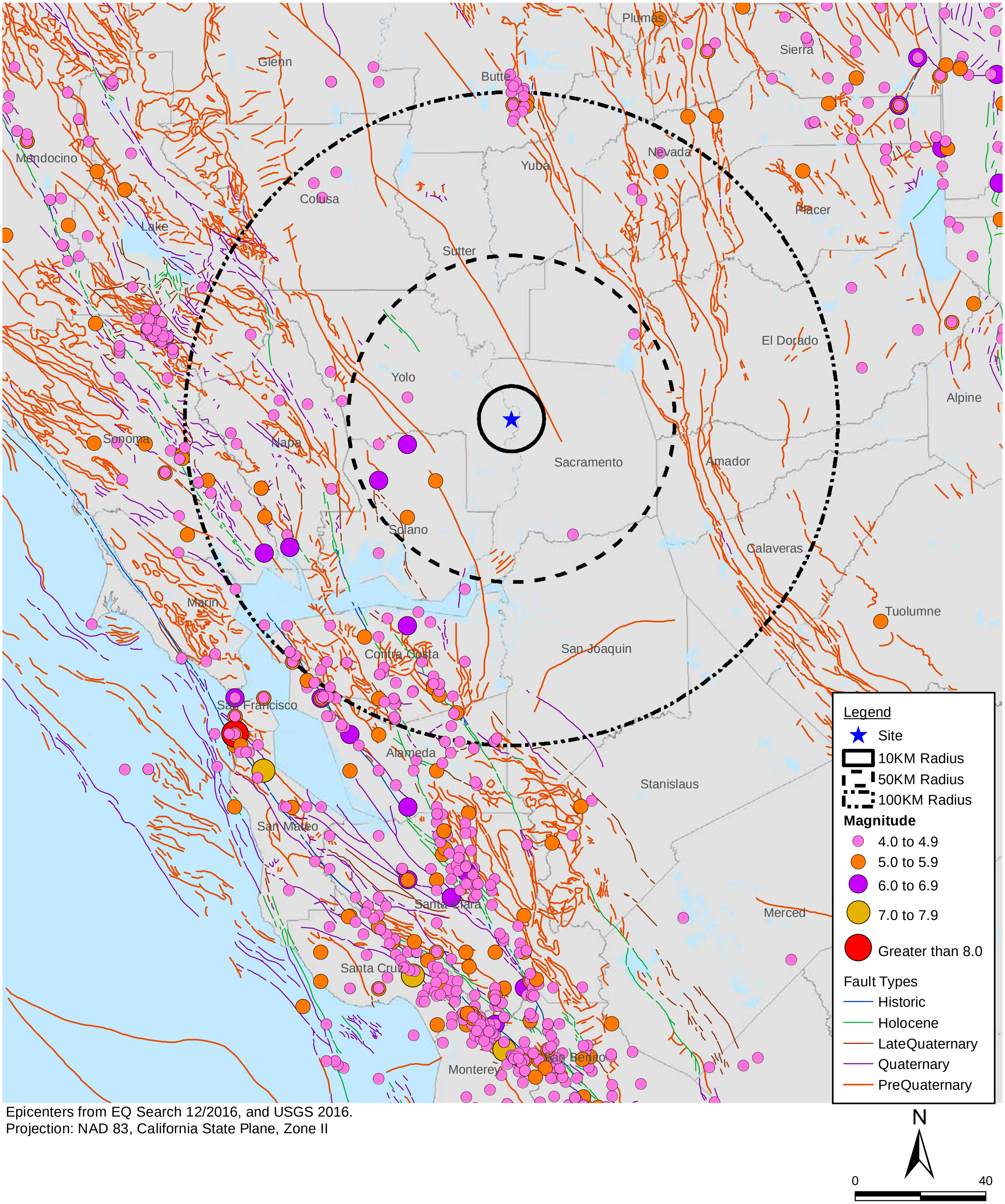
FAULT MAP

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	6
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19

WKA NO. 12277.01P



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

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EPICENTER MAP

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	7
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

Project: Westmore Oaks Modernization	LOG OF SOIL BORING D1
Project Location: West Sacramento, California	Sheet 1 of 1
WKA Number: 12277.01P	

Date(s) Drilled	3/12/19	Logged By	JRY	Checked By	MSM
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	21.5 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	11.0	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks	Bulk (0-3')			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0	SM	Brown, moist, loose, silty fine SAND (SM)						
				■	D1-1I	8	17.9	95	TR
	5			■	D1-2I	2	39.6	75	
		CH	Brown, moist, very soft, fat CLAY (CH)						
	10	SM	Gray brown, moist to wet, loose, silty fine to medium SAND (SM)	■	D1-3I	3			
	15	CH	Dark brown, wet, stiff, fat CLAY (CH)	■	D1-4I	11			
	20		brown	■	D1-5I	14			

Boring terminated at 21 1/2 feet below existing site grade
Groundwater was encountered initialy at 12 feet below site grade

BORING LOG 12277.01P - WESTMORE OAKS MODERNIZATION.GPJ WKA.GDT 4/2/19 9:06 AM

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FIGURE 8

Project: Westmore Oaks Modernization Project Location: West Sacramento, California WKA Number: 12277.01P	**LOG OF SOIL BORING D2** Sheet 1 of 1

Date(s) Drilled	**3/12/19**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**20.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**8.0**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk (0-3'), PI, EI, RV**			Driving Method and Drop	**140lb auto. hammer with 30" 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	SM	Yellowish brown, moist to wet, loose, silty fine SAND (SM)						
				■	D2-1I	5	27.8	83	
				■	D2-2I	5	14.4	83	
	5								
				■	D2-3I	5			
		CH	Dark brown with brown mottling, wet, medium stiff, fat CLAY (CH)						
	10								
				■	D2-4I	7			
	15								
			Brown with dark brown mottling, very stiff	■	D2-5I	19			
	20								

Boring terminated at 20 feet below existing site grade
Groundwater was encountered initialy at 8 1/2 feet below site grade

BORING LOG 12277.01P - WESTMORE OAKS MODERNIZATION.GPJ WKA.GDT 4/2/19 9:06 AM

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FIGURE 9

Project: Westmore Oaks Modernization
Project Location: West Sacramento, California
WKA Number: 12277.01P

LOG OF SOIL BORING D3

Date(s) Drilled	3/12/19	Logged By	JRY	Checked By	MSM
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	20.0 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	7.0	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks	Bulk (0-3')			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0	Yellowish brown, moist to wet, loose, silty fine SAND (SM)					
			D3-1I	7	6.6	95	
		very loose	D3-2I	4	10.3	76	
	5						
	8.5	Dark brown with brown mottling, wet, soft, fat CLAY (CH)	D3-3I	3			PI
	10						
		stiff	D3-4I	9			
	15						
		brown with dark brown mottling, very stiff	D3-5I	14			
	20						

Boring terminated at 20 feet below existing site grade
Groundwater was encountered initialy at 7 feet below site grade

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FIGURE 10

Project: Westmore Oaks Modernization	LOG OF SOIL BORING D4
Project Location: West Sacramento, California	
WKA Number: 12277.01P	Sheet 1 of 1

Date(s) Drilled	3/12/19	Logged By	JRY	Checked By	MSM
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	21.5 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	9.0	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks	Bulk (0-3')			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0	SM	Yellowish brown, moist, very loose, silty fine SAND (SM)						
				■	D4-1I	4	17.6	82	
	5			■	D4-2I	3	27.5	90	
		CH	Olive brown with orange mottling, moist, soft, fat CLAY (CH)						
	10		dark brown, wet, sandy	■	D4-3I	3			
	15		no sand	■	D4-4I	11			
	20		brown with dark brown mottling, stiff	■	D4-5I	13			
			Boring terminated at 21 1/2 feet below existing site grade Groundwater was encountered initialy at 10 feet below site grade						

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FIGURE 11

Project: Westmore Oaks Modernization **Project Location: West Sacramento, California** **WKA Number: 12277.01P**	**LOG OF SOIL BORING D5** Sheet 1 of 1

Date(s) Drilled	**3/12/19**	Logged By	**JRY**	Checked By	**MSM**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**21.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**12.0**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk (0-3')**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Light brown, moist, loose, silty fine SAND (SM)						
	5			■	D5-1I	5			
	10		Dark brown with brown mottling, moist to wet, very soft, fat CLAY (CH)	■	D5-2I	2			
	15		Brown to dark brown, wet, loose, fine SAND (SP)	■	D5-3I	5			
			Dark brown, wet, medium stiff, lean CLAY (CL)						
	20		Dark brown, wet, medium dense, fine SAND (SP)	■	D5-4I	17			
			Dark brown with brown mottling, wet, very stiff, lean CLAY (CL)						

Boring terminated at 21 1/2 feet below existing site grade
Groundwater was encountered initialy at 13 feet below site grade

BORING LOG 12277.01P - WESTMORE OAKS MODERNIZATION.GPJ WKA.GDT 4/2/19 9:06 AM

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FIGURE 12

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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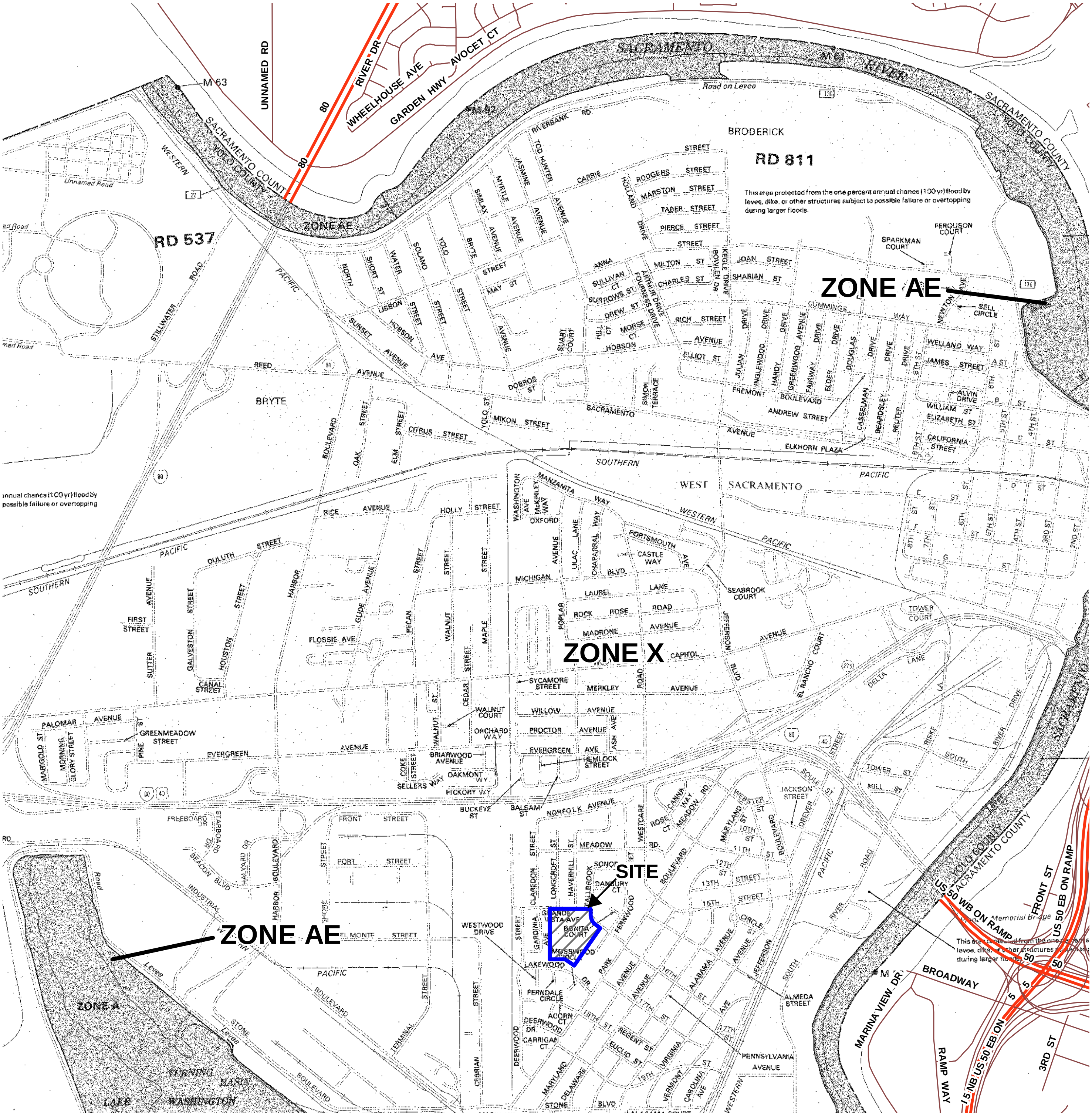
UNIFIED SOIL CLASSIFICATION SYSTEM

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	13
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19

WKA NO. 12277.01P



Flood Map provided by FEMA, panel 5 of 10.
Map number 0607280005B dated January 19, 1995.
Projection: NAD 83, California State Plane, Zone II

Legend

Zone A	No base flood elevations determined
Zone AE	Base flood elevations determined
Zone X	Areas of 500-year flood; areas of 100-year flood with average depths of less that 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood.

N

0 1,000 2,000
Feet

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FEMA FLOOD MAP

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	14
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

APPENDICES

APPENDIX A
General Project Information, Laboratory Testing and Results

APPENDIX A
WKA No. 12277.01P

A. GENERAL INFORMATION

The performance of a geotechnical engineering and geologic hazards study for the proposed Westmore Oaks Modernization project located at 1504 Fallbrook Street in West Sacramento, California, was verbally authorized by our client, Washington Unified School District, on March 1, 2019. Authorization was for a geotechnical engineering and geologic hazards study as described in our proposal letter dated February 18, 2019 and sent to our client, Washington Unified School District, whose mailing address is 930 Westacre Road, in West Sacramento, California 95691; telephone (916) 375-7604.

In performing this study, we made reference to a *Conceptual Site Plan* drawn by the project architect, BCA Architects, whose mailing address is 980 9th Street, Suite 2050, in Sacramento, California 95814; telephone (916) 254-5600.

B. FIELD EXPLORATIONS

As part of our study, the field exploration program included the advancement of two cone penetrometer test (CPT) soundings (CPT1 and CPT2) and the drilling and sampling of five borings (D1 through D5) at the approximate locations shown on Figure 2.

CPT soundings CPT1 and CPT2 were advanced at the site on March 8, 2019, utilizing a 25-ton, truck-mounted CPT rig provided by Gregg Drilling, LLC of Martinez, California. The CPT's consisted of advancing a 15-square-centimeter cone penetrometer at a rate of about one inch per second to a depth of about 50 feet below existing site grades. Data was collected from the cone penetrometer at an approximate depth interval of two inches.

Borings D1 through D5 were drilled at the site on March 12, 2019, utilizing a CME-55 HT truck-mounted drill rig equipped with six-inch diameter, solid stem augers, provided by V&W Drilling, Inc. of Galt, California. The borings were drilled to depths ranging from about 20 to 21½ feet below existing site grades. At various intervals relatively undisturbed soil samples were recovered with a 2½-inch outside diameter (O.D.), 2-inch inside diameter (I.D.), modified California split-spoon sampler driven by a 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each six-inch interval was recorded. The sum of the blows required to drive the sampler the lower 12-inch interval, or a portion thereof, is designated as the penetration resistance or "blow count" for that particular drive. The samples were retained in two-inch diameter by six-inch long thin-walled brass tubes contained within the sampler. After recovery, the soils in the tubes were visually classified by the field representative and the ends of the tubes were sealed to preserve the natural moisture contents.

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

The Logs of Soil Borings, Figures 8 through 12, contain descriptions of the soils encountered at each boring location. A legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 13.

Copies of the reports for CPT1 and CPT2 provided by Gregg Drilling, LLC are included in Appendix C.

C. LABORATORY TESTING

Selected undisturbed soil samples were tested to determine dry unit weight (ASTM D2937) and natural moisture content (ASTM D2216). The results of these tests are included on the boring logs at the depth each tested sample was obtained.

A sample of the near-surface soil was tested for Triaxial Shear Strength testing (ASTM D4767). The results of this test are presented in Figure A1.

Two samples of near-surface soil were subjected to Atterberg Limits tests (ASTM D4318). The results of these tests are presented in Figure A2.

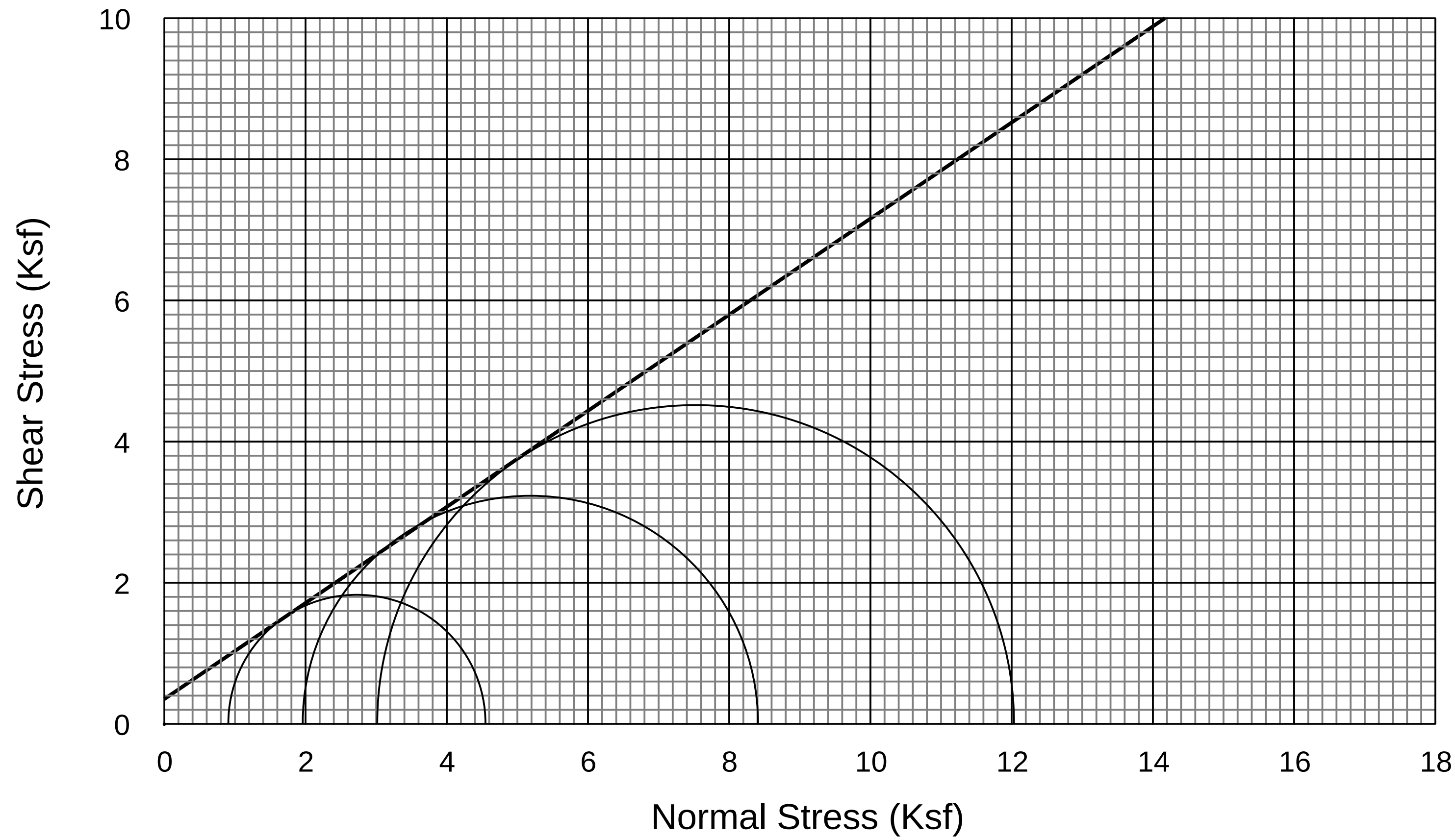
A sample of the near-surface soil was tested for Expansion Index (ASTM D4829). The results of this test are presented in Figure A3.

A bulk sample of near-surface soil was subjected to Resistance-value ("R") 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 A4.

A sample of near-surface soil was 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 A5 and A6.

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. : D1-1I

SAMPLE CONDITION : Undisturbed

SAMPLE DESCRIPTION : Brown, fine sand

DRY DENSITY (PCF) : 95.0
INITIAL MOISTURE (%) : 17.9
FINAL MOISTURE (%) : 28.5

ANGLE OF INTERNAL FRICTION (ø) : 34°
COHESION (PSF) : 357

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TRIAXIAL COMPRESSION TEST RESULTS

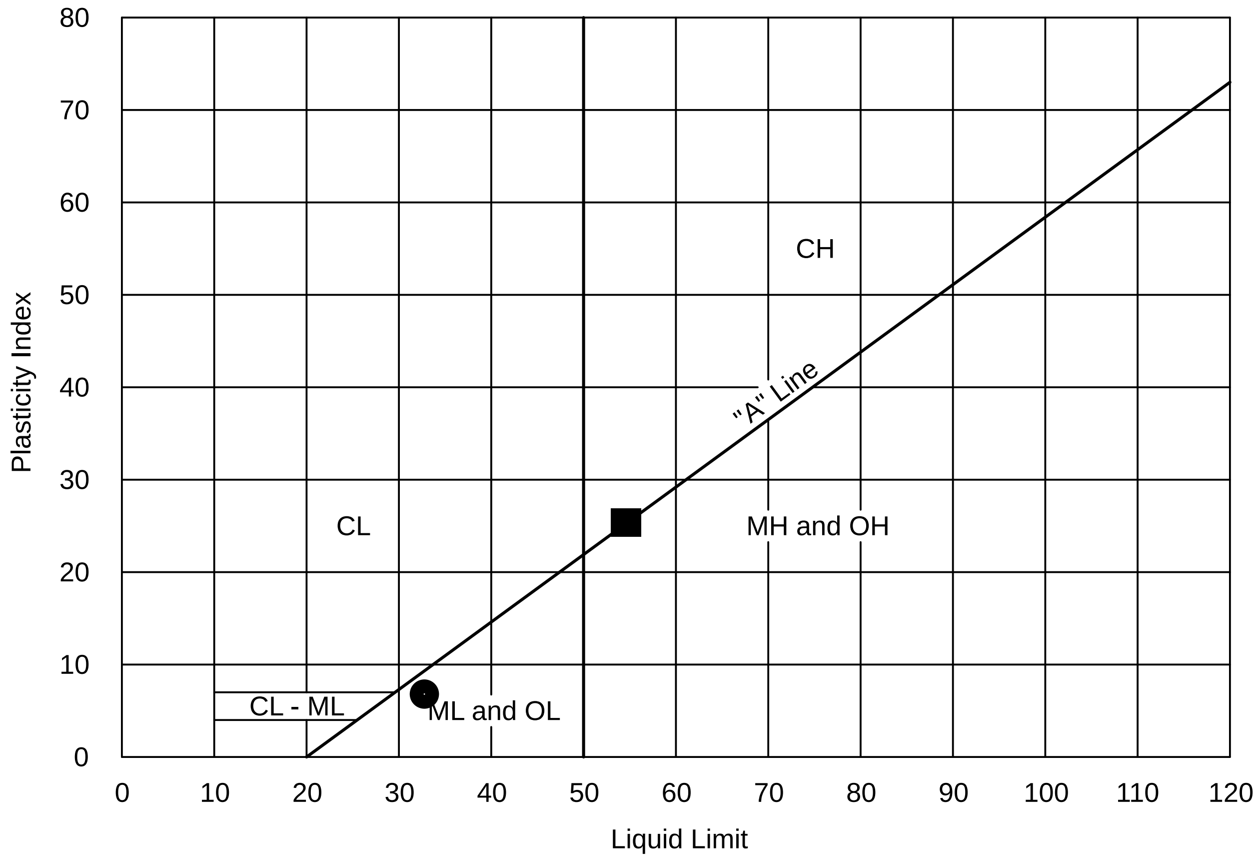
WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

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 (%)		
●	D2	0'-3.0'	---	33	8	---	ML
■	D3-3I	9.5'-10.0'	---	54	25	---	CH-MH

ATTERBERG LIMITS

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19

WKA NO.12277.01P

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Yellowish brown, silty fine sand

LOCATION: D2

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 3'	12.1	23.6	103	**35**

CLASSIFICATION OF EXPANSIVE SOIL *

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

* From ASTM D4829, Table 1

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EXPANSION INDEX

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	A3
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO.12277.01P	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Yellowish brown, silty fine sand (SM)

LOCATION: D2 (0' - 3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	115	14.7	476	53	229	65
2	111	15.5	154	11	48	8
3	113	15.2	254	22	95	16

R-Value at 300 psi exudation pressure = **25**

WallaceKuhl & ASSOCIATES

RESISTANCE VALUE TEST RESULTS

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

Sunland Analytical

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

Date Reported 03/20/2019
Date Submitted 03/14/2019

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 : 12277.01P Site ID : D4@0-3FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 79157-165364.

EVALUATION FOR SOIL CORROSION

Soil pH	6.82		
Minimum Resistivity	6.77 ohm-cm (x1000)		
Chloride	2.0 ppm	00.00020	%
Sulfate	4.4 ppm	00.00044	%

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

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

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

Date Reported 03/20/2019
Date Submitted 03/14/2019

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 : 12277.01P Site ID : D4@0-3FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 79157-165365.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	5.7	mg/kg

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

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

WESTMORE OAKS MODERNIZATION

West Sacramento, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	03/19
WKA NO. 12277.01P	

APPENDIX B

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APPENDIX B - REFERENCES

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APPENDIX C

CPT Logs and Liquefaction Analysis Results

GREGG DRILLING, INC.
www.greggdrilling.com

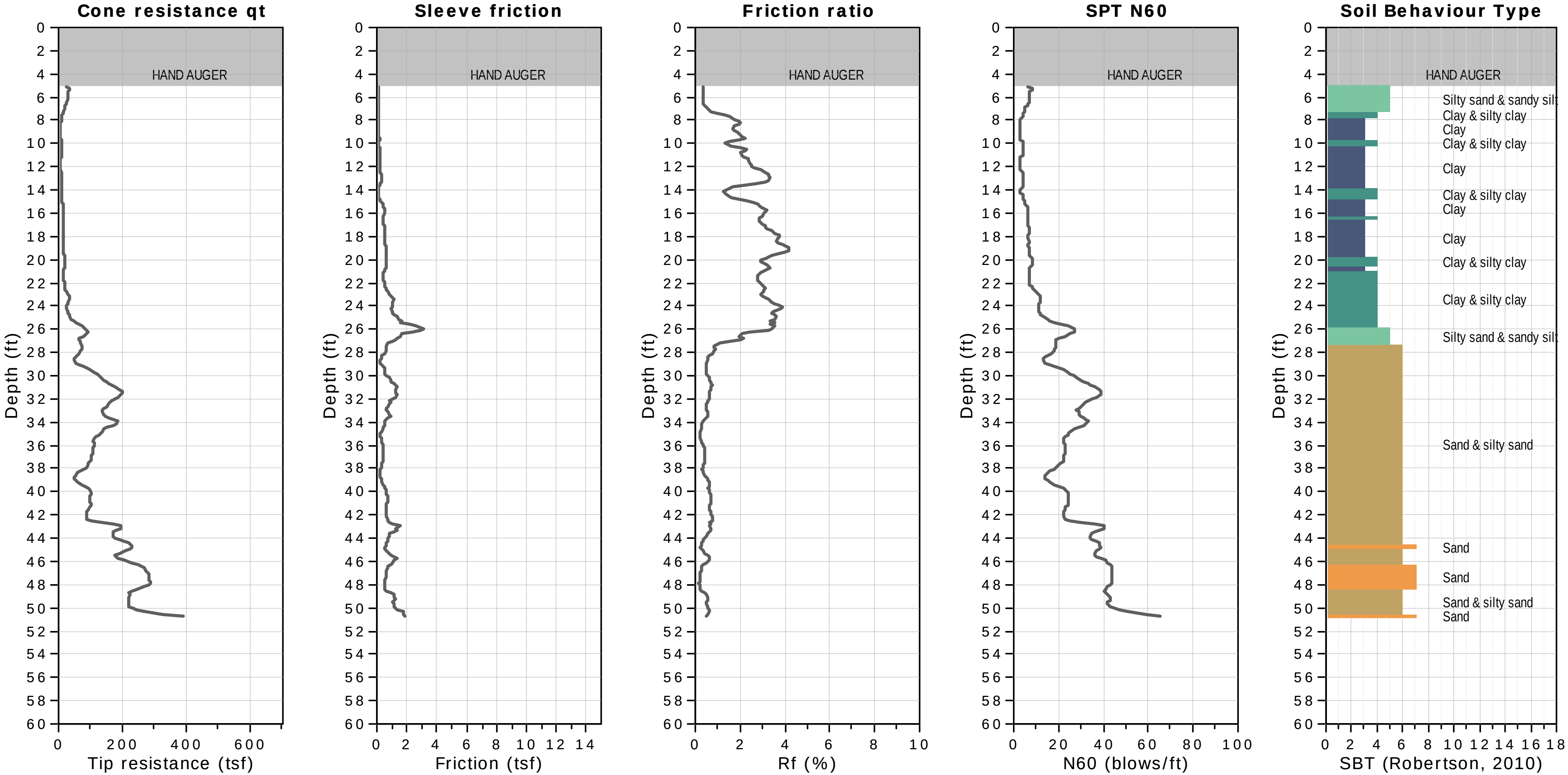
CPT: CPT-01

CLIENT: WALLACE-KUHL AND ASSOCIATES

SITE: WESTMORE OAKS - 1504 FALLBROOK STREET, WEST SACRAMENTO, CA

Field Rep: JOSEPH YBARRA

Total depth: 50.69 ft, Date: 3/8/2019



SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GREGG DRILLING, INC.
www.greggdrilling.com

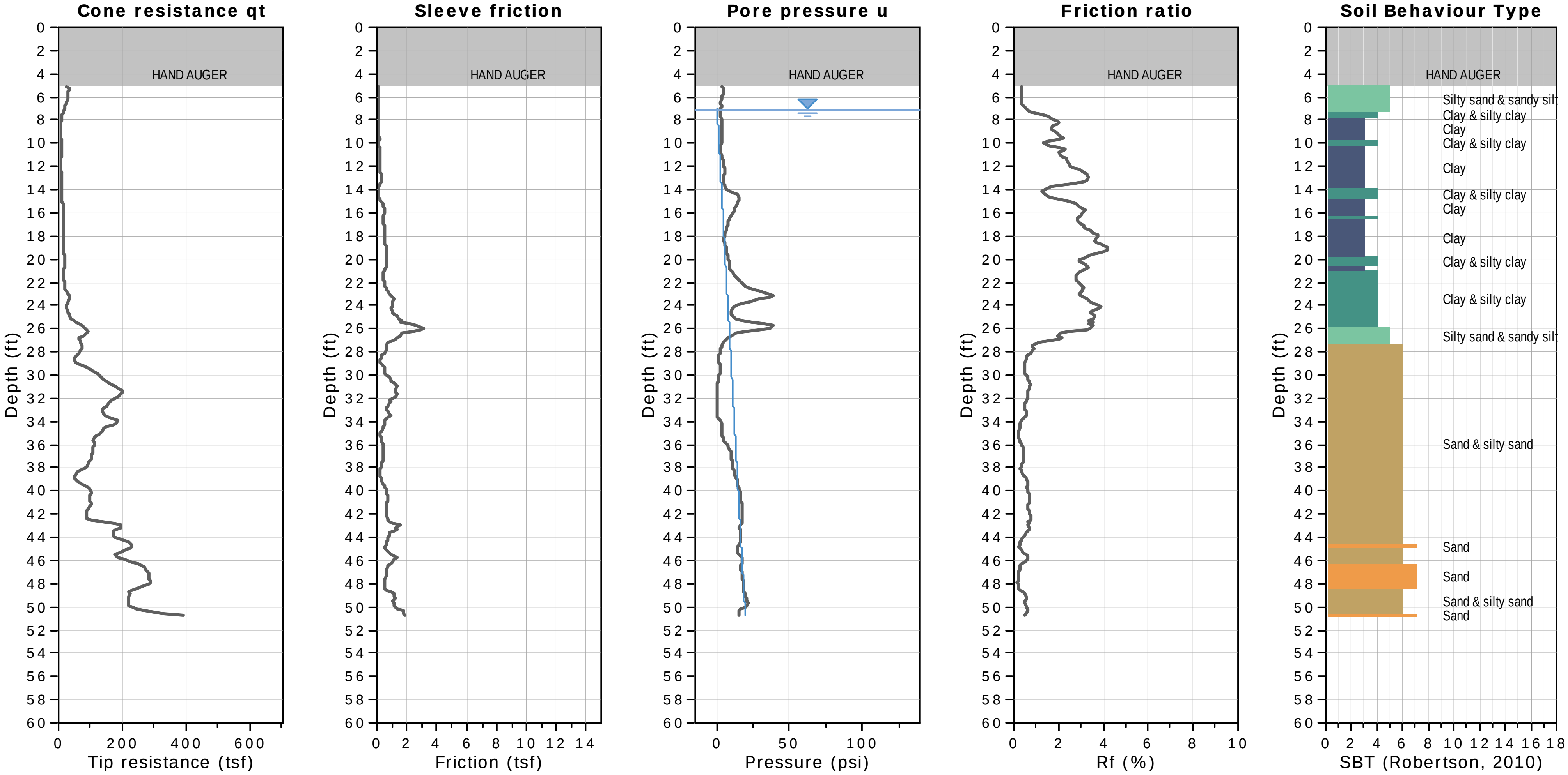
CPT: CPT-01

CLIENT: WALLACE-KUHL AND ASSOCIATES

SITE: WESTMORE OAKS - 1504 FALLBROOK STREET, WEST SACRAMENTO, CA

Field Rep: JOSEPH YBARRA

Total depth: 50.69 ft, Date: 3/8/2019



SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

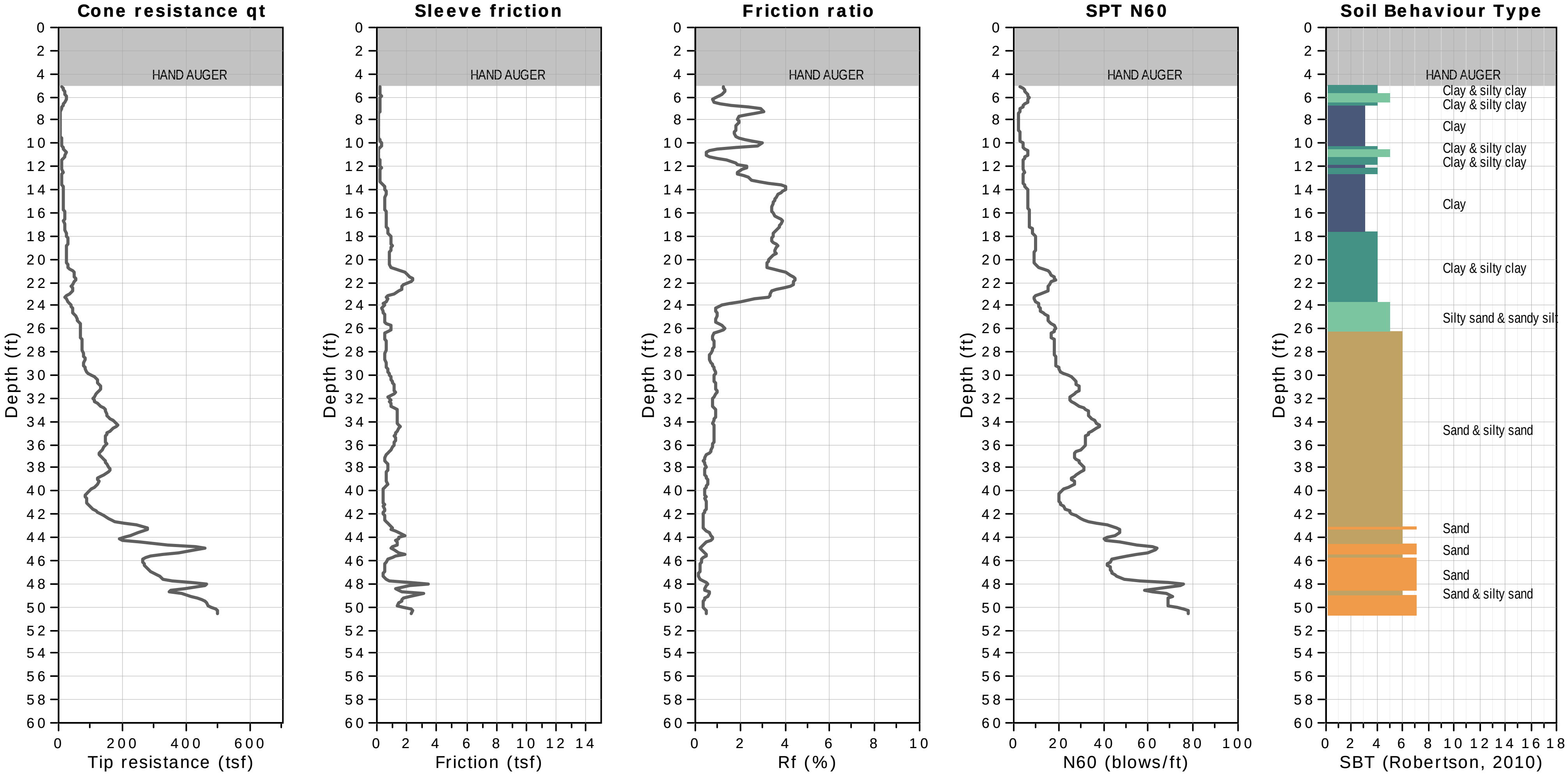
CPT: CPT-02

CLIENT: WALLACE-KUHL AND ASSOCIATES

SITE: WESTMORE OAKS - 1504 FALLBROOK STREET, WEST SACRAMENTO, CA

Field Rep: JOSEPH YBARRA

Total depth: 50.52 ft, Date: 3/8/2019



SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GREGG DRILLING, INC.
www.greggdrilling.com

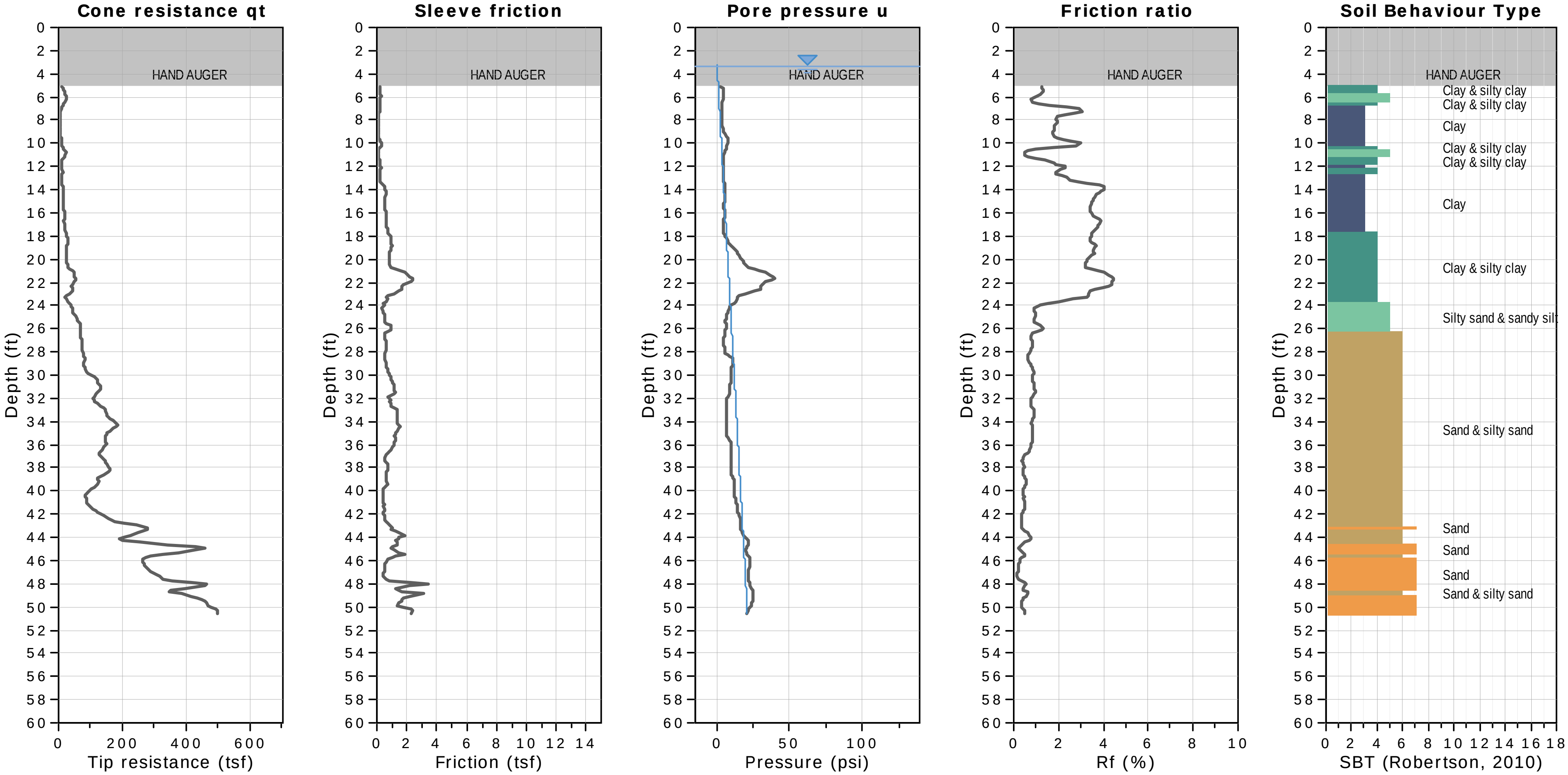
CPT: CPT-02

CLIENT: WALLACE-KUHL AND ASSOCIATES

SITE: WESTMORE OAKS - 1504 FALLBROOK STREET, WEST SACRAMENTO, CA

Field Rep: JOSEPH YBARRA

Total depth: 50.52 ft, Date: 3/8/2019



SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

WallaceKuhl & ASSOCIATES

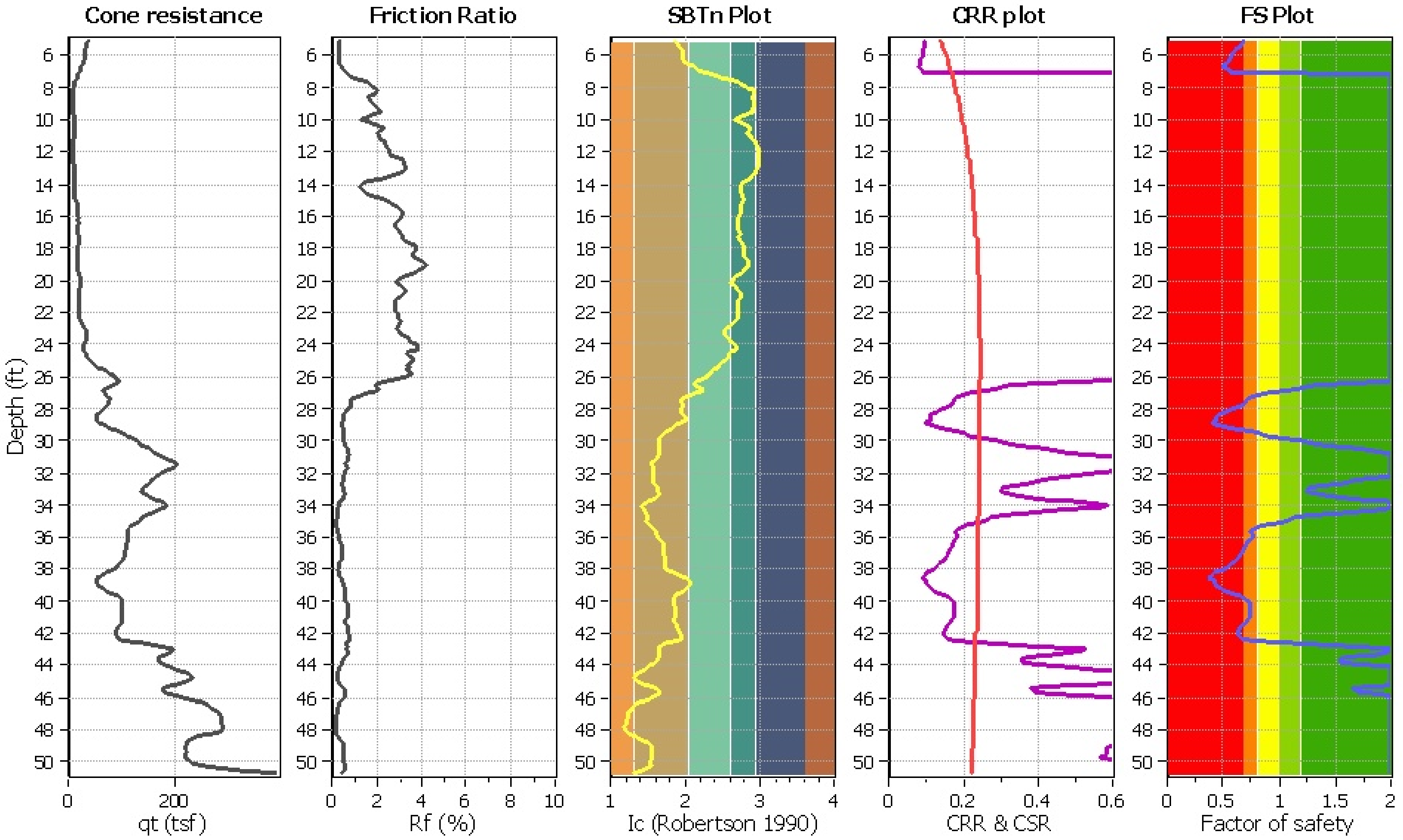
LIQUEFACTION ANALYSIS REPORT

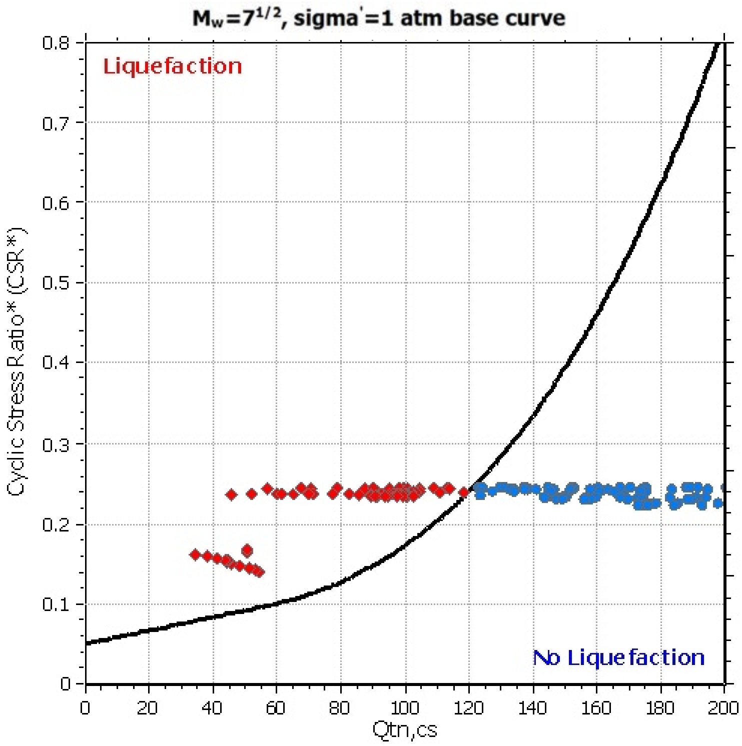
Project title : 12277.01P - Westmore Oaks **Location : West Sacramento, CA**

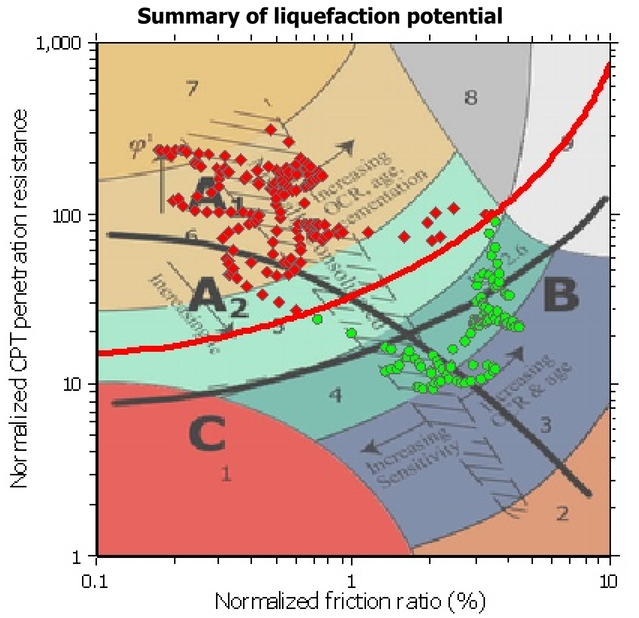
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	6.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.30	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.32	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

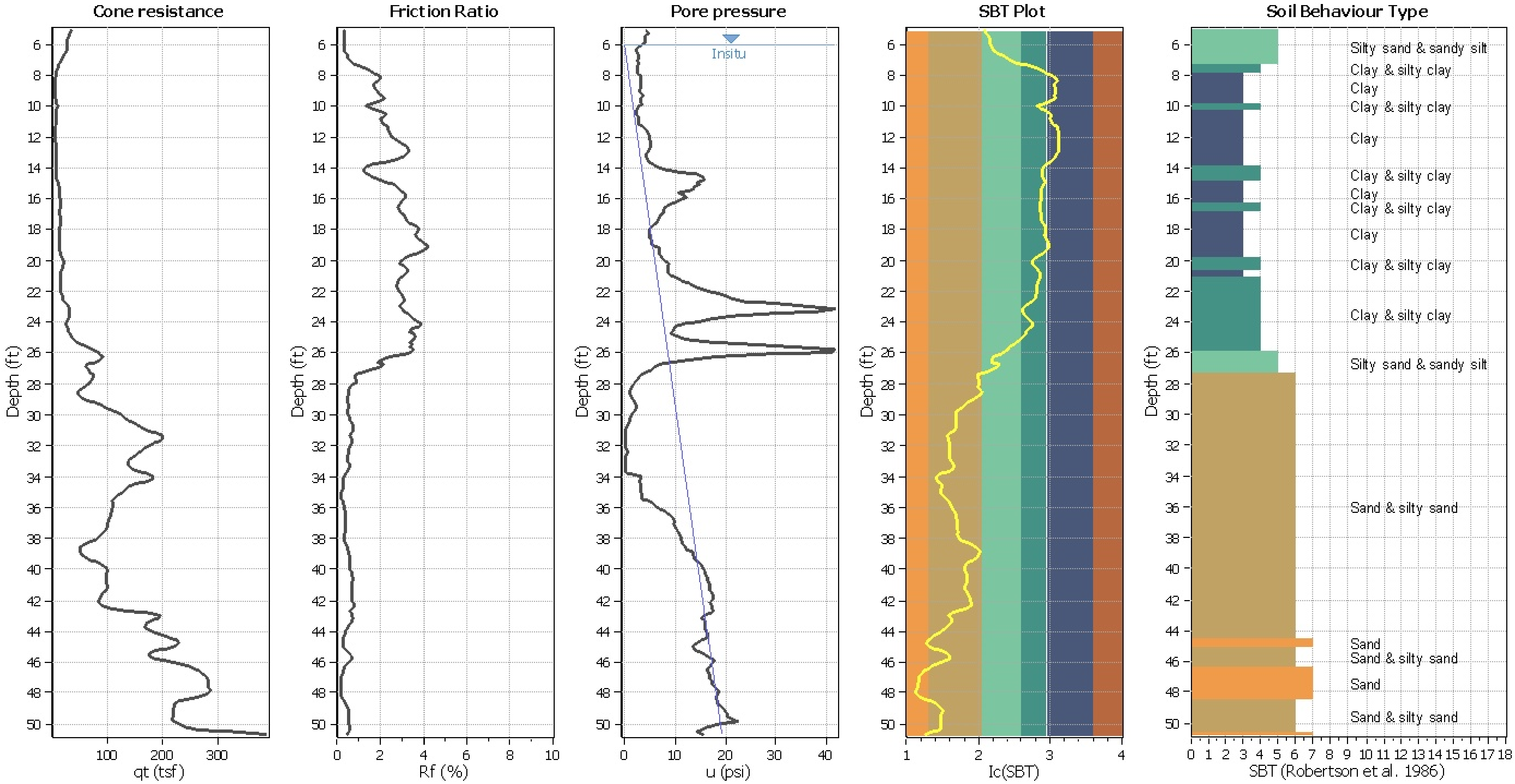






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plc



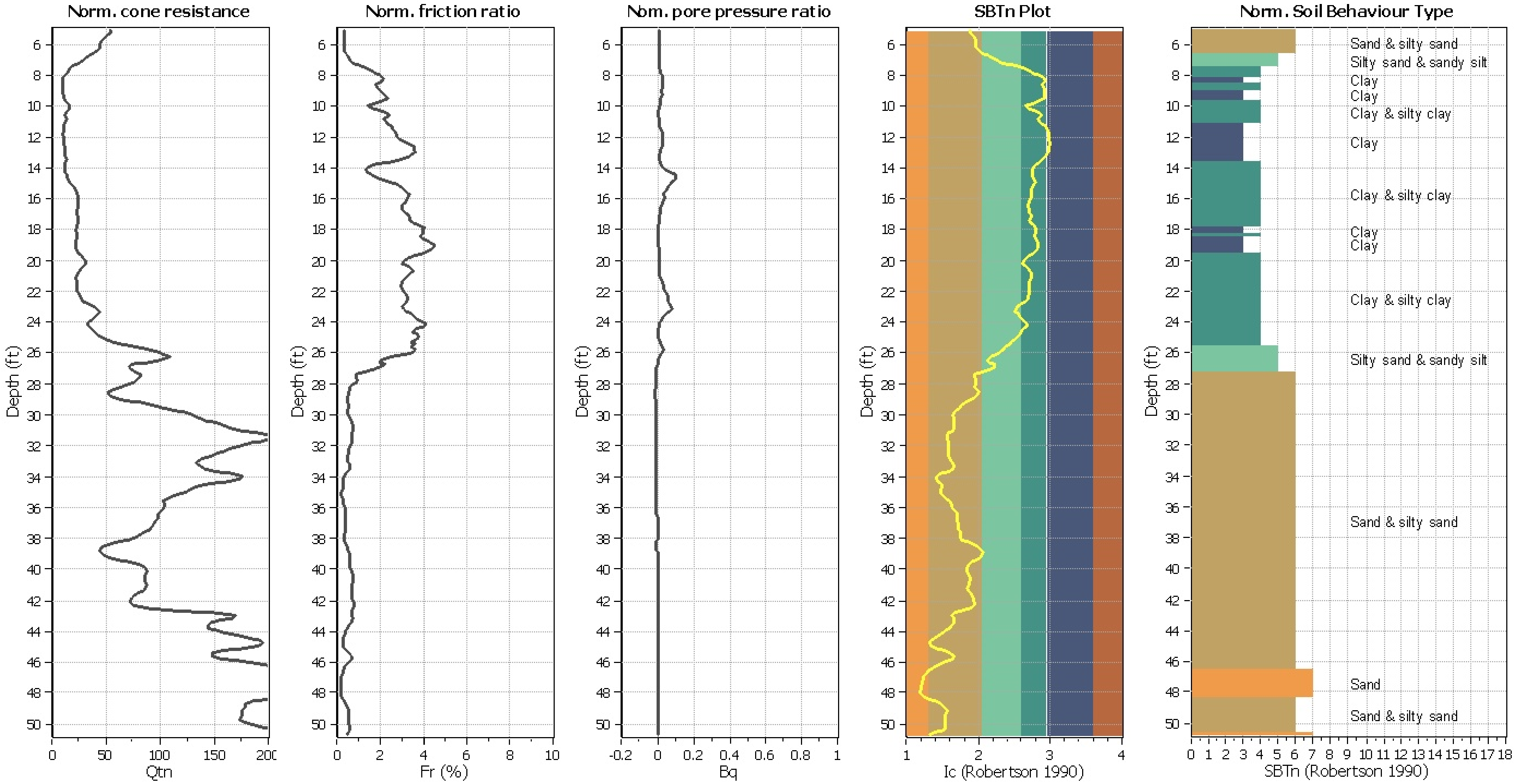
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normaliz



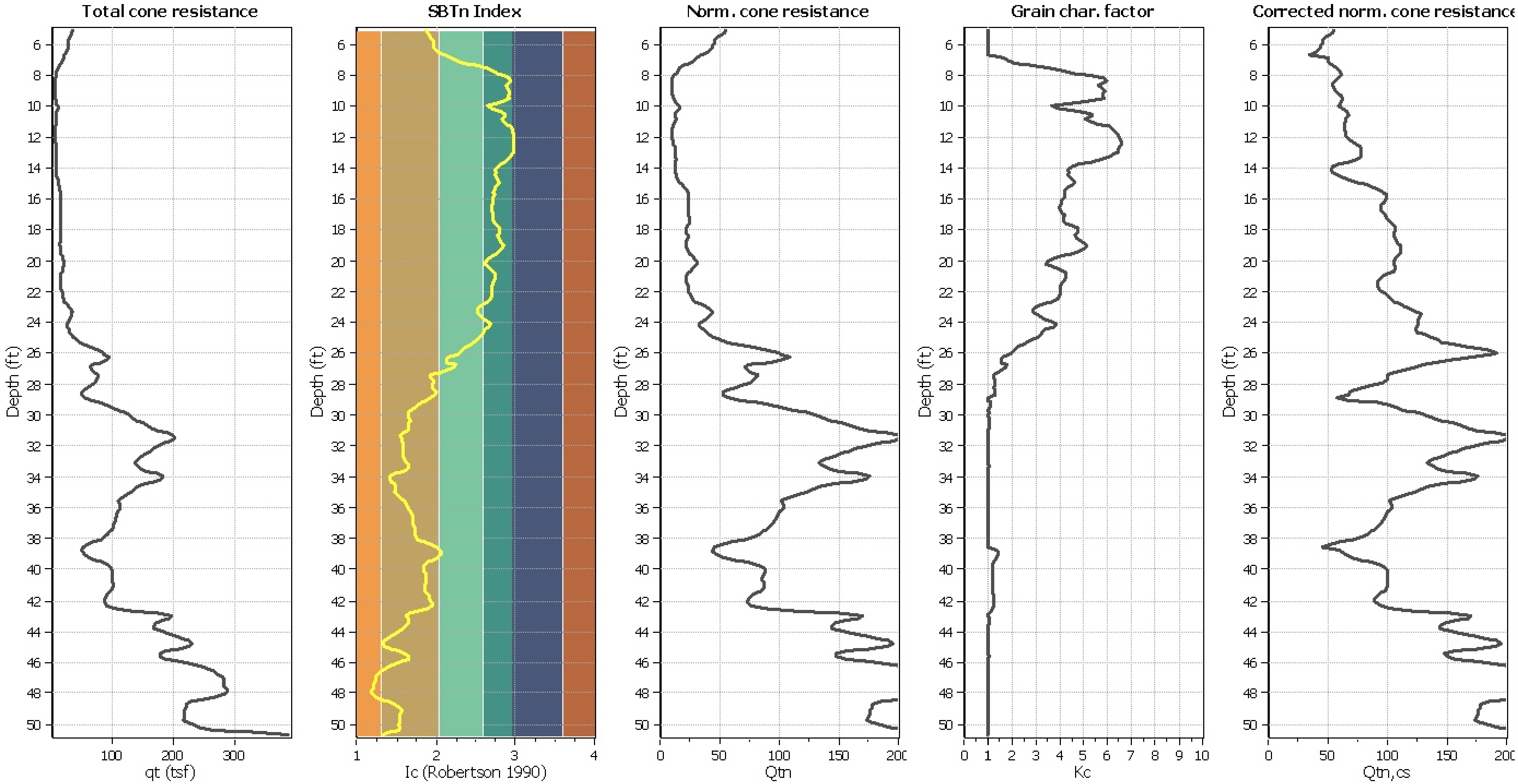
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

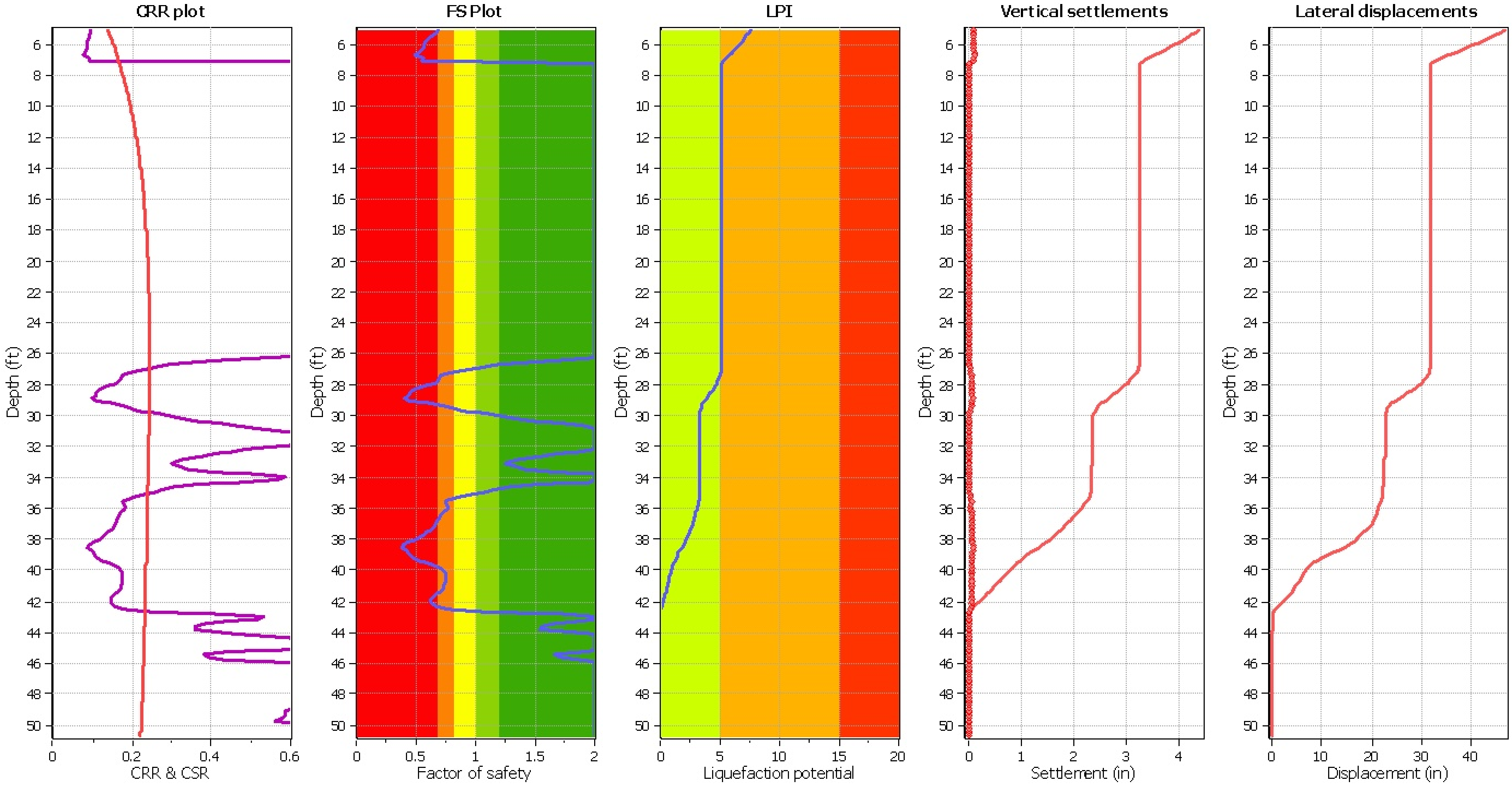
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

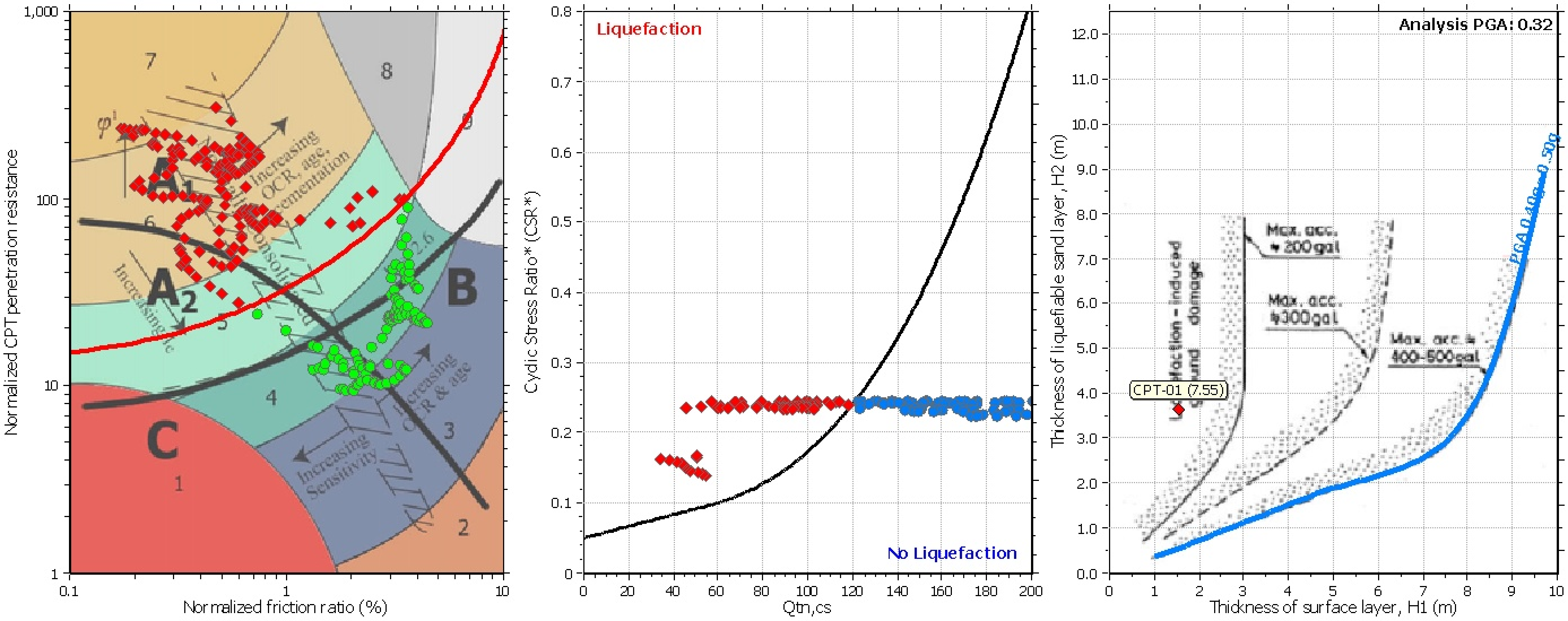
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

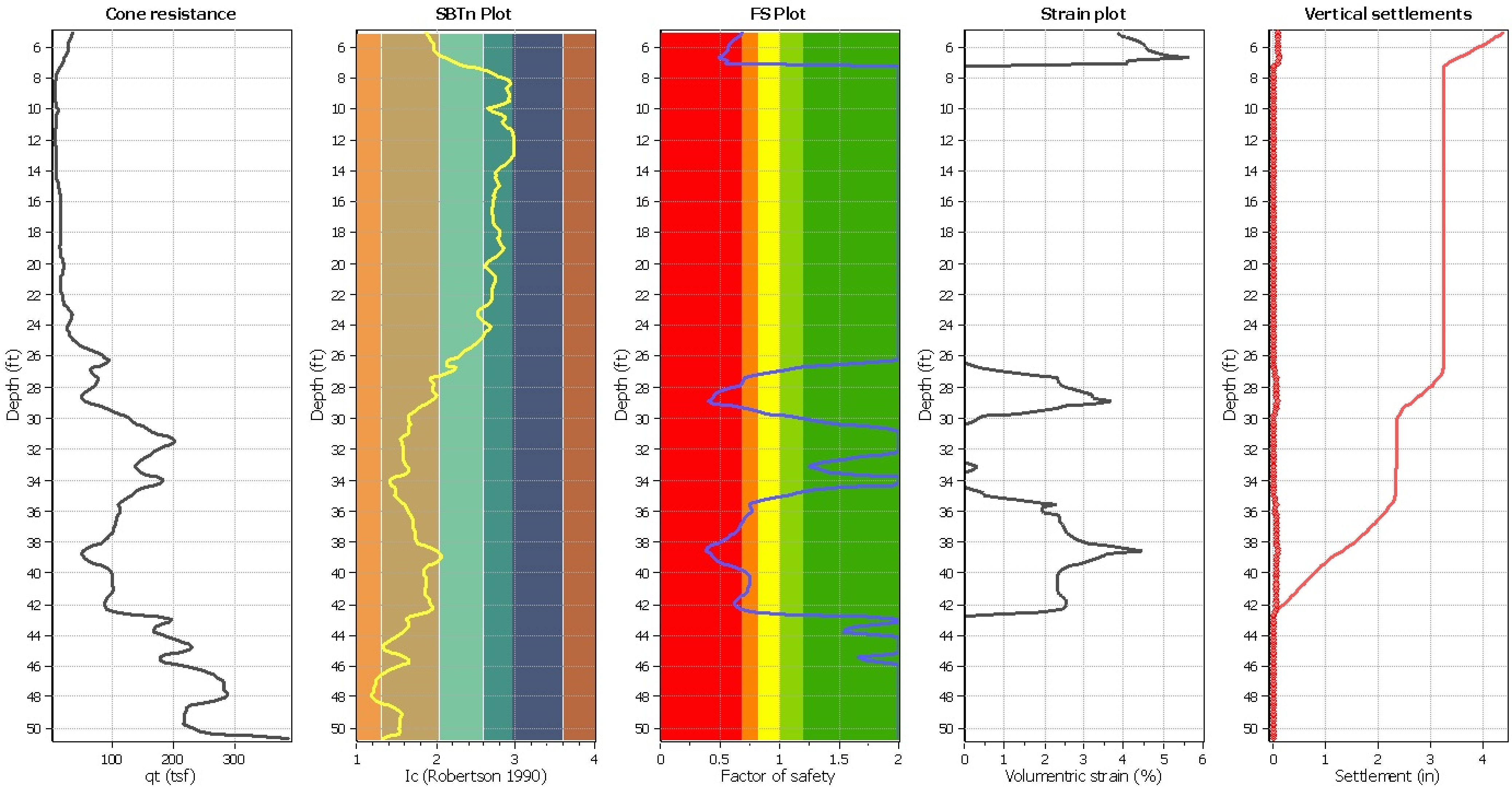
Liquefaction analysis summary plo



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements

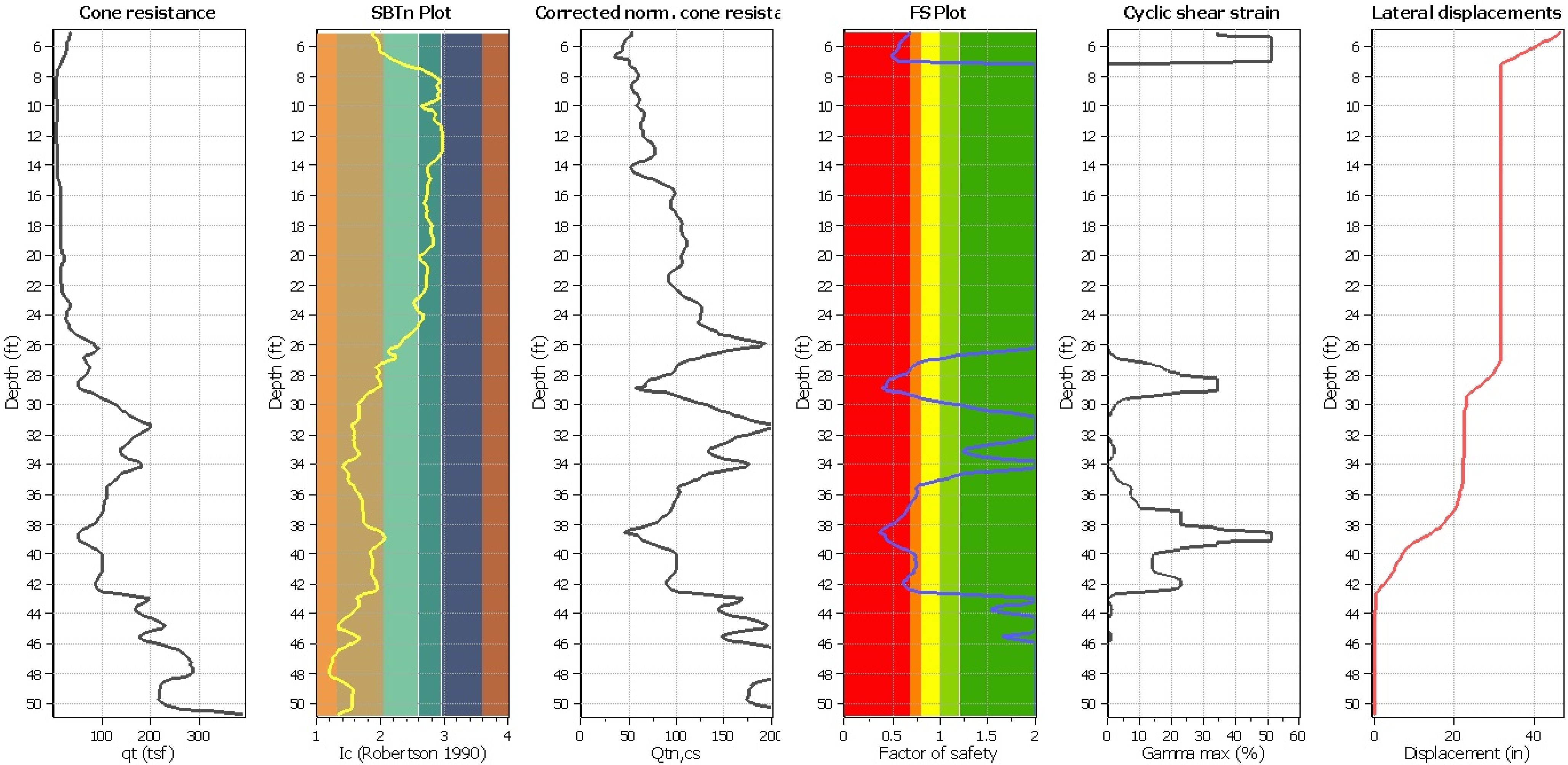


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)

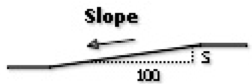


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



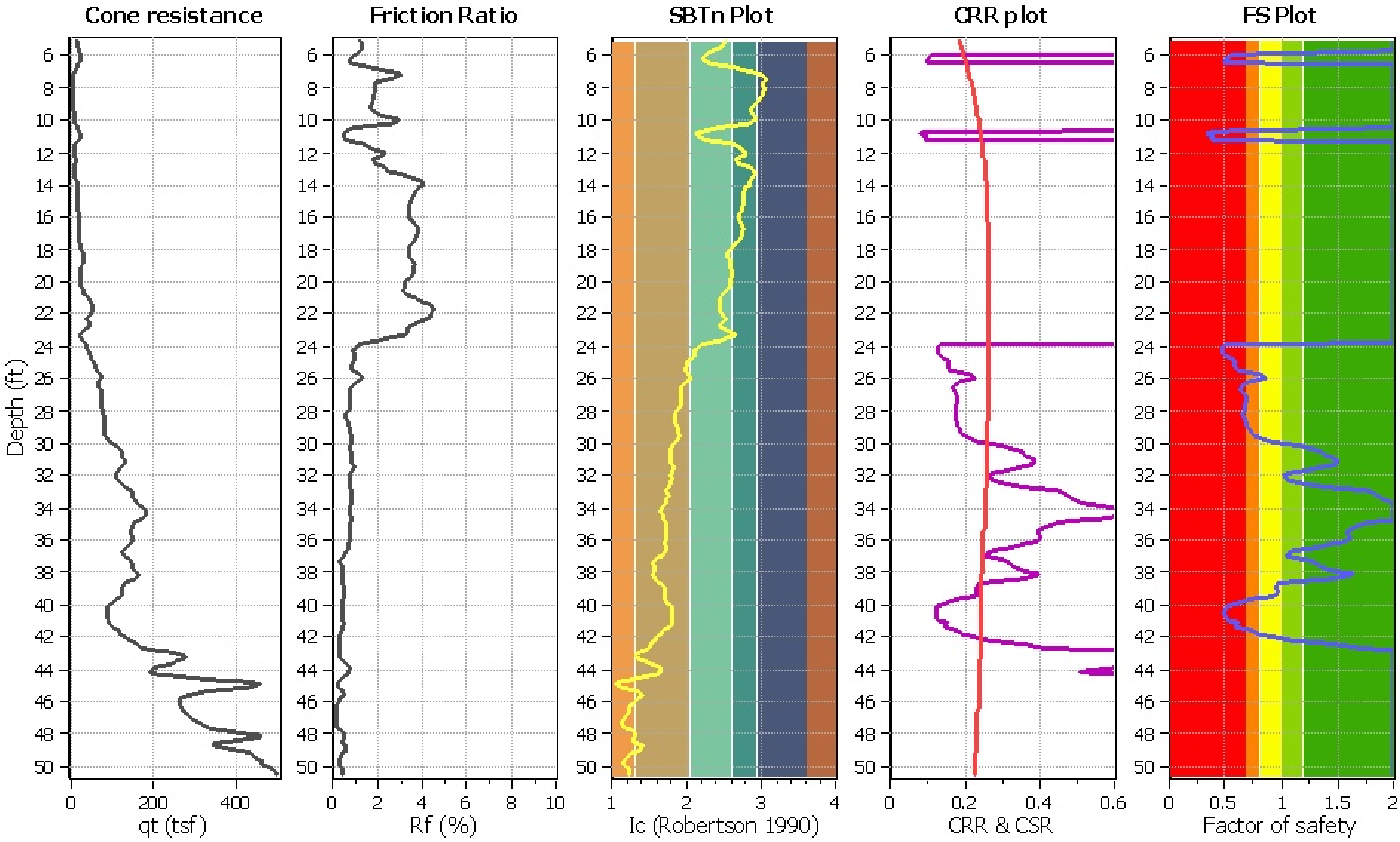
LIQUEFACTION ANALYSIS REPORT

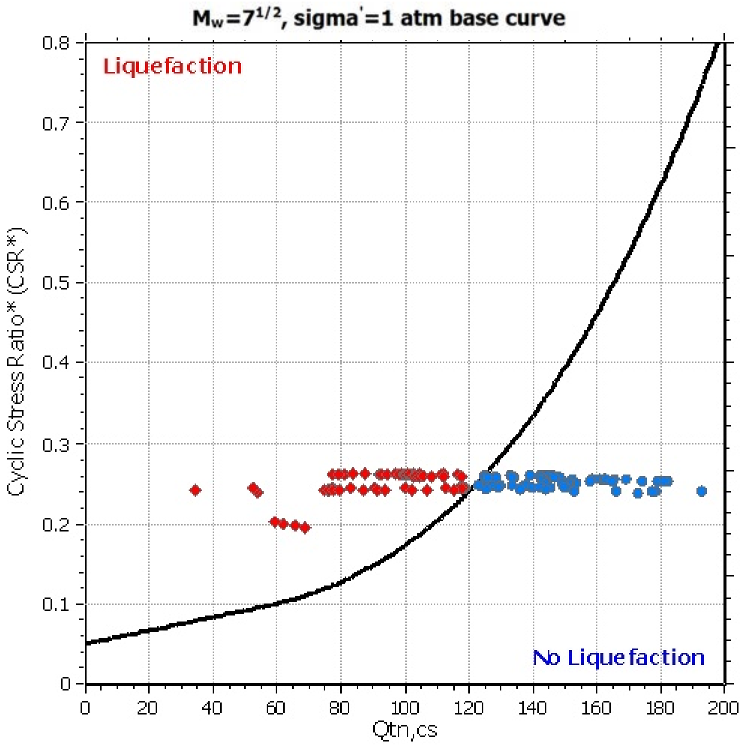
Project title : 12277.01P - Westmore Oaks **Location : West Sacramento, CA**

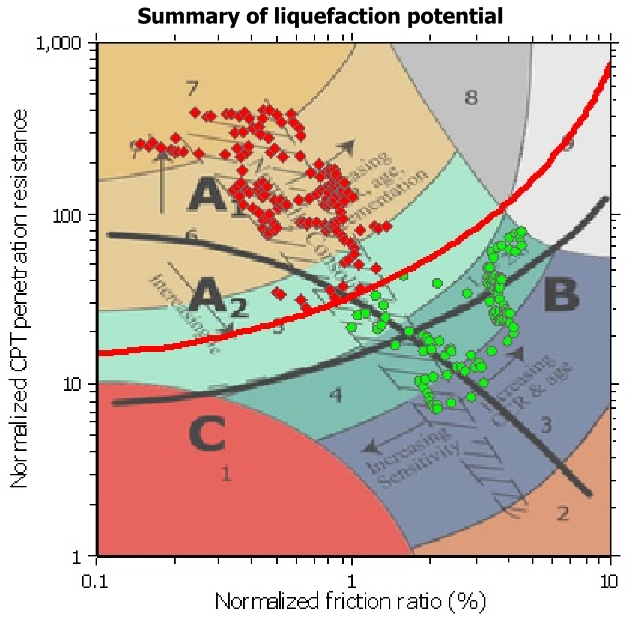
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	3.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.30	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.32	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

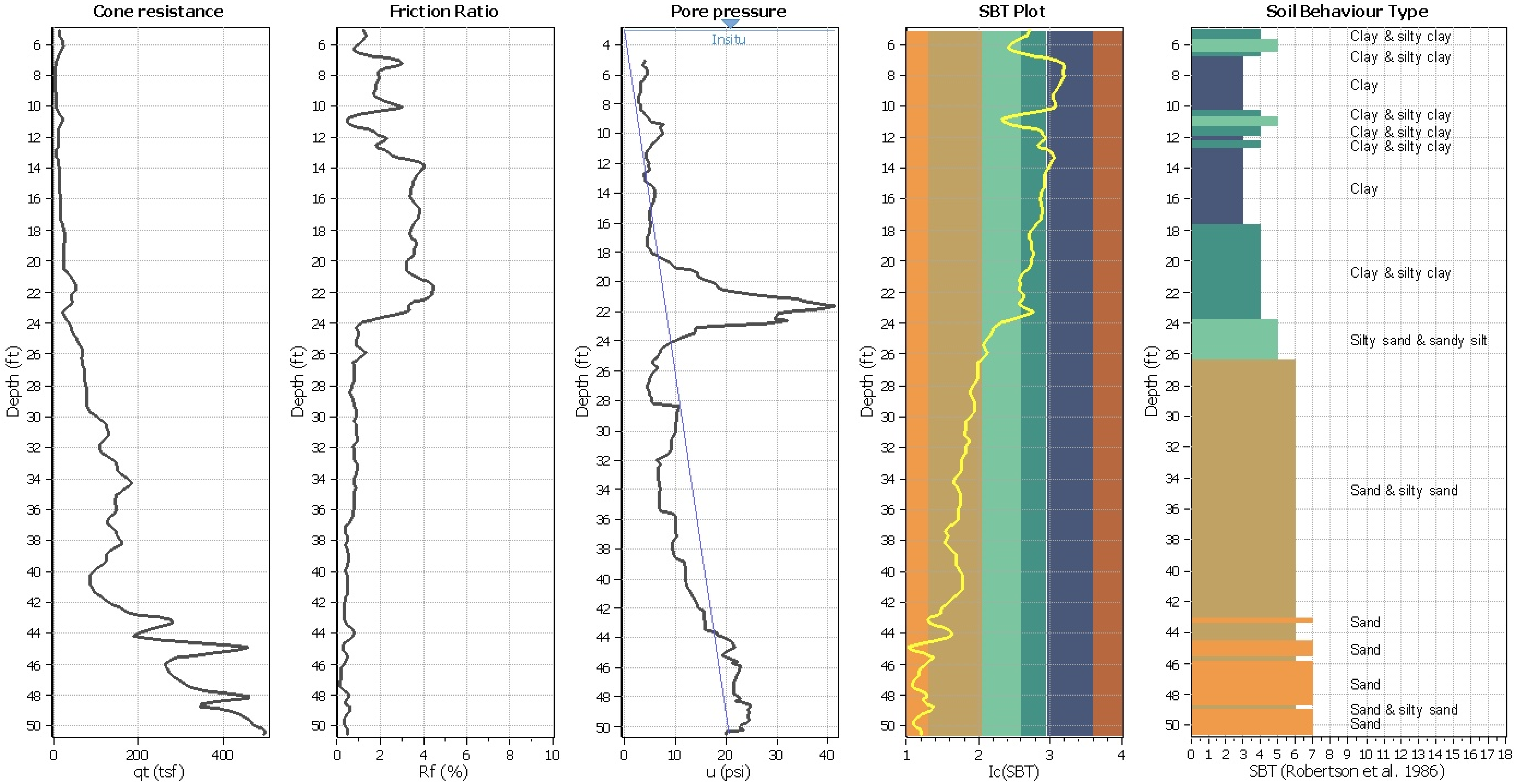






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plc



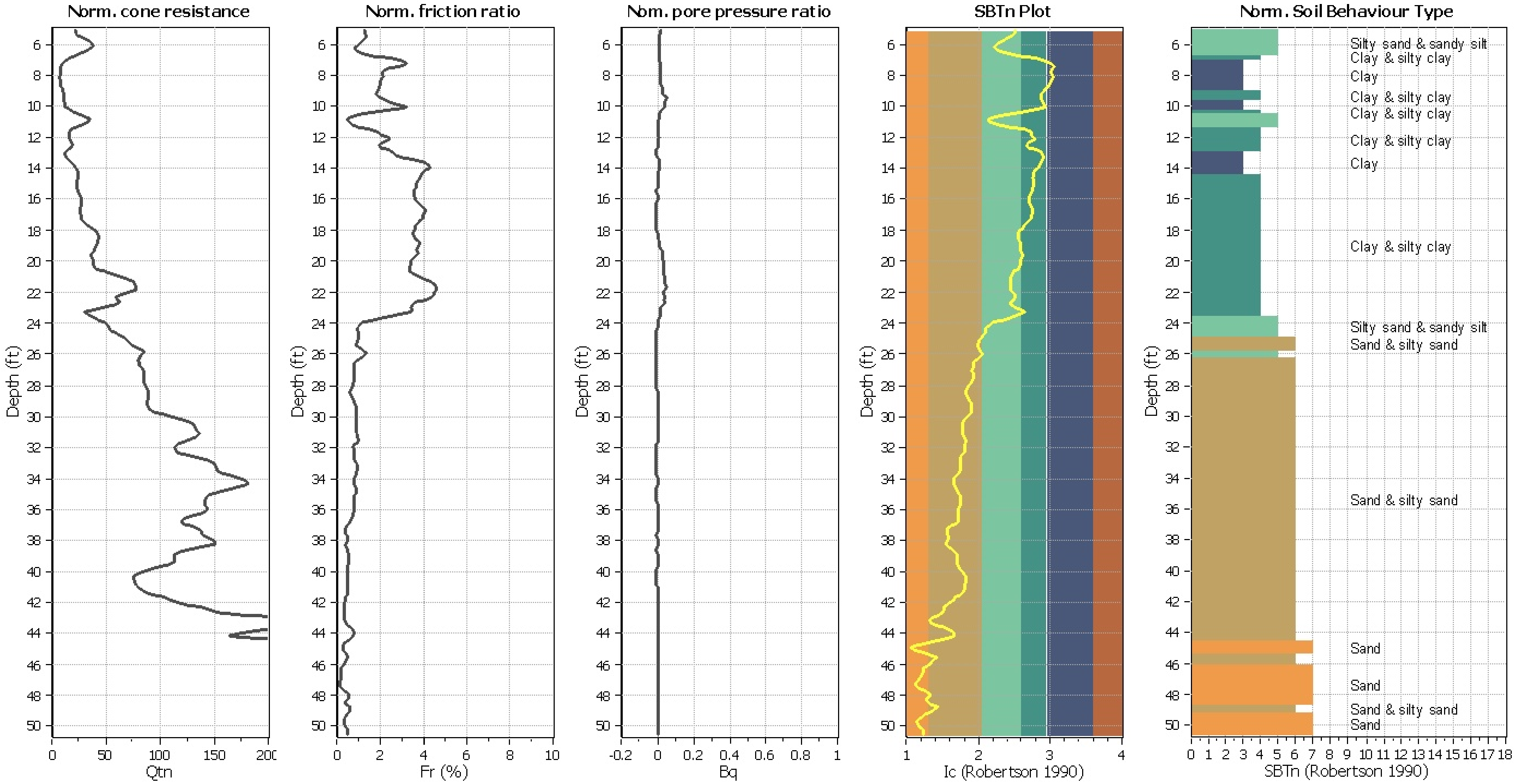
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normaliz



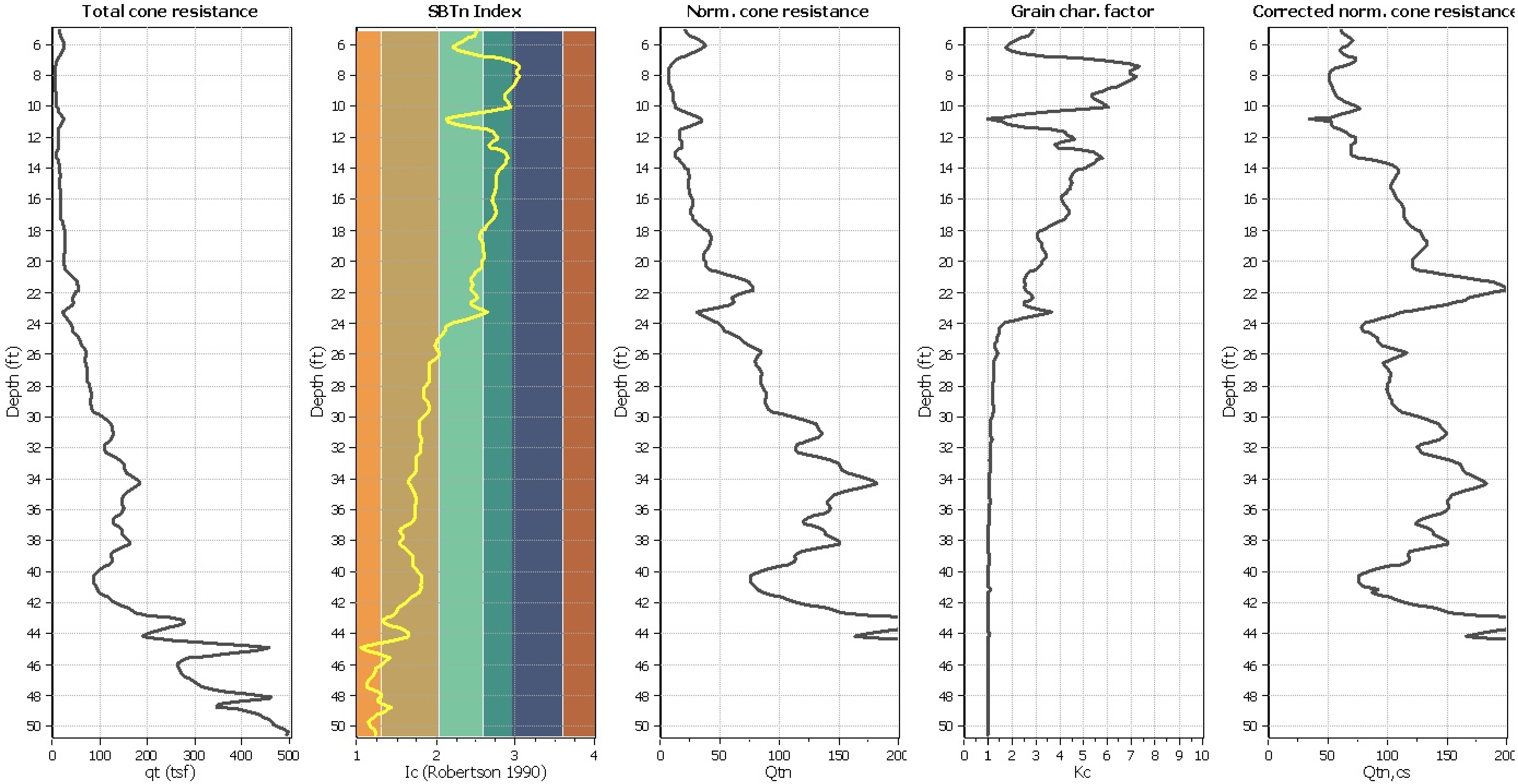
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

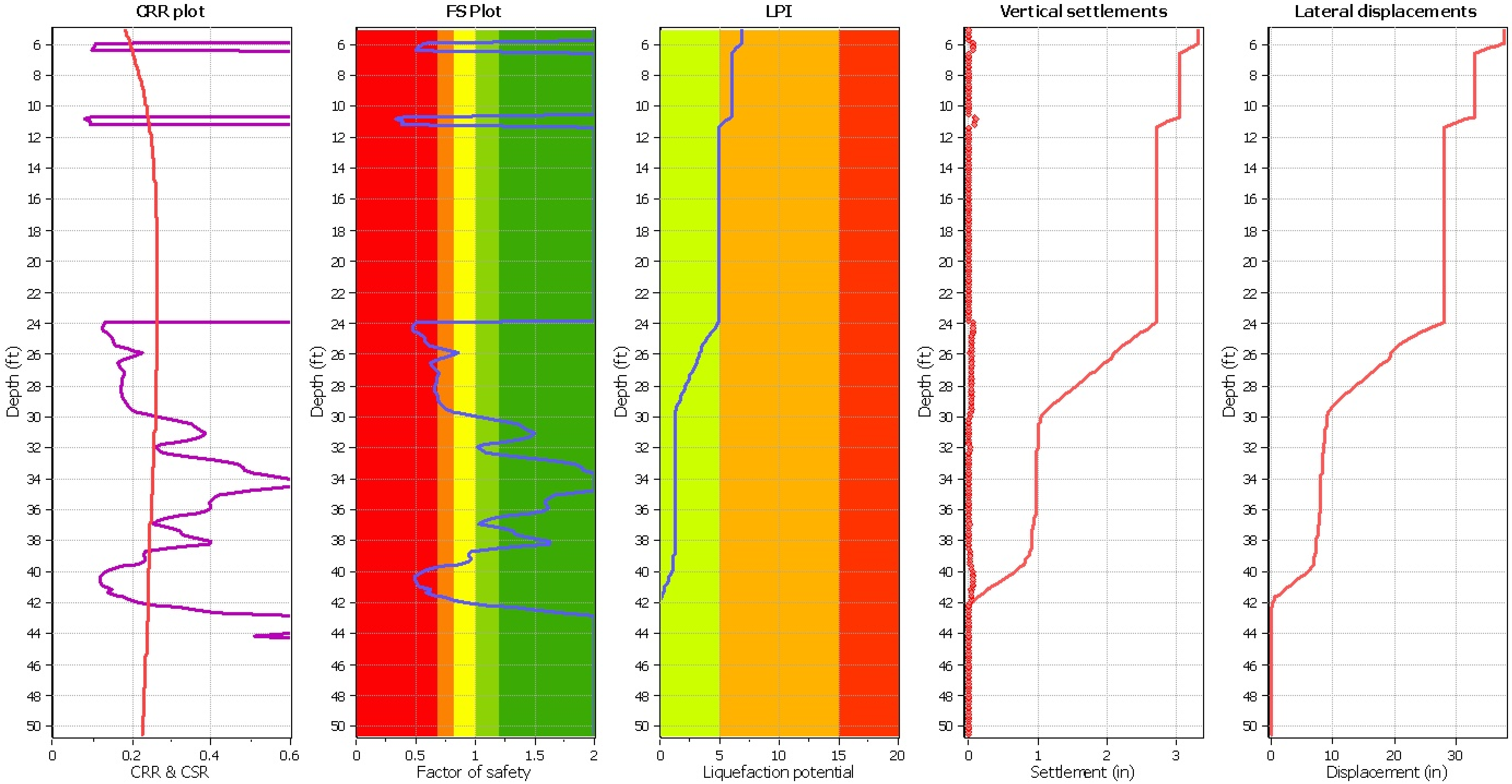
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

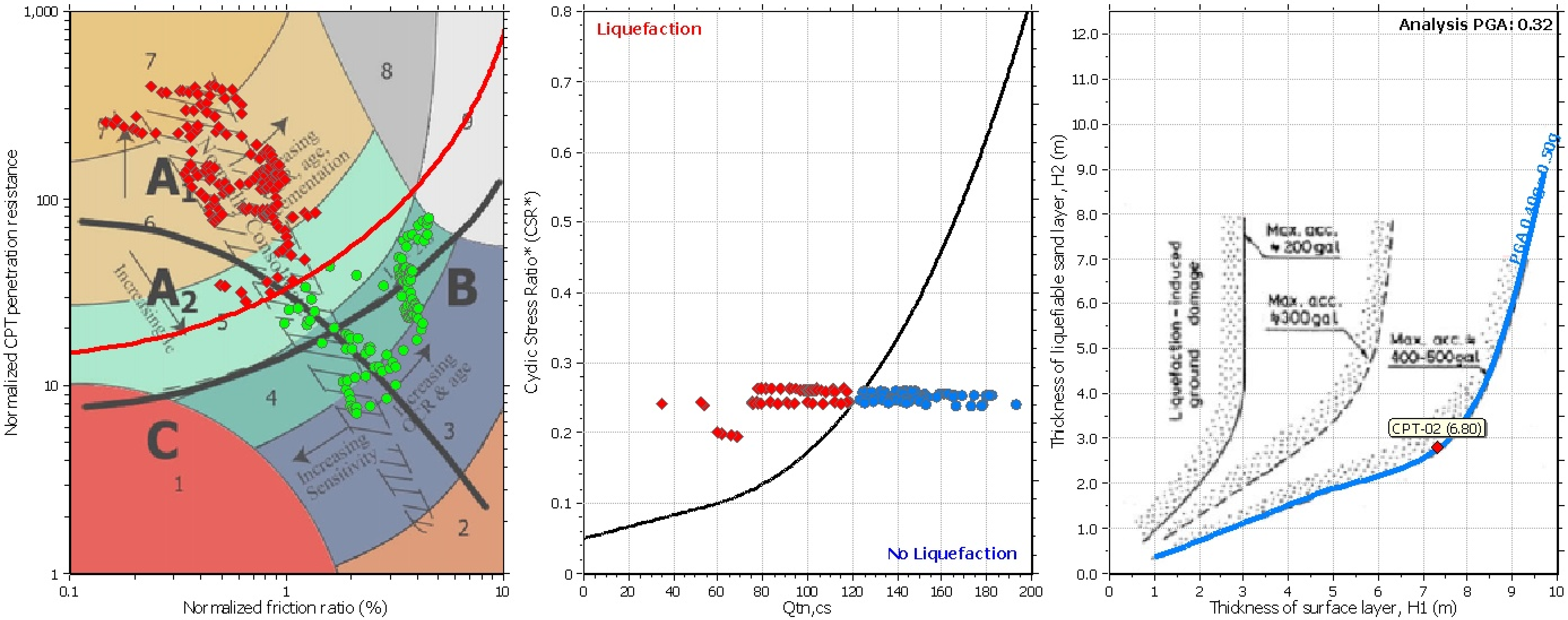
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

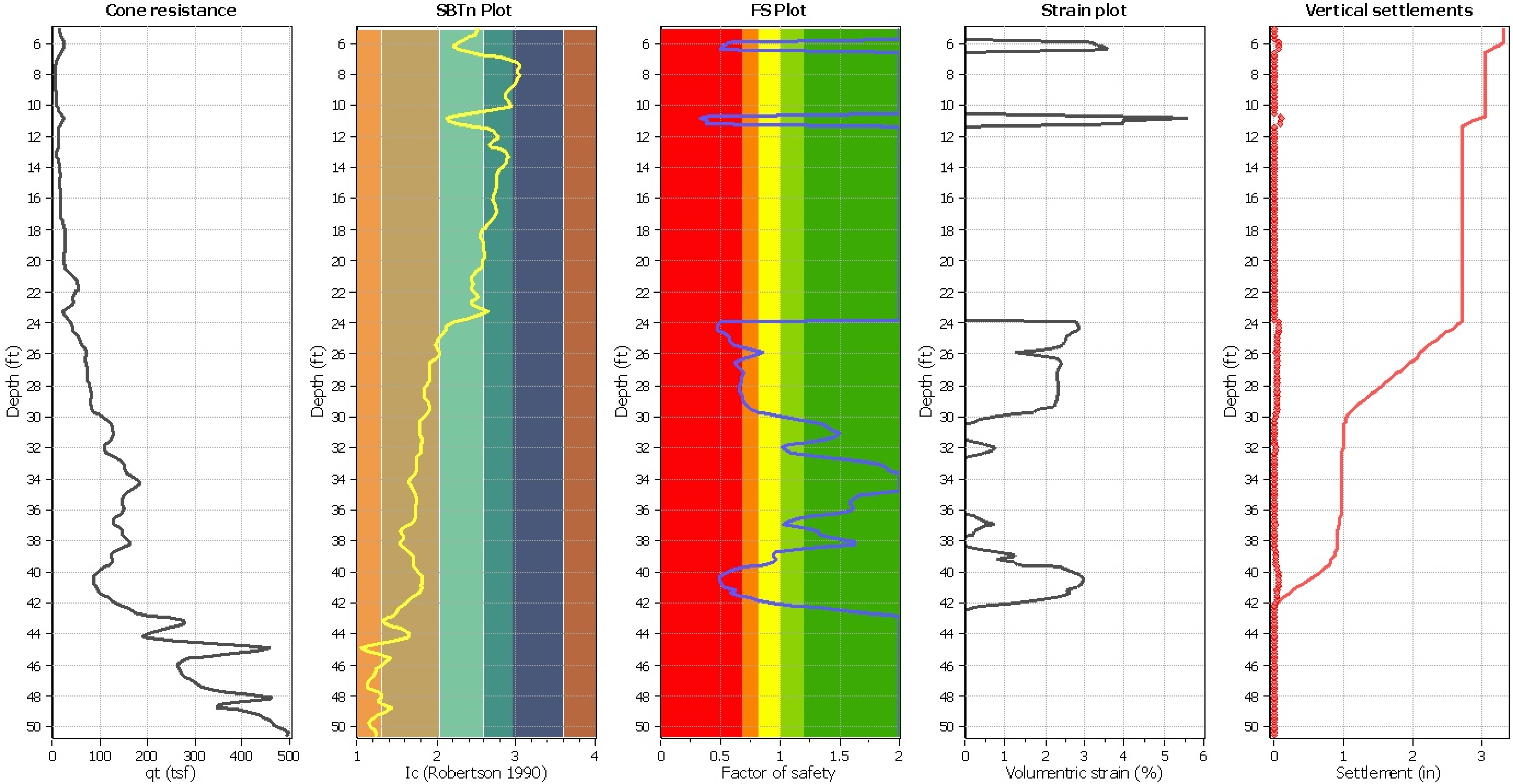
Liquefaction analysis summary plo



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.30	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.00 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements

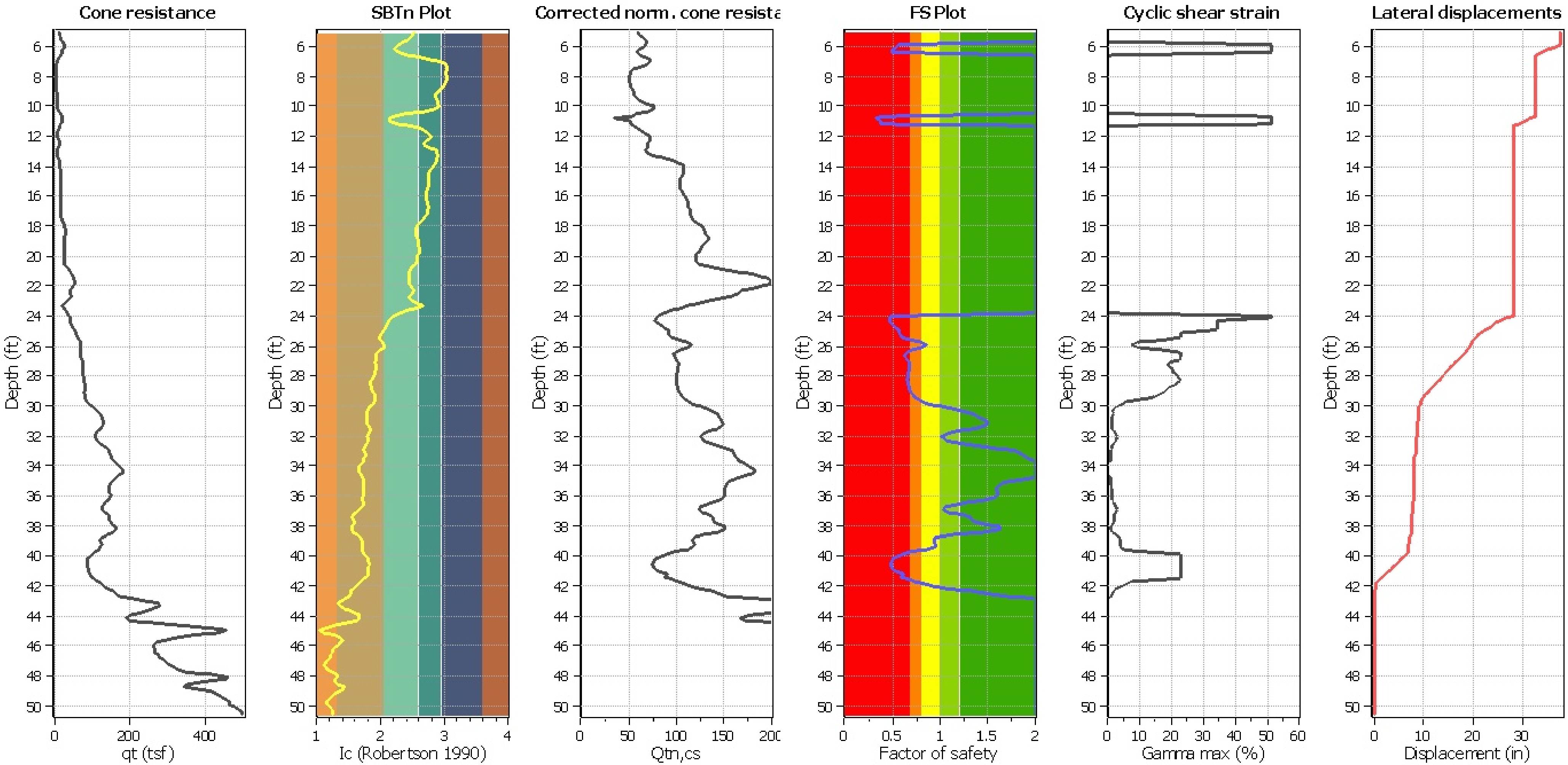


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)

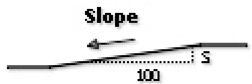


Abbreviations

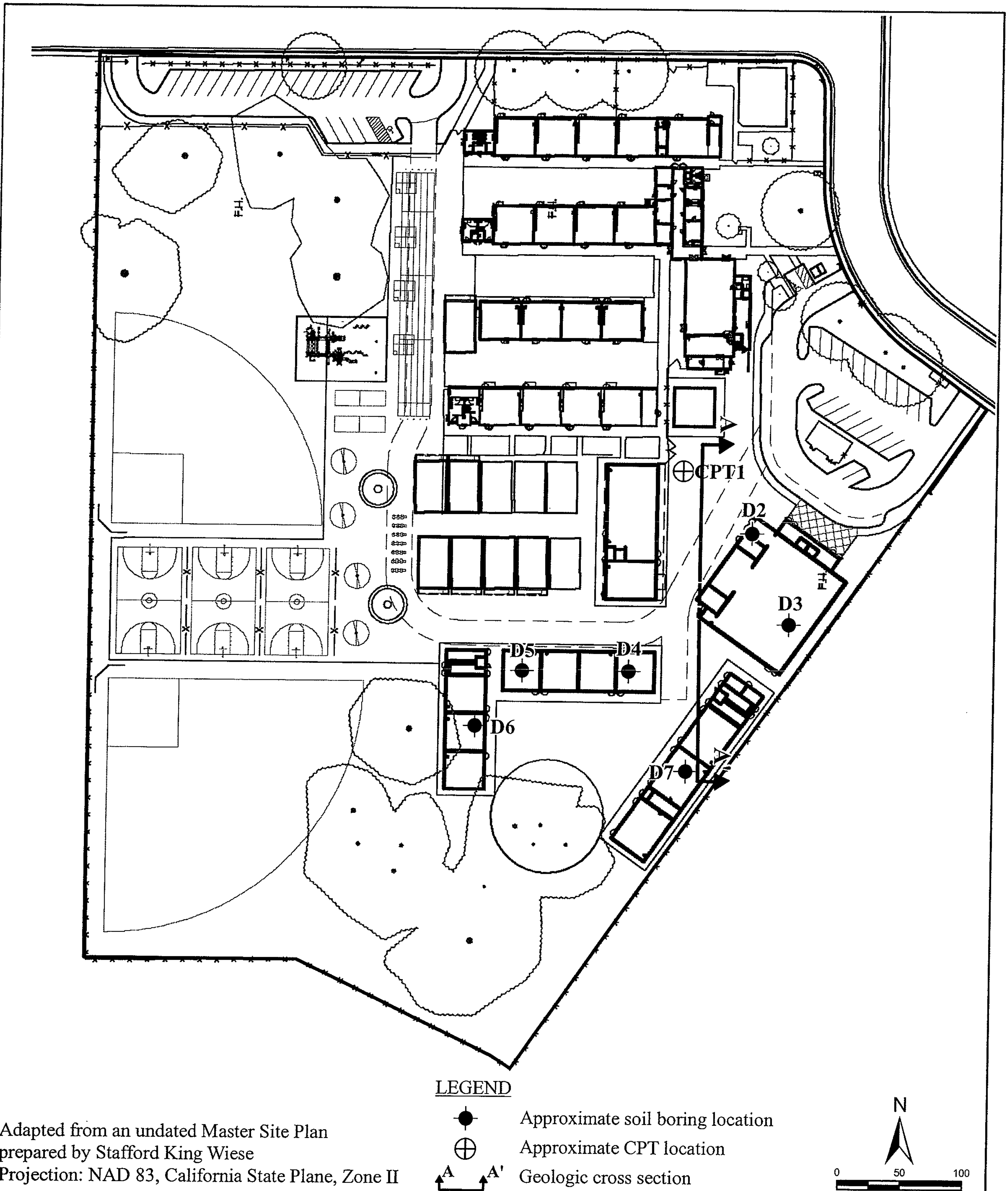
q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



APPENDIX D
2007 Exploration Logs and Laboratory Test Results



Adapted from an undated Master Site Plan
prepared by Stafford King Wiese
Projection: NAD 83, California State Plane, Zone II

Wallace Kuhl & Associates Inc

SITE PLAN

WESTMORE OAKS ELEMENTARY SCHOOL

West Sacramento, California

FIGURE	2
DRAWN BY	JG
CHECKED BY	DLP
PROJECT MGR	DLP
DATE	12/07

WKA NO. 7874.01

Project: Westmore Oaks Elementary School
Project Location: West Sacramento, California
WKA Number: 7874.01P

LOG OF SOIL BORING D2

Sheet 1 of 1

Date(s) Drilled	11/10/07	Logged By	NTW	Checked By	DLP
Drilling Method	Solid Flight Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	16.5 feet
Drill Rig Type	CME 55	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	14.0	Sampling Method(s)	Open drive sampler with 6-inch sleeves	Drill Hole Backfill	Neat cement
Remarks				Driving Method and Drop	140 lb hammer, 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0	SM	Tan, silty fine sand (SM)						
				■	D2-1I	14	4.9	93	TR
	5		Color changes to reddish brown at 5 feet						
				■	D2-2I	11	5.9	89	
	10	CL	Dark gray, silty clay (CL)						
	15			■	D2-3I	12	34.1	86	
			Boring terminated at ±16.5 feet.						

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FIGURE 6

Project: Westmore Oaks Elementary School
Project Location: West Sacramento, California
WKA Number: 7874.01P

LOG OF SOIL BORING D3

Date(s) Drilled	11/10/07	Logged By	NTW	Checked By	DLP
Drilling Method	Solid Flight Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	20.0 feet
Drill Rig Type	CME 55	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	14.0	Sampling Method(s)	Open drive sampler with 6-inch sleeves	Drill Hole Backfill	Neat cement
Remarks				Driving Method and Drop	140 lb hammer, 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		Reddish brown, silty fine sand (SM)						
					D3-1I	7	7.8	88	
	5				D3-2I	12			
			Color changes to brown at 8.5 feet, trace gravels						
			Dark gray, silty clay (CL)						
	10				D3-3I	5	47.3	70	
			Color changes to reddish brown at 13.5 feet						
	15				D3-4I	10			
	20				D3-5I	19			
			Boring terminated at ±20 feet.						

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FIGURE 7

Project: Westmore Oaks Elementary School
Project Location: West Sacramento, California
WKA Number: 7874.01P

LOG OF SOIL BORING D4

Sheet 1 of 1

Date(s) Drilled	11/10/07	Logged By	NTW	Checked By	DLP
Drilling Method	Solid Flight Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	10.0 feet
Drill Rig Type	CME 55	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not encountered	Sampling Method(s)	Open drive sampler with 6-inch sleeves	Drill Hole Backfill	Neat cement
Remarks				Driving Method and Drop	140 lb hammer, 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Tan, silty fine sand (SM)						
					D4-1I	14	9.9	92	
			Color changes to reddish brown at 3.5 feet						
	5				D4-2I	16	10.4	96	
	10				D4-3I	12			
			Boring terminated at ±10 feet. Groundwater was not encountered.						

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FIGURE 8

Project: Westmore Oaks Elementary School **Project Location:** West Sacramento, California **WKA Number:** 7874.01P	**LOG OF SOIL BORING D5** Sheet 1 of 1

Date(s) Drilled	**11/10/07**	Logged By	**HML**	Checked By	**DLP**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**Wallace-Kuhl & Associates**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**Gator 4x6**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not encountered**	Sampling Method(s)	**Open drive sampler with 6-inch sleeves**	Drill Hole Backfill	**Neat cement**
Remarks				Driving Method and Drop	**70 lb hammer,**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
				SAMPLE DATA			TEST DATA		
	0		2" sod layer over silty fine sand (SM)						
				■	D5-1I	7	5.1	85	
	5		Brown, sandy, clayey silt (ML)						
			Grayish brown, silty clay (CL)	■	D5-2I	13			
	10		-Bottom of hole at 10.0 feet -No groundwater encountered						

Project: Westmore Oaks Elementary School **Project Location: West Sacramento, California** **WKA Number: 7874.01P**	**LOG OF SOIL BORING D6** Sheet 1 of 1

Date(s) Drilled	**11/10/07**	Logged By	**HML**	Checked By	**DLP**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**Wallace-Kuhl & Associates**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**Gator 4x6**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not encountered**	Sampling Method(s)	**Open drive sampler with 6-inch sleeves**	Drill Hole Backfill	**Neat cement**
Remarks				Driving Method and Drop	**70 lb hammer,**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
					SAMPLE DATA		**TEST DATA**		
			2 inch sod layer over silty fine sand (SM)		D6-1I	8			
	5		Brown, sandy silt (ML)		D6-2I	11	21.8	71	
			Grayish brown, silty clay (CL)						
	10		-Bottom of hole at 10.0 feet -No groundwater encountered						

BORING LOG 7874.01P - WESTMORE OAKS ELEMENTARY SCHOOL.GPJ WKA.GDT 12/12/07 2:39 PM

Wallace Kuhl & ASSOCIATES INC

FIGURE 10

Project: Westmore Oaks Elementary School Project Location: West Sacramento, California WKA Number: 7874.01P	**LOG OF SOIL BORING D7** Sheet 1 of 1

Date(s) Drilled	11/10/07	Logged By	NTW	Checked By	DLP
Drilling Method	Solid Flight Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	10.5 feet
Drill Rig Type	CME 55	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not encountered	Sampling Method(s)	Open drive sampler with 6-inch sleeves	Drill Hole Backfill	Neat cement
Remarks				Driving Method and Drop	140 lb hammer, 30-inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		Brown, silty fine sand / fine sandy silt (SM/ML) with trace roots						
				■	D7-1I	7	3.4	83	
	5								
	6		Grayish dark brown, clayey, silty fine sand (SM) with clay modules						
	10			■	D7-2I	7	15.4	104	
	10.5		Boring terminated at ±10.5 feet. Groundwater was not encountered.						

BORING LOG 7874.01P - WESTMORE OAKS ELEMENTARY SCHOOL.GPJ WKA.GDT 12/12/07 2:39 PM

Wallace Kuhl & Associates Inc

FIGURE 11

UNIFIED SOIL CLASSIFICATION SYSTEM

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

OTHER SYMBOLS

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

= Drive Sample: no recovery

= SPT Sample

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

PI = Plasticity Index

EI = Expansion Index

UCC = Unconfined Compression Test

TR = Triaxial Compression Test

GR = Gradational Analysis (Sieve)

K = Permeability Test

GRAIN SIZE CLASSIFICATION

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

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UNIFIED SOIL CLASSIFICATION SYSTEM

WESTMORE OAKS ELEMENTARY SCHOOL

West Sacramento, California

FIGURE 12	
DRAWN BY	JG
CHECKED BY	DLP
PROJECT MGR	DLP
DATE	12/07

WKA NO. 7874.01

APPENDIX A

A. GENERAL INFORMATION

The preparation of a geotechnical engineering report for the proposed Westmore Oaks Elementary School expansion, located at 1504 Fallbrook Street in West Sacramento, California, was authorized by Mr. Myles Billheimer of the Washington Unified School District on October 30, 2007. Authorization was for an investigation as described in our proposal letter of October 26, 2007, sent to our client, the Washington Unified School District, whose mailing address is 1706 Grande Vista Avenue, West Sacramento, California 95691; telephone (916) 375-7698; facsimile (916) 375-7699.

The architectural consultant for this project is Stafford King Wiese - Architects, whose mailing address is 622 20th Street, Sacramento, CA 95814; telephone (916) 930-0736, facsimile (916) 930-5848.

In performing this investigation, we made reference to November 19, 2007 *Master Site Plan*, provided by Stafford King Wiese - Architects.

B. FIELD EXPLORATION

At the approximate locations indicated on Figure 2, six exploratory borings were drilled on November 10, 2007. Four of the borings were performed utilizing a CME-55 truck-mounted drill rig to a maximum depth of about 20 feet using six-inch diameter solid stem augers. At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D. California sampler driven by a 140 pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each 6-inch interval was recorded with the sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, being designated the penetration resistance or "blow count" for that particular drive. The other two borings were performed utilizing a Giddings JD 6x4 limited access drill rig to a maximum depth of about 10 feet equipped with 4-inch diameter solid-stem helical flight augers. At various depths relatively undisturbed soil samples were collected from these test borings using a 12-inch long, 2½-inch O.D., 2-inch I.D. modified California sampler driven by a 70-pound hand-operated slide hammer.

At the approximate location indicated on Figure 2, one Cone Penetration Test (CPT) sounding was advanced to a maximum depth of approximately 50 feet below existing site grades.

The samples were retained in 2-inch diameter by 6-inch long thin-walled brass tubes contained within the sampler. Immediately after recovery, the soils in the tubes were visually classified by the field engineer and the ends of the tubes were sealed to preserve the natural moisture contents. Bulk samples of the near-surface soils also were collected for subgrade analysis. All samples were taken to our laboratory for soil classification and selection of samples for testing. The Logs of Test Borings, Figures 3 through 8, contain descriptions of the soils encountered in each boring. A Boring Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained in Figure 9. The Cone Penetration Test results are presented in Appendix C.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937) and natural moisture content (ASTM D2216). The results of the moisture content and dry unit weight tests are included on the boring logs at the depth each sample was obtained.

An undisturbed sample of the subsurface site soil was subjected to triaxial compression testing (ASTM D4767) to determine the shear strength properties of the materials. The results of the test are presented on Figure A1.

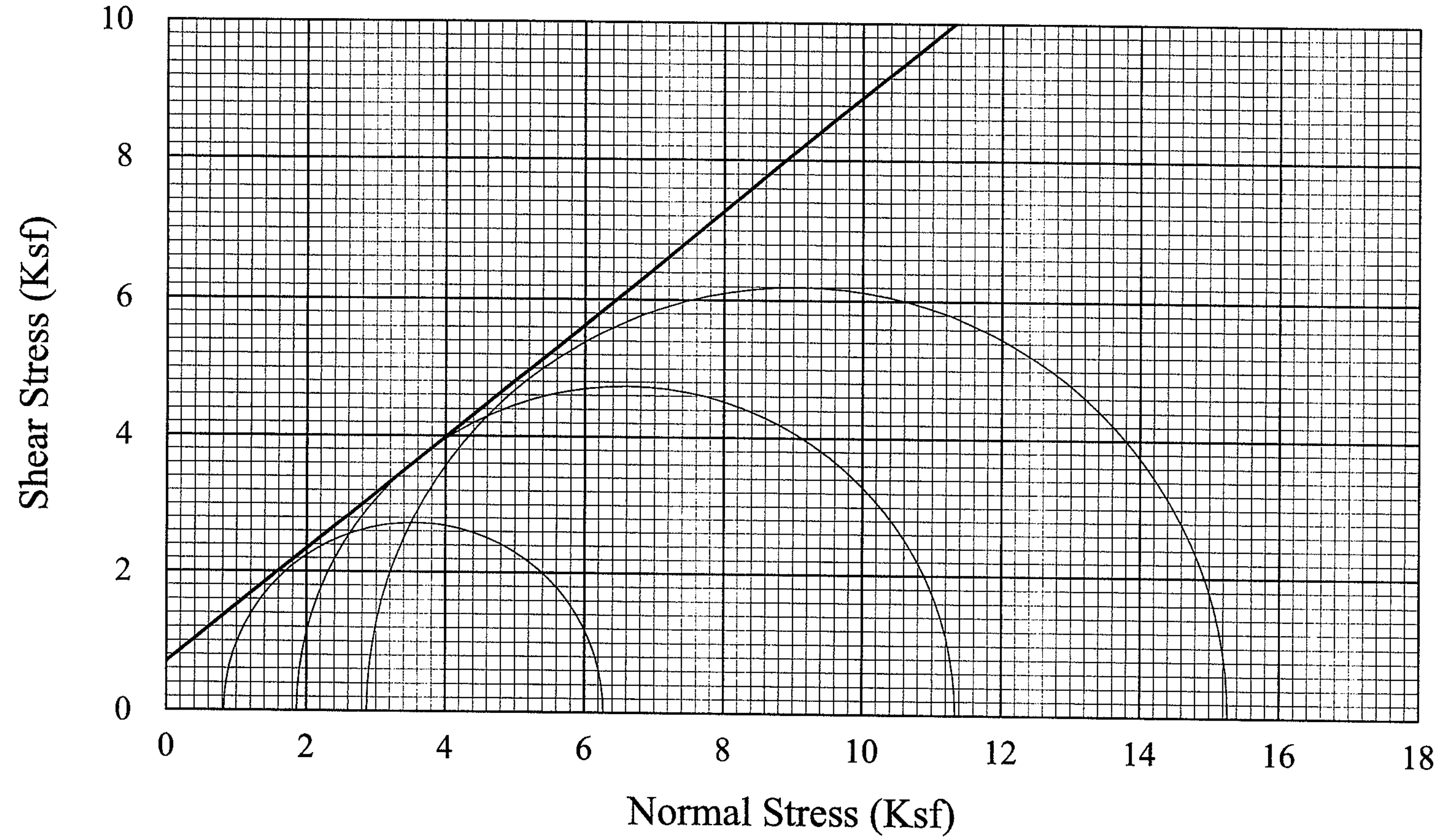
One bulk sample anticipated pavement subgrade soils was subjected to Resistance-value ("R") testing in accordance with California Test (CT) 301. The results of the R-value tests are presented on Figure A2.

One representative bulk sample of near-surface soils was tested by Sunland Analytical Lab to determine the preliminary corrosion characteristics of the soil (CT 417, 422, 643). The results of the tests are presented on Figure A3.

One representative bulk samples collected from proposed play field areas were submitted to Sunland Analytical for landscape analysis. Results and recommendations from this testing are included in Appendix E.

TRIAXIAL COMPRESSION TEST

ASTM D4767-04



SAMPLE NO. :	D2-1I	DRY DENSITY (PCF) :	93
SAMPLE CONDITION :	Undisturbed	INITIAL MOISTURE (%) :	4.9
SAMPLE DESCRIPTION :	Brown, silty fine sand	FINAL MOISTURE (%) :	16.8
		ANGLE OF INTERNAL FRICTION (ø) :	40°
		COHESION (PSF) :	700

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TRIAXIAL COMPRESSION TEST

WESTMORE OAKS ELEMENTARY SCHOOL

West Sacramento, California

FIGURE A1

DRAWN BY	HCS
CHECKED BY	DLP
PROJECT MGR	DLP
DATE	12/07

WKA NO. 7874.01

RESISTANCE VALUE TEST RESULTS
(California Test 301)

MATERIAL DESCRIPTION: Brown silty fine sand

LOCATION: D3 @ 0' - 3'

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R Value
1	113.0	13.8	141	0	54
2	112.3	12.5	506	0	65
3	113.7	11.6	743	0	68

R-Value at 300 psi exudation pressure = 60

WallaceKuhl & ASSOCIATES INC

RESISTANCE VALUE TEST RESULTS

WESTMORE OAKS ELEMENTARY SCHOOL

West Sacramento, California

FIGURE A2	
DRAWN BY	JG
CHECKED BY	DLP
PROJECT MGR	DLP
DATE	12/07
WKA NO. 7874.01	

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 11/30/2007
Date Submitted 11/27/2007

To: David Perry
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 : 7874.01 WESTMORE OAK Site ID : D2-1II.
Your purchase order number is 2362.
Thank you for your business.

* For future reference to this analysis please use SUN # 52199-104374.

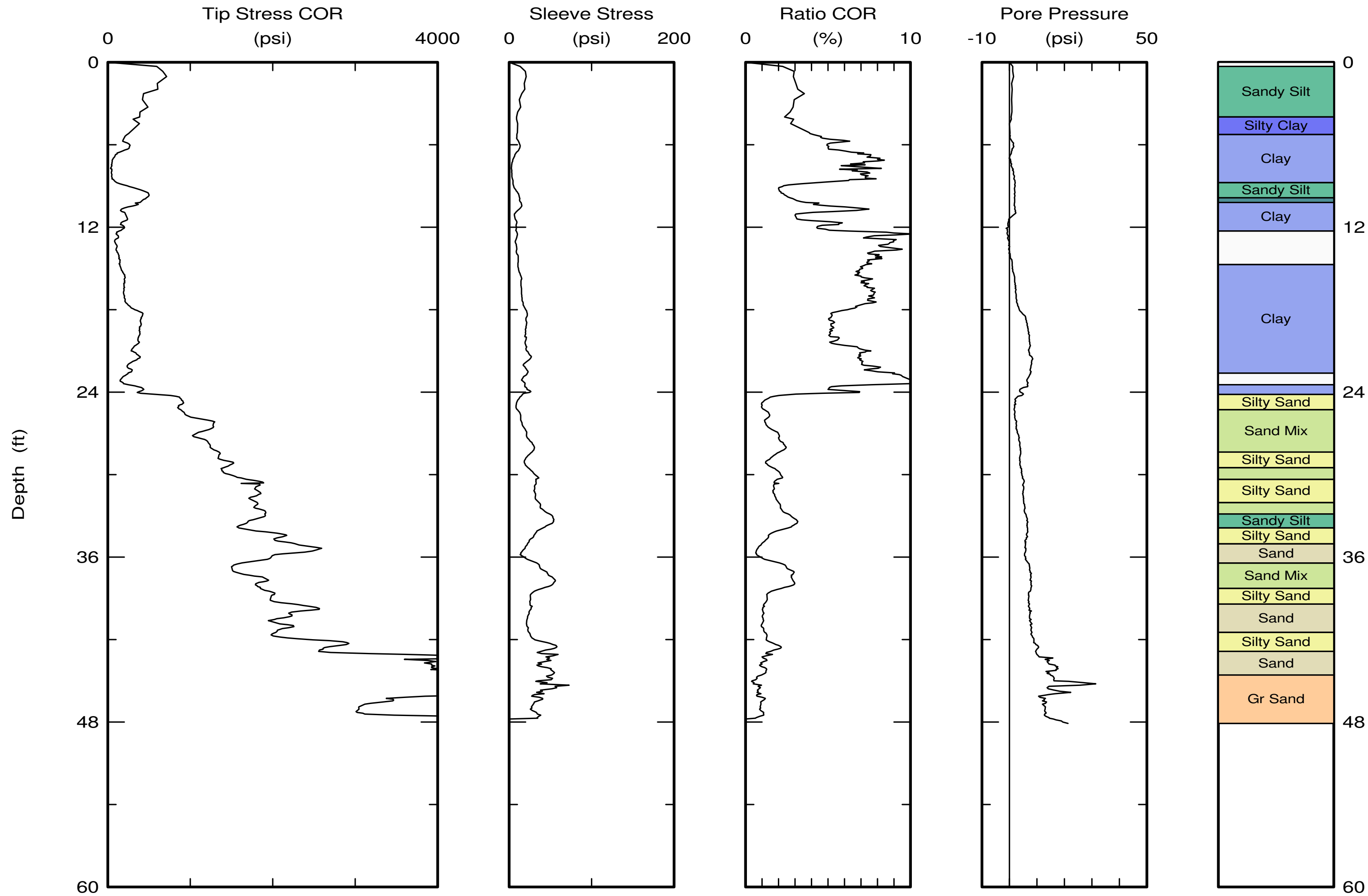
EVALUATION FOR SOIL CORROSION

Soil pH	7.40		
Minimum Resistivity	5.63	ohm-cm (x1000)	
Chloride	7.3 ppm	00.00073	%
Sulfate	47.8 ppm	00.00478	%

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

WallaceKuhl & ASSOCIATES INC	CORROSION TEST RESULTS WESTMORE OAKS ELEMENTARY SCHOOL West Sacramento, California	FIGURE A3	
		DRAWN BY	HCS
		CHECKED BY	DLP
		PROJECT MGR	DLP
		DATE	12/07
		WKA NO. 7874.01	

Test America Inc. Anaheim CA 714-939-6850 www.tadrillingcorp.com	Northing: Easting: Elevation:	Date: 10/Nov/2007 Test ID: 7874.01 cpt-01 Project: 07-2976
	Client: Wallace & Kuhl Job Site: 7874.01	



Maximum depth: 48.10 (ft)