# Geotechnical Engineering Report
# JOHN F. KENNEDY HIGH SCHOOL ATHLETIC FIELD IMPROVEMENTS

**6715 Gloria Drive**
**Sacramento, California 95823**

**Prepared for:**
Verde Designs, Inc.
1024 Iron Point Road, Suite 100 #1241
Folsom, California 95630

**Prepared by:**
Universal Engineering Sciences
3050 Industrial Boulevard
West Sacramento, CA 95691

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## 1.0 INTRODUCTION

We have completed a geotechnical engineering study for the proposed athletic field improvements to be constructed at the existing John F. Kennedy High School campus located at 6715 Gloria Drive in Sacramento, California. The purposes of our work have been to explore the existing site, soil and groundwater conditions, and to provide geotechnical engineering conclusions and recommendations for the design and construction of the proposed construction and associated improvements.

### 1.1 Scope of Work

Our scope of work included the following tasks:

1. a site reconnaissance;
2. review of previous geotechnical reports prepared by our firm on the campus;
3. review of United States Geological Survey (USGS) topographic map, historical aerials, and available groundwater information relevant to the site;
4. subsurface exploration, including three soil borings to depths ranging from approximately 50 to 60½ feet below the ground surface and two hand auger samples to depths of approximately 5 feet below the ground surface;
5. laboratory testing of selected soil samples;
6. engineering analyses; and,
7. preparation of this report.

### 1.2 Project Description

We understand the project will consist of the construction of new baseball and softball field improvements, including new dugouts, backstops, and field lighting. We understand the fields will remain natural turf. Additional improvements will also include renovations to the existing tennis courts at the southeast end of the campus.

### 1.3 Related Experience

We have reviewed our Geologic Hazards and Geotechnical Engineering Report (Wallace-Kuhl & Associates [WKA], Inc. No. 7915.01P, dated February 2008) prepared for construction of a performing arts building in the northeastern portion of the existing campus; Geologic Hazards and Geotechnical

Engineering Report (Wallace-Kuhl & Associates [WKA], Inc. No. 7915.05P, dated August 2013) prepared for improvements to the athletic field facilities in the western portion of the existing campus; Geotechnical Engineering Report (Wallace-Kuhl & Associates [WKA], Inc. No. 12197.01P, dated February 2019) prepared for improvement of existing campus buildings and addition of metal canopy structures; and Pavement Design Recommendations (Universal Engineering Sciences [UES] No. 4630.2200147.0016, dated August 2022) prepared for replacement of the parking lot in the north portion of the existing campus. Previous subsurface exploration and laboratory test results obtained at the site will be used to prepare the requested geotechnical engineering report.

### 1.4 Figures and Attachments

This report contains a Vicinity Map as Figure 1, a Site Plan showing the approximate boring locations as Figure 2, the Logs of Soil Borings as Figures 3 through 5, and the Logs of Hang Auger Borings as Figure 6. An explanation of the symbols and classification system used on the logs is contained in Figure 7. Appendix A contains information of a general nature regarding project concepts, exploratory methods used during the field exploration phase of our investigation, and laboratory test results. Appendix B contains the Logs of Soil Borings from the previous geotechnical investigation performed in 2013.

## 2.0 FINDINGS

### 2.1 Site Description

The subject site is located within the southern portion of the John F. Kennedy High School campus which is located at 6715 Gloria Drive in Sacramento, California (Figure 1). The campus is on an approximately 43-acre parcel identified as Sacramento County Assessor Parcel Number 030-0370-021-0000. The project site is bounded to the north by school's track and field area and asphalt concrete play areas; to the east by additional auxiliary play fields, beyond which are the school's tennis courts; to the south by a residential subdivision; and to the west by Florin Road, beyond which is a residential subdivision.

At the time of our field explorations on August 10, 2023, the site was developed with existing grass-covered baseball and softball fields which includes backstops, dugouts, temporary bleachers, batting cages, and fencing.

Surface elevation of the site is approximately +10 feet North American Vertical Datum of 1988 (NAVD88) and the elevation estimates are based on the United States Geologic Survey (USGS) topographic data shown on the 7.5-Minute Map of the Clarksburg Quadrangle, California, dated 2015.

## 2.2 Historical Aerial Photograph Review

We reviewed historical aerial photographs from 1947, 1957, 1964, 1966, 1984, 1993, 1998, 2005, 2009, 2010, 2012, 2014, 2016, 2018, 2020 through 2023. Review of the photographs from 1947 through 1964 indicate the entire site to be a grass field without any campus buildings. Aerial imagery from 1966 shows beginning phases of construction of the campus, and aerial imagery from 1984 shows existing school building, asphalt concrete play areas, large parking area in the northwest portion of the campus, a dirt track and field area in the western portion of the site, tennis courts in the southeastern portion of the campus, and the site area itself as a baseball field surrounded by grass play field. The 1998 photographs show addition of a concrete path just east of the track and field and what looks to be dugouts on the baseball field. 2005 imagery shows a clear softball infield in the southwestern portion of the site area. Photographs from 2014 show beginning phases of construction to replace the dirt track and field area, and photographs from 2016 show a completion of track and field improvements. The photographs from 2016 through 2023 reveals the site is in a similar condition as it was during our field work in August 2023.

## 2.3 Soil Conditions

On August 10, 2023, three exploratory borings (D1 through D3) and two hand auger borings (HA1 and HA2) were performed at the project site. The approximate locations are shown in the attached Site Plan (Figure 2).

The soil conditions encountered at the boring locations generally consist of soft to stiff clay underlain by loose to very loose sand and silt extending to the explored depths of about 50 to 61½ feet below ground surface (bgs).

The soil conditions encountered at the boring locations are generally consistent with the soil conditions previously encountered at the site.

For soil conditions at a particular location, refer to the attached Logs of Soil Borings shown in Figures 3 through 5 and Logs of Hand Auger Samples shown in Figure 6.

### 2.4 Groundwater

Groundwater was encountered at approximately 10 feet bgs of the borings performed at the school site on August 10, 2023. Groundwater was also encountered at approximately 9 feet bgs during previously performed explorations by our firm at the site in June of 2013.

To supplement our study, we reviewed available groundwater elevation data obtained from a California Department of Water Resources (DWR) monitoring well as identified as State Well Number 385021N1214948W001, located about two miles east of the site. The ground surface elevation at the well is +8 NAVD88, which is about zero to one foot higher than the subject site. Groundwater measurements obtained from the well indicate a "high" groundwater elevation of 1 foot NAVD88 (about 7 feet bgs at the well) occurred on April 11, 2023, and a "low" groundwater elevation of approximately -22 feet NAVD88 (about 30 feet bgs at the well) occurred on October 10, 1987.

## 3.0 CONCLUSIONS

### 3.1 2022 CBC and ASCE 7-16 Seismic Design Parameters

The 2022 California Building Code (CBC) currently references the American Society of Civil Engineers (ASCE) Standard 7-16 for seismic design. The seismic design parameters provided in Table 1 were developed based on a Site Classification D, and the latitude and longitude for the site using the web interface developed by the *Structural Engineers Association of California* (SEAOC) and *California's Health Care Access and Information (HCAI)*. Since $S_1$ is greater than 0.2g, the coefficient values $F_v$, $S_{M1}$, and $S_{D1}$ presented in Table 1 below are valid for this project, provided the requirements in Exception Note No. 2 in Section 11.4.8 of ASCE 7-16 apply. If not, a site-specific ground motion hazard analysis is required. However, based on our experience with similar structures we anticipate the exception will be met. However, this should be verified by the project structural engineer.

**Table 1: 2022 CBC/ASCE 7-16 Seismic Design Parameters**

| Latitude: 38.5018° N<br>Longitude: 121.5343° W | ASCE 7-16<br>Table/Figure | 2022 CBC<br>Table/Figure | Factor/Coefficient | Value |
|---|---|---|---|---|
| 0.2-second Period MCE | Figure 22-1 | Figure 1613.2.1(1) | $S_S$ | 0.62 g |
| 1.0-second Period MCE | Figure 22-2 | Figure 1613.2.1(3) | $S_1$ | 0.266 g |
| Soil Class | Table 20.3-1 | Section 1613.2.2 | Site Class | D |
| Site Coefficient | Table 11.4-1 | Table 1613.2.3(1) | $F_a$ | 1.304 |

| Latitude: 38.5018° N Longitude: 121.5343° W | ASCE 7-16 Table/Figure | 2022 CBC Table/Figure | Factor/Coefficient | Value |
|---|---|---|---|---|
| Site Coefficient | Table 11.4-2 | Table 1613.2.3(2) | $F_v$ | 2.068** |
| Adjusted MCE Spectral Response Parameters | Equation 11.4-1 | Equation 16-20 | $S_{MS}$ | 0.809 g |
| | Equation 11.4-2 | Equation 16-21 | $S_{M1}$ | 0.550 g* |
| Design Spectral Acceleration Parameters | Equation 11.4-3 | Equation 16-22 | $S_{DS}$ | 0.539 g |
| | Equation 11.4-4 | Equation 16-23 | $S_{D1}$ | 0.367 g* |
| Seismic Design Category | Table 11.6-1 | Section 1613.2.5(1) | Risk Category I through IV | D |
| | Table 11.6-2 | Section 1613.2.5(2) | Risk Category I through IV | D |

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

### 3.2 Liquefaction Potential

Liquefaction is a soil strength loss phenomenon that typically occurs in loose, saturated cohesionless sands 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 soil investigation and the groundwater conditions beneath the site. Hazards to buildings associated with liquefaction include shallow and deep foundation bearing capacity failure, lateral spreading of soil, and differential settlement of soils below foundations, all of which can contribute to structural damage or collapse.

The results of our subsurface soil exploration at the site indicate the underlying soils generally consist of interbedded sandy and silty layers extending to the maximum explored depth of 51½ feet below the existing ground surface. Based upon the relatively low risk of seismic activity, the presence of interbedded cohesive soils, and the lack of historic occurrence of liquefaction, it is our opinion that the potential for liquefaction of the soils beneath most of the site is relatively low. However, relatively loose sands were encountered at the boring locations and previous boring logs show groundwater at about four feet below the existing ground surface. These site conditions require than an evaluation of the liquefaction potential be performed at school sites per CGS Note 48 and Special Publication 117.

A liquefaction analysis to determine factors of safety against liquefaction was performed for the soil and groundwater conditions encountered at borings D1 and D106.

## 3.3 Liquefaction Analysis and Results

In performing our analyses, we used the soil liquefaction assessment software known as LiqSVs, Version 2.2 developed by GeoLogismiki that utilizes data collected from SPT blow counts and laboratory measurements of density and fine content 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 SPT blow counts performed during our field exploration. A design static groundwater level of approximately four feet below existing ground surfaces was used in our analysis based on the groundwater level encountered at the boring location and our review of historic groundwater levels. The peak ground acceleration ($PGA_M$) used in the liquefaction analysis consisted of 0.46g and A mode magnitude earthquake of 6.46 was used for this analysis based on deaggregation of site specific hazard analysis from USGS interactive hazard evaluation tool (Appendix C).

Our analysis of the SPT data indicates that the majority of the soils encountered in the upper 50 feet of the borings are liquefiable under considered earthquake. Liquefaction potential at the site can be 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, and not just discrete layers (Toprak and Holzer, 2003; Iwasaki, 1986). 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 low (0-5), high (5.1 to 15), or very high risk (>15). Surface manifestations of liquefaction occur at LPI ≥ 5. The LPI's and post-liquefaction settlement estimates for the soil conditions at the D1 and D106 based on Youd et al. (2001) calculation method are reported in Table 6.

**TABLE 6**
**LIQUEFACTION POTENTIAL AND SETTLEMENT**

| Exploration Identification | Calculated Liquefaction Potential Index (LPI) | Anticipated Seismic Settlement (inches) | Anticipated Lateral Spreading (feet) |
|---|---|---|---|
| D1 (2023) | 50 | 24 | 16 |
| D106 (2013) | 62 | 30 | 18 |

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

Because the current improvement does not include inhabitable structures mitigation measures for liquefaction is not required and provided herein.

### 3.4 Soil Expansion Potential

Laboratory tests performed on representative near surface clay samples revealed high plasticity when tested in accordance with the American Society of Testing and Materials (ASTM) International D4318 test method (see Figure A1). Additional laboratory testing of soils collected revealed the near-surface clay soils possesses “high” expansion potential when testing in accordance with ASTM D4829 test method (see Figure A2), which is consistent with the test results previously performed at the site.

Based on the laboratory test results, we conclude the native clays are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork.

Recommendations to mitigate the effects of potentially expansive clays, such as granular import material to construct the building pads, replacement of expansive clays with crushed rock, and deepened foundations are provided in this report.

### 3.5 Bearing Capacity

In our opinion, the native soils are capable of supporting the proposed improvements. Our experience in the area also indicates that engineered fills composed of native soils or approved import soils that are placed and compacted in accordance with general engineering practices will be suitable for support of the proposed improvements.

### 3.6 Pavement Subgrade Quality

Laboratory tests results indicate the surface and near-surface soil possesses Resistance ("R") values of 11 and 12 when tested in accordance with California Test 301 (Figure A3). Previous samples tested at the site in June of 2013 revealed similar R-values of 10. Based on the laboratory test results and our previous experience at the site with similar soil types we have selected an R-value of 10 for our design.

## 3.7 Groundwater Effect on Development

Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation, local pumping, and other factors.

Our experience indicates that groundwater should not be a significant factor in development of this site, provided excavations do not extend deeper than about 4 feet below the existing ground surface. Saturation of surface soils should be expected above these levels from rainfall, surface run-off, irrigation, or seepage from perched groundwater sources. In general, standard sump pit and pumping procedures should be adequate to control localized seepage. Excavations planned for utilities or other structures extending deeper than about 4 feet bgs will require more rigorous dewatering methods (such as well points or submersible pumps in slotted casings).

If excavations extend deeper than about 4 feet below the ground surface, dewatering may be required. 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.

## 3.8 Excavation Conditions

The surface and near-surface soils at the site should be readily excavatable with conventional earthmoving and trenching equipment. Based on our borings, excavations associated with building foundations, shallow trenches for utilities, and other excavations less than five feet deep associated with the proposed construction, should stand vertically for short periods of time (i.e. less than one day) required for construction. However, cohesionless, saturated or disturbed soils, if encountered, 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 California Occupational Safety and Health Administration (Cal/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.

### 3.9 Material Suitability for Engineered Fill Construction

The existing on-site native soils encountered at the boring locations are considered suitable for use as engineered fill construction, provided these materials do not contain significant quantities of organics, rubble and deleterious debris, and are at a proper moisture content capable of achieving the desired degree of compaction.

However, near-surface clays should not be used within the upper 12 inches of the final subgrade within interior and exterior slab-on-grade improvements. Imported materials, if necessary, should be granular and approved by our office prior to importing the materials to the site.

### 3.10 Preliminary Soil Corrosion Potential

One sample of near-surface soil was submitted to Sunland Analytical of Rancho Cordova, California, for testing to determine pH, chloride and sulfate concentrations, and minimum resistivity to help evaluate the potential for corrosive attack upon buried concrete. The results of the corrosivity testing are summarized below in Table 2. A copy of the test report is presented in Figure A4.

| TABLE 2: SOIL CORROSIVITY TESTING | | |
|---|---|---|
| Analyte | Test Method | Sample Identification |
| | | B2 (0-5') |
| pH | CA DOT 643 Modified* | 6.89 |
| Minimum Resistivity | CA DOT 643 Modified* | 1470 Ω-cm |
| Chloride | CA DOT 422 | 9.4 ppm |
| Sulfate | CA DOT 417 | 17.8 ppm |

Notes: * = Small cell method; Ω-cm = Ohm-centimeters; ppm = Parts per million

The California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, Corrosion Guidelines (Version 3.2, dated May 2021), considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 1500 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) 318-19, Section 19.3 – Concrete Durability Requirements, as referenced in Section 1904.1 of the 2022 CBC, indicates the severity of sulfate exposure for the sample tested is Exposure Class S0 (water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern). The project Structural Engineer should evaluate the requirements of ACI 318-19 and determine their applicability to the site.

Soil pH is 6.89, which is mildly acidic. Soil Resistivity is 1,470 ohm-centimeters, which is moderately corrosive. This soil is classified mildly to moderately corrosive to ferrous and other metals.

Universal Engineering Sciences 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.

# 4.0 RECOMMENDATIONS

## 4.1 General

The recommendations in this report are based on assumed excavations and fills on the order of about one to three feet for the development of the site, except for the construction of piles. 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 spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and spring months and will not be compactable without drying by aeration or chemical treatment. Soils present beneath existing slabs and pavements will be wet regardless of the time of year of construction. 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.

## 4.2 Site Clearing

Existing improvements to be abandoned, including but not limited to: existing pavements, foundations (if encountered), and underground utilities, should be completely removed from the site. Areas of new construction should also be cleared of vegetation and irrigation systems. Excavations to remove these items should extend to undisturbed native soils. All trees/large brush designated for removal, if any, should include the rootball and roots ½ inch or larger in size.

Where practical, the clearing should extend a minimum of five feet beyond the limits of the proposed structural areas of the site which include the new building, pavements and slab-on-grade concrete.

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

Existing pavements and flatwork (asphalt concrete and concrete), if any, that are not incorporated into the new design should be broken up and removed from the site. Alternatively, pulverized asphalt and Portland cement concrete rubble and any underlying aggregate base 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 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.

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.

### 4.3 Subgrade Preparation

Site clearing is expected to disturb the upper one to two feet of the site, and deeper disturbance will result where deeper underground utilities are removed or piers supporting pole mounted structures are removed. Subgrade preparation of the subgrade soils should include all soil that has been disturbed and/or areas where existing structures are removed to provide a uniform layer of engineered fill for support of the planned structures.

Due to the potential expansion characteristics of the native soils, the upper 12 inches of the final subgrade below the proposed building and exterior concrete flatwork, and the tennis courts should consist of imported non-expansive engineered fill.

Following site clearing and stripping operations, areas to receive fill or to remain at-grade should be scarified to a depth of at least 12 inches, moisture conditioned to at least two percent above the optimum moisture content and uniformly compacted to not less than 90 percent of the ASTM D1557 maximum dry density or to the highest degree possible for the soil moisture content and stability at the time of construction. Scarification and recompaction should extend at least five feet beyond the perimeter of structural areas and two feet beyond the outer edge of pavements. Unstable areas may require a layer of geotextile reinforcement at the time of construction. The need for geotextile reinforcement should be determined by the Geotechnical Engineer once the final subgrade has been exposed. If required, the building pad may be restored to grade with engineered fill compacted in lifts as recommended in this report. All fill soils should be compacted to at least 90 percent relative compaction.

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.

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.

### 4.4 Engineered Fill Construction

On-site soils are suitable for engineered fill construction in structural areas provided the materials do not contain rubbish, rubble greater than three inches, and significant organic concentrations. Imported fill materials, if required, should be compactable, granular soils with an Expansion Index of 20 or less, and contain no particles greater than three inches in maximum dimension. Imported soils should be approved by our office prior to being transported to the site. In addition, if required for fire lane or vehicular pavement areas, imported fill within the upper three feet of pavement areas should possess an R-value of at least 20. Also, if import fills are required (other than aggregate base), the contractor must provide appropriate documentation that the import is clean of known contamination per Department of Toxic Substances Control (DTSC) and within acceptable corrosion limits.

Engineered fill should be placed in lifts that do not exceed six inches in compacted thickness. Native or imported clayey materials should be thoroughly moisture conditioned to at least two percent above the optimum moisture content and uniformly compacted to at least 90 percent of the ASTM D1557 maximum dry density. Approved granular imported fill materials should be uniformly moisture conditioned to at least the optimum moisture content and compacted to at least 90 percent relative compaction. Relative compaction should be based on the ASTM D1557 maximum dry density.

The upper 12 inches of final building pad subgrades, including adjacent exterior flatwork areas, should consist of non-expansive granular on-site or import soils compacted to at least 90 percent relative compaction at the optimum moisture content or above. The upper 6 inches of tennis court subgrade should consist of ¾ inch crushed rock compacted to at least 90 percent relative compaction at the optimum moisture content or above.

The upper six inches of pavement subgrades should be uniformly compacted to at least 95 percent of the maximum dry density at a moisture content of at least two percent above the optimum moisture content, and must be stable under construction traffic prior to placement of aggregate base.

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

### 4.5 Engineered Fill Controlled Low Strength Material

If required, the use of Controlled Low Strength Material (CLSM) should be placed in accordance with Section 1803A.5.9 of the 2019 CBC. The CLSM should possess a compressive strength between 50 and 150 psf as determined by ASTM D4832. A minimum slump is not required for CLSM provided the material submittal is reviewed prior to use. Prior to placement, the area to receive the material should be clean of loose soil, water and debris and approved by a representative of the Geotechnical Engineer. The material should be submitted for review and approval by the Geotechnical Engineer prior to placement. Compressive strength testing of CLSM is not considered necessary provided the placement is observed by the Geotechnical Engineer and the CLSM used at the site is approved by the Geotechnical Engineer before being placed.

### 4.6 Utility Trench Backfill

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

Backfill for the upper 12 inches of trenches must match the adjacent materials. That is, if the upper 12 inches of subgrades for the building pad and exterior flatwork consists of granular fill materials, the top 12 inches of trench backfill should consist of the same materials or Class 2 aggregate base.

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

Additionally, trenches parallel to existing foundations should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

### 4.7 Foundation Design

The proposed structures may be supported upon a continuous perimeter foundation with continuous and/or isolated interior spread foundations embedded at least 24 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. For this project, 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. The project structural engineer should determine the final dimensions and structural reinforcement of the foundations.

Interior isolated spread foundations should be tied in two directions by tie/grade beam or slabs.

Foundations constructed within the building pads prepared as recommended may be sized utilizing a net allowable bearing capacity of 1500 pounds per square foot (psf) for dead plus live loads (based on a Factor of Safety of 2.0). This value may be increased by 1/3 to include wind or seismic forces. The weight of foundation concrete extending below the lowest adjacent soil grade may be disregarded in sizing computations.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.25, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 150 psf per foot of depth. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

### 4.8 Drilled. Cast-in-Place Reinforced Concrete Piers (Drilled Piers)

Drilled piers extending at least eight feet below the ground surface may be sized utilizing a maximum allowable vertical bearing capacity of 1,500 psf or an allowable skin friction of 200 psf for dead plus live loads, which may be applied over the surface of the pier. These values may be increased by one-third to include the short-term wind or seismic forces. The weight of foundation concrete below grade may be disregarded in sizing computations for the end-bearing condition.

Uplift resistance of pier foundations may be computed using the following resisting forces, where applicable: 1) weight of the pier concrete (150 pounds per cubic foot), and 2) the allowable skin friction of 100 psf applied over the shaft area of the pier. Increased uplift resistance can be achieved by increasing the diameter of the pier or increasing the depth.

The upper 12 inches of skin friction should be disregarded unless the pier is completely surrounded by concrete or pavements for a distance of at least three feet from the edge of the foundation pier.

Lateral resistance of pier foundations may be evaluated by applying a passive earth pressure of equivalent to a fluid pressure of 200 psf per foot of depth. The upper 12 inches of the subgrade should be disregarded for the non-constrained condition.

The structural engineer should determine if reinforcement of the piers is required and determine the reinforcing requirements. The bottom of the pier excavations should not contain loose or disturbed soils prior to placement of the concrete and reinforcement (if required). Cleaning of the bearing surface should be verified by the Geotechnical Engineer's representative prior to concrete placement. Concrete and reinforcement (if required) should be placed in the pier excavations as soon as possible, after the excavations are completed. The intent of this recommendation is to minimize the chances of sidewall caving into the excavations. Although we do not anticipate excessive sloughing of the sidewalls during pier construction, we recommend that the pier contractor be prepared to case the pier holes if conditions require.

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

Based on the explorations performed at the site and review of historical groundwater data pertinent to the site, excavations extending below approximately four feet BGS may encounter groundwater. If groundwater is encountered, groundwater likely will not be controlled, such that more than six inches of water accumulates at the bottom of the pier excavation. After it is confirmed that the excess water cannot be removed from the drilled pier excavation by bailing or with pumps, concrete should be placed using a tremie. For concrete placed using the tremie method, a design slump of six to eight inches, and a maximum aggregate size of ¾-inch is recommended. The required slump should be obtained by using plasticizers or water-reducing agents. Addition of water on-site to establish the recommended slump should not be allowed.

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

Sizing of piers to resist lateral loads can be evaluated using Section 1807.1 of the 2022 CBC. A value of 200 pcf for lateral bearing as defined in Table 1806.2 of the CBC may be used for the coefficients $S_1$ and $S_3$ for the non-constrained and constrained conditions, respectively. Per section 1806.1 of the 2022 CBC, an increase of 1/3 is permitted when using the alternate load combinations in Section 1605.3.2 that include wind or earthquake loads. The upper 12 inches of the subgrade should be disregarded for the non-constrained condition.

The bottom of the pier excavations should be free of loose or disturbed soils prior to placement of the concrete. Cleaning of the bearing surface may be done mechanically with the belling bucket, but should be verified by the geotechnical engineer prior to concrete placement. Reinforcement and concrete should be placed in the pier excavations as soon as possible after excavation is completed to reduce the potential of sidewall caving into the excavations.

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

### 4.9 Interior Floor Slab Support

Interior concrete slab-on-grade floors can be supported upon non-expansive imported materials soil subgrade prepared in accordance with the recommendations in this report and maintained in a moist condition and are protected from disturbance. If this is not the case and the subgrade soils become dry and/or disturbed, the building pad will require additional scarification, moisture conditioning and compaction prior to construction of the interior floor slabs.

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

Floor slabs that will receive moisture sensitive floor covering (e.g. vinyl covering, wood-laminate, etc.) should be underlain by a layer of free-draining crushed rock or gravel, serving as a deterrent to migration of capillary moisture. The gravel/crushed rock layer should be between four and six inches thick and graded such that 100 percent passes a one-inch sieve and no appreciable amount passes a No. 4 sieve. Additional moisture protection may be provided by placing a plastic, water vapor retarder (at least 15-mils thick) directly over the gravel/crushed rock. The water vapor retarder should meet or exceed the minimum specifications for plastic water vapor retarders as outlined in ASTM E1745 and be installed in strict conformance with the manufacturer's recommendations.

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

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

### 4.10 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.

### 4.11 Exterior Flatwork Construction (Non-Pavement)

The upper 12 inches of final soil subgrade for exterior concrete flatwork areas should consist of compactable, onsite native and/or imported very low-expansive (Expansion Index ≤ 20) granular soils placed and compacted in accordance with the Engineered Fill Construction recommendations included in this report. Exterior flatwork subgrade soils should be maintained in a moist condition and protected from disturbance.

Exterior flatwork should be underlain by at least four inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. The aggregate base can be included in the 12 inches of very-low expansive granular soils, or the very-low expansive layer can be completely composed off Class 2 aggregate base.

Exterior flatwork concrete should be at least four inches thick. Consideration should be given to thickening the edges of the slabs at least twice the slab thickness where wheel traffic is expected over the slabs. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of other structural elements by the placement of a layer of felt material between the flatwork and the structural element. Doweling of new flatwork into existing improvements (i.e., adjacent buildings, existing flatwork, etc.) is not recommended. The slab designer should determine the final thickness, strength and joint spacing of exterior slab-on-grade concrete. The slab designer should also determine if slab reinforcement for crack control is required and determine final slab reinforcing requirements.

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

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

### 4.12 Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from structures and prevent ponding of water adjacent to the foundations. The grade adjacent to the relocated structures should be sloped away from foundations at a minimum two percent slope for a distance of at least five feet, where possible. Ponding of surface water should not be allowed adjacent to the structure or exterior concrete flatwork.

### 4.13 Pavement Design

We are providing several pavement design alternative designs based on the soil conditions encountered at the site, our experience, and using design Traffic Indices (TIs) considered appropriate for the proposed construction.

Based on laboratory test results for the surface and near-surface clay soils present at the site and our experience in the area, we used a Resistance (“R”) value of 10 for pavement subgrades. Pavement sections presented in Table 3 have been calculated using the above R-values and traffic indices (TIs) assumed to be appropriate for this project. The procedures used for pavement design are in general conformance with Chapters 600 to 670 of the *California Highway Design Manual*, 7$^{th}$ Edition. The project civil engineer should determine the appropriate traffic index for pavements based on anticipated traffic conditions. If needed, we can provide additional pavement sections for different traffic indices.

**Table 3: Pavement Design Alternatives**

| Traffic Index (TI) | Pavement Use | Pavement Subgrade R-value = 10 | | |
|---|---|---|---|---|
| | | Asphalt Concrete (inches) | Class 2 Aggregate Base (inches) | Portland Cement Concrete (inches) |
| 4.5 | Automobile Parking | 2½* | 8 | -- |
| | | -- | 6 | 4 |
| 6.5 | Emergency Vehicles | 4* | 13 | -- |
| | | -- | 7 | 5 |

* = Asphalt concrete thickness contains the Caltrans safety factor.

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

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

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

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, PCC pavements should be used in areas subjected to concentrated heavy wheel loading, such as entryways, in front of trash enclosures, and/or within loading areas. Alternate PCC pavement sections have been provided above in Table 3.

We suggest concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards, latest edition. Reinforcing for crack control, if desired, should be provided in accordance with ACI guidelines. At a minimum, we recommend No. 3 reinforcing bars at 18 inches on center for crack control. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform to the current PCA or ACI guidelines. PCC should achieve a minimum compressive strength of 3,500 pounds per square inch at 28 days.

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

### 4.14 Geotechnical Engineering Construction Observation Services

Site preparation should be accomplished in accordance with the recommendations of this report. Representatives of the Geotechnical Engineer should be present during site preparation and all grading operations to observe and test the fill to verify compliance with our recommendations and the job specifications. Testing frequency will depend on how the site is graded and should be determined during the rough grading operations. These services are beyond the scope of work authorized for this investigation.

In the event that Universal Engineering Sciences 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. A final report by the Geotechnical Engineer providing construction testing services should be prepared upon completion of the project.

### 4.15 Additional Services

Our firm should 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.

## 5.0 LIMITATIONS

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

If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at the boring locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

We emphasize that this report is applicable only to the proposed construction and the investigated site, and should not be utilized for construction on any other site.

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

**Universal Engineering Sciences (UES)**

R. Foroozan
Roozbeh Foroozan, PhD, PE
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
ROOZBEH FOROOZAN
No. 94770
EXP.06/30/2025
CIVIL
STATE OF CALIFORNIA

Dharmesh Amin
Dharmesh Amin, MS, PE, GE
Regional Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
DHARMESH AMIN
2553
GEOTECHNICAL
STATE OF CALIFORNIA

Kathlyn Ortega
Kathlyn Ortega, MS
Staff Geologist

**FIGURES**


*Spatial Data provided by Esri, NOAA, and USGS.*
*Projection: NAD 1983 2011 StatePlane California II FIPS 0402 Ft US*

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**VICINITY MAP**

JOHN F. KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS

Sacramento, California

| FIGURE | 1 |
|---|---|
| DRAWN BY | KO |
| CHECKED BY | JRY |
| PROJECT MGR | JRY |
| DATE | 10/2023 |
| 4630.2300076.0016 | |


*Aerial imagery provided by Esri.*
*Projection: NAD 1983 2011 StatePlane California II FIPS 0402 Ft US*

UES

**SITE PLAN**

JOHN F. KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS

Sacramento, California

| | |
|---|---|
| FIGURE | 2 |
| DRAWN BY | GHZ |
| CHECKED BY | JRY |
| PROJECT MGR | JRY |
| DATE | 10/2023 |
| 4630.2300076.0016 | |

| | |
|---|---|
| **Project:** | **John F. Kennedy High School Athletic Improvements** |
| **Project Location:** | **Sacramento, California** |
| **Project Number:** | **4630.2300076.0016** |

# LOG OF SOIL BORING D1

Sheet 1 of 1

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | **8/10/23** | Logged By | **GHZ** | Checked By | **JRY** |
| Drilling Method | **Hollow Stem Auger** | Drilling Contractor | **V&W Drilling** | Total Depth of Drill Hole | **51.5 feet** |
| Drill Rig Type | **CME 75** | Diameter(s) of Hole, inches | **8"** | Approx. Surface Elevation, ft MSL | **7.0** |
| Groundwater Depth [Elevation], feet | **10.0 [-3.0]** | Sampling Method(s) | **2.0" Modified California with 6-inch sleeve** | Drill Hole Backfill | **Neat Cement** |
| Remarks | **Bulk (0-5'); RV = 11** | | | 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 |
|---|---|---|---|---|---|---|---|
| 5 | | Brown, moist, stiff, silty lean CLAY (CL) with some fine sand | | | | | |
| | | grayish brown | D1-1I | 6 | | | PP = 1.25tsf |
| 0 | 5 | | D1-2I | 4 | | | PP = 0.5tsf |
| -5 | 10 | gray, wet, very soft, fine to coarse sand | D1-3I | 0 | | | PP = 0.1tsf |
| -10 | 15 | sandy, medium stiff | D1-4I | 8 | | | PP = 0.5tsf |
| -15 | 20 | very soft | D1-5I | 0 | | | PP = 0.25tsf |
| | | Gray, wet, very loose, sandy SILT (ML) | | | | | |
| -20 | 25 | | D1-6I | 0 | | | |
| -25 | 30 | | D1-7I | 3 | | | |
| | | Gray, wet, loose, silty fine SAND (SM) | | | | | |
| -30 | 35 | | D1-8I | 5 | | | |
| -35 | 40 | | D1-9I | 4 | | | |
| -40 | 45 | | D1-10I | 3 | | | |
| | 50 | | D1-11I | 9 | | | |

Boring was terminated at approximately 51½ feet below existing ground surface.
Groundwater encountered at approximately 10 feet below existing ground surface.

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**FIGURE 3**

| Project: John F. Kennedy High School Athletic Improvements | LOG OF SOIL BORING D2 |
|---|---|
| Project Location: Sacramento, California | Sheet 1 of 1 |
| Project Number: 4630.2300076.0016 | |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | **8/10/23** | Logged By | **GHZ** | Checked By | **JRY** |
| Drilling Method | **Hollow Stem Auger** | Drilling Contractor | **V&W Drilling** | Total Depth of Drill Hole | **61.5 feet** |
| Drill Rig Type | **CME 55 HT** | Diameter(s) of Hole, inches | **7"** | Approx. Surface Elevation, ft MSL | **8.0** |
| Groundwater Depth [Elevation], feet | **10.0 [-2.0]** | Sampling Method(s) | **2.0" Modified California with 6-inch sleeve** | Drill Hole Backfill | **Neat Cement** |
| Remarks | **Bulk (0-5'); PI = 20, EI = 93, CR** | | | Driving Method and Drop | **140lb auto. hammer with 30" drop** |

| ELEVATION, feet | DEPTH, feet | ENGINEERING CLASSIFICATION AND DESCRIPTION | SAMPLE NUMBER (SAMPLE DATA) | NUMBER OF BLOWS (SAMPLE DATA) | MOISTURE CONTENT, % (TEST DATA) | DRY UNIT WEIGHT, pcf (TEST DATA) | ADDITIONAL TESTS (TEST DATA) |
|---|---|---|---|---|---|---|---|
| | | Dark brown, moist, very stiff, fat CLAY (CH) | | | | | |
| 5 | | | D2 (0-5') D2-1I | 13 | 27.9 | 78 | PP = 3.5tsf |
| | 5 | medium stiff, increased moisture content | D2-2I | 3 | 59.7 | 61 | PP = 0.5tsf |
| 0 | | | | | | | |
| | 10 | Dark brown, wet, very soft, lean CLAY (CL) with sand | D2-3I | 2 | | | FC = 86.1% |
| -5 | | Dark brown, wet, very loose, clayey fine SAND (SC) | | | | | |
| | 15 | | D2-4I | 4 | | | FC = 49.7% |
| -10 | | | | | | | |
| | 20 | | D2-5I | 4 | | | FC = 48.9% |
| -15 | | Dark brown, wet, very loose, sandy SILT (ML) | | | | | |
| | 25 | | D2-6I | 3 | | | FC = 57.0% |
| -20 | | | | | | | |
| | 30 | | D2-7I | 4 | | | FC = 57.9% |
| -25 | | | | | | | |
| | 35 | | D2-8I | 2 | | | FC = 60.0% |
| -30 | | | | | | | |
| | 40 | brown | D2-9I | 4 | | | FC = 69.7% |
| -35 | | | | | | | |
| | 45 | loose | D2-10I | 6 | | | FC = 93% |
| -40 | | | | | | | |
| | 50 | grayish brown | D2-11I | 5 | | | FC = 52.9% |
| -45 | | | | | | | |
| | 55 | very loose | D2-12I | 3 | | | FC = 68.9% |
| -50 | | Grayish brown, wet, silty fine SAND (SM) | | | | | |
| | 60 | | D2-13I | 12 | | | |

Boring was terminated at approximately 61½ feet below existing ground surface.
Groundwater encountered at approximately 10 feet below existing ground surface.

UES™

FIGURE 4

**Project:** John F. Kennedy High School Athletic Improvements
**Project Location:** Sacramento, California
**Project Number:** 4630.2300076.0016

# LOG OF SOIL BORING D3



| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 8/10/23 | Logged By | GHZ | Checked By | JRY |
| Drilling Method | Hollow Stem Auger | Drilling Contractor | V&W Drilling | Total Depth of Drill Hole | 50.0 feet |
| Drill Rig Type | CME 55 HT | Diameter(s) of Hole, inches | 7" | Approx. Surface Elevation, ft MSL | 8.0 |
| Groundwater Depth [Elevation], feet | 10.0 [-2.0] | Sampling Method(s) | 2.0" Modified California with 6-inch sleeve | Drill Hole Backfill | Neat Cement |
| Remarks | Bulk (0-5') | | | Driving Method and Drop | 140lb auto. hammer with 30" drop |

| ELEVATION, feet | DEPTH, feet | ENGINEERING CLASSIFICATION AND DESCRIPTION | SAMPLE NUMBER (SAMPLE DATA) | NUMBER OF BLOWS (SAMPLE DATA) | MOISTURE CONTENT, % (TEST DATA) | DRY UNIT WEIGHT, pcf (TEST DATA) | ADDITIONAL TESTS (TEST DATA) |
|---|---|---|---|---|---|---|---|
| 8 | 0 | Light brown, moist, medium stiff, lean CLAY (CL) | D3-1I | 7 | | | |
| 3 | 5 | soft, with sand | D3-2I | 2 | | | |
| 0 | 8 | Grayish brown, wet, loose, clayey fine SAND (SC) | D3-3I | 2 | | | |
| -5 | 15 | | D3-4I | 5 | | | |
| -10 | 17 | Dark gray, wet, loose, sandy SILT (ML); fine sand | D3-5I | 3 | | | |
| -15 | 25 | very loose | D3-6I | 0 | | | |
| -20 | 30 | | D3-7I | 1 | | | |
| -25 | 35 | | D3-8I | 3 | | | |
| -30 | 40 | | D3-9I | 3 | | | |
| -35 | 42 | Gray, wet, very loose, silty fine SAND (SM) | D3-10I | 3 | | | |
| -40 | 50 | dark gray, loose | D3-11I | 7 | | | |

Boring was terminated at approximately 50 feet below existing ground surface.
Groundwater encountered at approximately 10 feet below existing ground surface.

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FIGURE 5

Hand Auger HA1

0 – 5" 5 inches Asphalt Concrete

5" – 10" 5 inches Aggregate Base

10" – 5’ Brown, moist, stiff, lean CLAY (CL) with some fine sand

Hand auger sampling terminated at approximately 5 feet below the ground surface.
No groundwater was encountered.

Hand Auger HA2

0 – 5" 5 inches Asphalt Concrete

5" – 10" 5 inches Aggregate Base

10" – 5’ Brown, moist, stiff, lean CLAY (CL) with some fine sand

Hand auger sampling terminated at approximately 5 feet below the ground surface.
No groundwater was encountered.

UES™

**LOGS OF HAND AUGER SAMPLES**

JOHN F. KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS

Sacramento□C□li□□r□i□

| FIGURE | 6 |
|---|---|
| DRAWN BY | KRO |
| CHECKED BY | JRY |
| PROJECT MGR | JRY |
| DATE | 09/2023 |

4630.2300076.0016

# 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)<br>coarse<br>fine | 3" to No. 4<br>3" to 3/4"<br>3/4" to No. 4 | 75 to 4.75<br>75 to 19<br>19 to 4.75 |
| SAND<br>coarse<br>medium<br>fine | No. 4 to No. 200<br>No. 4 to No. 10<br>No. 10 to No. 40<br>No. 40 to No. 200 | 4.75 to 0.075<br>4.75 to 2.00<br>2.00 to 0.425<br>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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[illegible]

JOHN F. KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS

Sacramento, California

| | |
|---|---|
| FIGURE | 7 |
| DRAWN BY | KO |
| CHECKED BY | JRY |
| PROJECT MGR | JRY |
| DATE | 09/2023 |
| 4630.2300076.0016 | |

**APPENDIX A**

**General Project Information, Laboratory Testing and Results**

# APPENDIX A

## A. GENERAL INFORMATION

The performance of a geotechnical engineering study for the proposed John F. Kennedy High School Athletic Improvements project located at John F. Kennedy High School in Sacramento, California was authorized by Chris Sullivan of Verde Designs, Inc. on July 18, 2023. Authorization was for a study as described in our proposal letter dated March 10, 2023, sent to Verde Designs, Inc., whose mailing address is 1024 Iron Point Road, Suite 100, Folsom, California, 95630.

## B. FIELD EXPLORATIONS

As part of our study for the proposed improvements, our field exploration included drilling and sampling of 3 borings (D1 through D3) and 2 hand auger samples (HA1 and HA2) at the approximate locations shown on Figure 2.

The soil borings D1 through D3 were performed on August 10, 2023, to depths ranging from about 50 to 61½ feet below existing site grades utilizing a CME-75 truck-mounted drilling rig equipped with six-inch-diameter solid flight augers. Soil samples were recovered at various intervals with a 2½-inch outside diameter (O.D.), 2-inch inside diameter (I.D.), modified California split-spoon sampler. The sampler was driven by an automatic 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive each six-inch interval of the 18-inch long samplers were recorded. The sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, is designated the penetration resistance or "blow count" for that particular drive.

The modified California samples were retained in 2-inch diameter by 6-inch long, thin-walled brass tubes contained within the sampler. After recovery, the field representative visually classified the soil recovered in the tubes. After the samples were classified, the ends of the tubes were sealed to preserve the natural moisture contents.

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

Pocket penetrometer testing was performed during drilling operations on select cohesive soil samples obtained at the boring locations. In pocket penetrometer testing, the unconfined compressive strength of a cohesive soil sample is estimated by measuring the resistance of the sample to penetration of a relatively small, calibrated, spring-loaded cylinder. The maximum capacity of the penetrometer is 4.5 tons per square-foot (tsf). The unconfined compressive strength estimated from pocket penetrometer testing on the select cohesive soil samples is included on the boring logs at the depth the sample tested was obtained. The approximate undrained shear strength of the samples tested is one-half of the unconfined compressive strength.

Samples HA1 and HA2 were also collected on August 10, 2023 utilizing a hand auger to depths of about 5 feet below existing site grades.

Descriptions of the soils encountered in the boring locations are presented on Figures 3 through 6. An explanation of the Unified Soil Classification System symbols used in the descriptions is presented on Figure 7.

## C. LABORATORY TESTING

One representative near-surface sample was subjected to Atterberg Limits tests (ASTM D4318). The results of this test are presented in Figure A1.

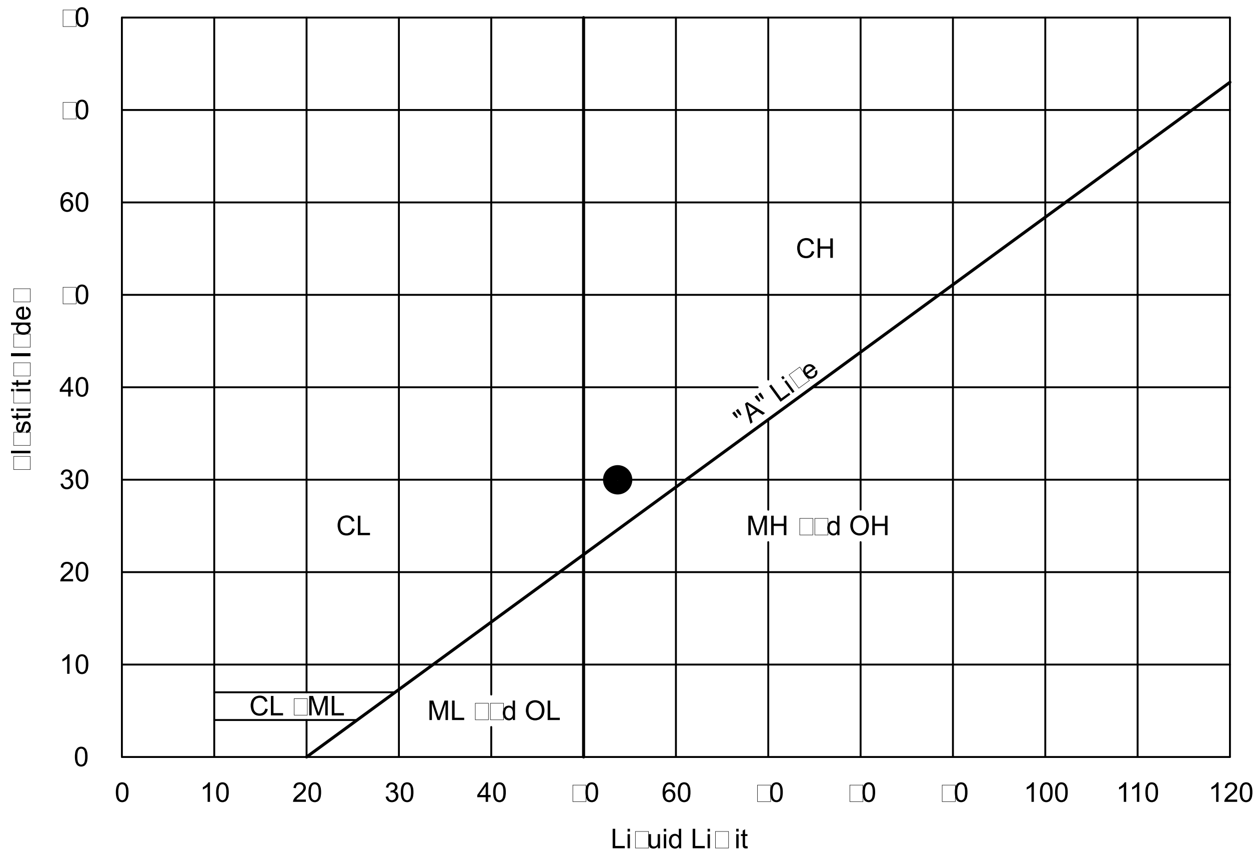
One representative near-surface soil sample was tested for Expansion Index (ASTM D4829) with results presented in Figure A2.

Two representative samples of near-surface soil were subjected to Resistance-value ("R") testing in accordance with California Test 301. The results of the R-value tests are presented in Figure A3.

One sample of the near-surface soil was submitted to Sunland Analytical to determine the soil pH, minimum resistivity (California Test 643), Sulfate concentration (California Test 417) and Chloride concentration (California Test 422). The results of these tests are presented on Figures A4.

# ATTERBERG LIMITS

## ASTM D4318




| KEY SYMBOL | LOCATION | SAMPLE DEPTH | NATURAL WATER CONTENT [illegible] | ATTERBERG LIMITS | | PASSING NO. 200 SIEVE [illegible] | UNIFIED SOIL CLASSIFICATION SYMBOL |
|---|---|---|---|---|---|---|---|
| | | | | LIQUID LIMIT [illegible] | PLASTICITY INDEX [illegible] | | |
| ● | D2 | 0[illegible].0[illegible] | [illegible] | [illegible]3 | 30 | [illegible] | CH |

[illegible]

JOHN F. KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS

[illegible]

| FIGURE | A1 |
|---|---|
| DRAWN BY | GH[illegible] |
| CHECKED BY | [illegible]R[illegible] |
| PROJECT MGR | [illegible]R[illegible] |
| DATE | 10[illegible]2023 |

4630.23000[illegible]6.0016

# EXPANSION INDEX TEST RESULTS

## ASTM D4829

MATERIAL DESCRIPTION: Dark Brown Fat CLAY (CH)

LOCATION: D2

| Sample Depth | Pre-Test Moisture (%) | Post-Test Moisture (%) | Dry Density (pcf) | Expansion Index |
|---|---|---|---|---|
| 0 [illegible].0[illegible] | 1[illegible].[illegible] | 33.3 | 10[illegible] | [illegible] |

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

| | | | |
|---|---|---|---|
| UES | [illegible] | FIGURE | A2 |
| | JOHN F. KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS | DRAWN BY | GH[illegible] |
| | Sacramento, California | CHECKED BY | [illegible]R[illegible] |
| | | PROJECT MGR | [illegible]R[illegible] |
| | | DATE | 10/2023 |
| | | 4630.23000[illegible]6.0016 | |

# RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Br□□ □□le□□ CLA□ □CL□with some fine sand

LOCATION: HA2 □1□3□□

| Specimen No. | Dry Unit Weight (pcf) | Moisture @ Compaction (%) | Exudation Pressure (psi) | Expansion (dial, inches x 1000) | Expansion (psf) | R Value |
|---|---|---|---|---|---|---|
| 1 | □6 | 2□.0 | 1□□ | 4 | 1□ | 6 |
| 2 | □6 | 24.3 | 2□1 | 1□ | □□ | 12 |
| 3 | □6 | 23.□ | 46□ | 36 | 1□6 | 24 |

R□V□lue □t 300 □si e□ud□ti□□ □ressure □ **12**

MATERIAL DESCRIPTION: Br□□ □□silt□le□□ CLA□ □CL□

LOCATION: D1 □0□□□

| Specimen No. | Dry Unit Weight (pcf) | Moisture @ Compaction (%) | Exudation Pressure (psi) | Expansion (dial, inches x 1000) | Expansion (psf) | R Value |
|---|---|---|---|---|---|---|
| 1 | □□ | 23.□ | 2□4 | 1□ | 6□ | □ |
| 2 | □□ | 22.□ | □4□ | 4□ | 204 | 34 |
| 3 | □□ | 23.1 | 3□0 | 26 | 113 | 1□ |

R□V□lue □t 300 □si e□ud□ti□□ □ressure □ **11**

UES™

**RESISTANCE VALUE TEST RESULTS**

□OHN F. KENNED□ HIGH SCHOOL ATHLETIC IM□ROVEMENTS

S□□r□□ e□t□□C□li□r□i□

| FIGURE | A3 |
|---|---|
| DRAWN BY | GH□ |
| CHECKED BY | □R□ |
| PROJECT MGR | □R□ |
| DATE | 10/2023 |
| 4630.23000□6.0016 | |

# *Sunland Analytical*

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

Date Reported 08/25/2023
Date Submitted 08/21/2023

To: Guang Zhu
Universal Engineering Science
3050 Industrial Blvd
West Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 4630.2300076.0016 Site ID : D2 0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 90381-187577.

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**EVALUATION FOR SOIL CORROSION**

| | | | |
|---|---|---|---|
| Soil pH | 6.89 | | |
| Minimum Resistivity | 1.47 ohm-cm (x1000) | | |
| Chloride | 9.4 ppm | 00.00094 | % |
| Sulfate | 17.8 ppm | 00.00178 | % |

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

| | | | |
|---|---|---|---|
| UES™ | **CORROSION TEST RESULTS**<br>□OHN F. KENNED□ HIGH SCHOOL ATHLETIC FIELD IM□ROVEMENTS<br>S□□r□□e□t□□C□li□□r□i□ | FIGURE | A4 |
| | | DRAWN BY | GH□ |
| | | CHECKED BY | □R□ |
| | | PROJECT MGR | □R□ |
| | | DATE | 10□2023 |
| | | 4630.23000□6.0016 | |

# APPENDIX B

## Previous Logs of Soil Borings

| | |
|---|---|
| **Project:** | **Kennedy High School Athletic Improvements** |
| **Project Location:** | **Sacramento, California** |
| **WKA Number:** | **7915.05P** |

# LOG OF SOIL BORING D101

Sheet 1 of 1

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | **6/7/13** | Logged By | **Joe Follettie** | Checked By | **MSM** |
| Drilling Method | **Solid Flight Augers** | Drilling Contractor | **Wallace Kuhl & Associates** | Total Depth of Drill Hole | **10.5 feet** |
| Drill Rig Type | **WKA John Deere 4x6 Gator** | Diameter(s) of Hole, inches | **4"** | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | **4.5** | Sampling Method(s) | **California Modified** | Drill Hole Backfill | **soil cuttings** |
| Remarks | | | | Driving Method and Drop | **70-lb slide hammer** |

| ELEVATION, feet | DEPTH, feet | GRAPHIC LOG | ENGINEERING CLASSIFICATION AND DESCRIPTION | SAMPLE DATA: SAMPLE | SAMPLE NUMBER | NUMBER OF BLOWS | TEST DATA: MOISTURE CONTENT, % | DRY UNIT WEIGHT, pcf | ADDITIONAL TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | | Brown, slightly moist, sandy silt with gravel (ML) | | | | | | |
| | | | Gray brown, moist to wet, silty clay (CL) | | | | | | |
| | | | | ■ | D101-1I | | 31.3 | 81 | |
| | 5 | | | ■ | D101-2I | | 41.0 | 72 | UCC= 1.6 TSF |
| | 10 | | | ■ | D101-3I | | | | |
| | | | Boring terminated at 10.5 feet | | | | | | |

BORING LOG 7915.05P - KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS.GPJ WKA.GDT 8/14/13 9:29 AM

Wallace Kuhl & ASSOCIATES

**FIGURE 9**

| | |
|---|---|
| **Project: Kennedy High School Athletic Improvements**<br>**Project Location: Sacramento, California**<br>**WKA Number: 7915.05P** | **LOG OF SOIL BORING D102**<br>Sheet 1 of 1 |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 6/7/13 | Logged By | Joe Follettie | Checked By | MSM |
| Drilling Method | Solid Flight Augers | Drilling Contractor | Wallace Kuhl & Associates | Total Depth of Drill Hole | 10.5 feet |
| Drill Rig Type | WKA John Deere 4x6 Gator | Diameter(s) of Hole, inches | 4" | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | 4.5 | Sampling Method(s) | California Modified | Drill Hole Backfill | soil cuttings |
| Remarks | | | | Driving Method and Drop | 70-lb slide hammer |

| ELEVATION, feet | DEPTH, feet | GRAPHIC LOG | ENGINEERING CLASSIFICATION AND DESCRIPTION | SAMPLE DATA: SAMPLE | SAMPLE NUMBER | NUMBER OF BLOWS | TEST DATA: MOISTURE CONTENT, % | DRY UNIT WEIGHT, pcf | ADDITIONAL TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | | Brown, slightly moist, sandy silt with gravel (ML) | | | | | | |
| | | | Dark brown, moist to wet, silty clay (CL) | | | | | | |
| | | | | | D102-1I | | 34.6 | 79 | |
| | 5 | | | | D102-2I | | 47.2 | 71 | UCC= 0.7 tsf |
| | 10 | | Dark brown, wet, clayey, fine sandy silt (ML) | | D102-3I | | | | |



Wallace Kuhl & ASSOCIATES

**FIGURE 10**

| Project: Kennedy High School Athletic Improvements<br>Project Location: Sacramento, California<br>WKA Number: 7915.05P | **LOG OF SOIL BORING D103**<br>Sheet 1 of 1 |
|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 6/12/13 | Logged By | Joe Follettie | Checked By | MSM |
| Drilling Method | Solid Flight Augers | Drilling Contractor | V & W Drilling | Total Depth of Drill Hole | 20.0 feet |
| Drill Rig Type | CME 75 | Diameter(s) of Hole, inches | 6" | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | 4.5 | Sampling Method(s) | California Modified | Drill Hole Backfill | Neat Cement |
| Remarks | w.o.h = weight of hammer | | | Driving Method and Drop | 140 lb automatic hammer |

| 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 | | Brown, very moist to wet, soft, clayey silt (ML) | | | | | | |
| | 4.5 | | ▽ | ■ | D103-1I | 3 | 45.4 | 70 | |
| | 5 | | Gray, wet, soft, silty clay (CL) | | | | | | |
| | | | Gray brown, wet, very loose, sandy silt (ML) | | | | | | |
| | 10 | | | ■ | D103-2I | w.o.h. | | | |
| | | | Gray, wet, very loose, sandy silt (ML) | | | | | | |
| | 15 | | | ■ | D103-3I | 2 | 23.9 | 102 | |
| | 20 | | | ■ | D103-4I | 2 | | | |

Boring terminated at 20 feet



Wallace Kuhl & ASSOCIATES

**FIGURE 11**

**Project:** Kennedy High School Athletic Improvements
**Project Location:** Sacramento, California
**WKA Number:** 7915.05P

# LOG OF SOIL BORING D104



| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 6/12/13 | Logged By | Joe Follettie | Checked By | MSM |
| Drilling Method | Solid Flight Augers | Drilling Contractor | V & W Drilling | Total Depth of Drill Hole | 21.5 feet |
| Drill Rig Type | CME 75 | Diameter(s) of Hole, inches | 6" | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | 4.5 | Sampling Method(s) | California Modified | Drill Hole Backfill | Neat Cement |
| Remarks | w.o.h = weight of hammer | | | Driving Method and Drop | 140 lb automatic hammer |

| 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 | | Brown, very moist to wet, very soft, clayey silt (ML) | | | | | | |
| | 5 | | Brown, wet, very loose silty fine sand (SM) | | D104-1I | w.o.h | 45.3 | 77 | TR |
| | 10 | | Gray, wet, very loose, sandy silt (ML) | | D104-2I | 2 | | | |
| | 15 | | | | D104-3I | 4 | | | |
| | 20 | | | | D104-4I | w.o.h | 32.8 | 89 | |
| | 21.5 | | Boring terminated at 21.5 feet | | | | | | |

BORING LOG 7915.05P - KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS.GPJ WKA.GDT 8/14/13 9:29 AM

Wallace Kuhl & Associates

FIGURE 12

| | |
|---|---|
| **Project:** Kennedy High School Athletic Improvements<br>**Project Location:** Sacramento, California<br>**WKA Number:** 7915.05P | **LOG OF SOIL BORING D105**<br>Sheet 1 of 1 |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 6/12/13 | Logged By | Joe Follettie | Checked By | MSM |
| Drilling Method | Solid Flight Augers | Drilling Contractor | V & W Drilling | Total Depth of Drill Hole | 20.0 feet |
| Drill Rig Type | CME 75 | Diameter(s) of Hole, inches | 6" | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | 4.5 | Sampling Method(s) | California Modified | Drill Hole Backfill | Neat Cement |
| Remarks | | | | Driving Method and Drop | 140 lb automatic hammer |

| ELEVATION, feet | DEPTH, feet | ENGINEERING CLASSIFICATION AND DESCRIPTION | SAMPLE NUMBER | NUMBER OF BLOWS | MOISTURE CONTENT, % | DRY UNIT WEIGHT, pcf | ADDITIONAL TESTS |
|---|---|---|---|---|---|---|---|
| | 0 | Brown, slightly moist, silty fine to coarse sand (SM) | | | | | |
| | | Brown, very moist to wet, very soft, clayey silt (ML) | | | | | |
| | 5 | | D105-1I | 2 | 47.5 | 73 | |
| | 8 | Gray, wet, very loose, fine sandy silt (ML) | | | | | |
| | 10 | | D105-2I | 1 | | | |
| | 13 | Gray, wet, very loose, silty fine sand (SM) | | | | | |
| | 15 | | D105-3I | 3 | | | |
| | 20 | | D105-4I | 2 | 25.8 | 99 | |

Boring terminated at 20 feet

BORING LOG 7915.05P - KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS.GPJ WKA.GDT 8/14/13 9:29 AM

Wallace Kuhl & ASSOCIATES

**FIGURE 13**

| | |
|---|---|
| **Project: Kennedy High School Athletic Improvements**<br>**Project Location: Sacramento, California**<br>**WKA Number: 7915.05P** | **LOG OF SOIL BORING D106**<br>Sheet 1 of 2 |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | **6/12/13** | Logged By | **Joe Follettie** | Checked By | **MSM** |
| Drilling Method | **Hollow Stem Auger** | Drilling Contractor | **V & W Drilling** | Total Depth of Drill Hole | **51.5 feet** |
| Drill Rig Type | **CME 75** | Diameter(s) of Hole, inches | **6"** | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | **4.5** | Sampling Method(s) | **Standard Penetration Test** | Drill Hole Backfill | **Neat Cement** |
| Remarks | **w.o.h = weight of hammer** | | | Driving Method and Drop | **140 lb automatic hammer** |

| 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 | | Brown, very moist to wet, very soft, clayey silt (ML) | | | | | | |
| | 5 | | | | D106-I1 | w.o.h. | | | |
| | 10 | | Gray, wet, loose to very loose, fine sandy silt (ML) | | D106-2I | 5 | | | |
| | 15 | | | | D106-3I | 3 | | | <#200= 55.3% |
| | 20 | | | | D106-4I | w.o.h. | | | <#200= 51.0% |
| | 25 | | | | D106-5I | w.o.h. | | | <#200= 54.7% |



Wallace Kuhl & ASSOCIATES

**FIGURE 14**

| Project: Kennedy High School Athletic Improvements | LOG OF SOIL BORING D106 |
|---|---|
| Project Location: Sacramento, California | Sheet 2 of 2 |
| WKA Number: 7915.05P | |

| 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 |
|---|---|---|---|---|---|---|---|---|---|
| | 30 | | | | D106-6I | 2 | | | |
| | 35 | | Gray, wet, very loose, silty fine sand (SM) | | D106-7I | 1 | | | <#200= 52.2% |
| | 40 | | | | D106-8I | 4 | | | |
| | 45 | | | | D106-9I | 7 | | | <#200= 47.9% |
| | 50 | | | | D106-10I | 5 | | | |
| | | | Boring terminated at 51.5 feet | | | | | | |

| | |
|---|---|
| **Project:** Kennedy High School Athletic Improvements<br>**Project Location:** Sacramento, California<br>**WKA Number:** 7915.05P | **LOG OF SOIL BORING D107**<br>Sheet 1 of 1 |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 6/12/13 | Logged By | Joe Follettie | Checked By | MSM |
| Drilling Method | Solid Flight Augers | Drilling Contractor | V & W Drilling | Total Depth of Drill Hole | 21.5 feet |
| Drill Rig Type | CME 75 | Diameter(s) of Hole, inches | 6" | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | 4.5 | Sampling Method(s) | California Modified | Drill Hole Backfill | Neat Cement |
| Remarks | | | | Driving Method and Drop | 140 lb automatic hammer |

| 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 | | Brown, slightly moist, silty fine to coarse sand (SM) | | | | | | |
| | | | Brown, very moist to wet, very loose, fine sandy silt (ML) | | | | | | |
| | 5 | | | | D107-1I | 1 | 41.6 | 74 | |
| | 10 | | Gray, wet, stiff, silty clay (CL) | | D107-2I | 7 | 32.8 | 89 | |
| | 15 | | Gray, wet, loose, silty fine sand (SM) | | D107-3I | 5 | | | |
| | 20 | | | | D107-4I | 1 | | | |
| | | | Boring terminated at 21.5 feet | | | | | | |

| Project: Kennedy High School Athletic Improvements | LOG OF SOIL BORING D108 |
|---|---|
| Project Location: Sacramento, California | |
| WKA Number: 7915.05P | Sheet 1 of 1 |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | 6/12/13 | Logged By | Joe Follettie | Checked By | MSM |
| Drilling Method | Hollow Stem Auger | Drilling Contractor | V & W Drilling | Total Depth of Drill Hole | 26.5 feet |
| Drill Rig Type | CME 75 | Diameter(s) of Hole, inches | 6" | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | 4.5 | Sampling Method(s) | California Modified | Drill Hole Backfill | Neat Cement |
| Remarks | | | | Driving Method and Drop | 140 lb automatic 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 | | Brown, slightly moist, silty fine to coarse sand (SM) | | | | | | |
| | | | Brown, very moist to wet, very soft, clayey silt (ML) | | | | | | |
| | 5 | | | | D108-1I | 2 | 42.6 | 77 | |
| | 10 | | Gray brown, wet, very loose, fine sandy silt (ML) | | D108-2I | 2 | | | |
| | 15 | | Gray, wet, very loose, silty fine sand (SM) | | D108-3I | 2 | 26.1 | 97 | |
| | 20 | | | | D108-4I | w.o.h. | | | |
| | 25 | | | | D108-5I | 1 | 24.6 | 105 | TR |
| | | | Boring terminated at 26.5 feet | | | | | | |

BORING LOG 7915.05P - KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS.GPJ WKA.GDT 8/14/13 9:29 AM

Wallace Kuhl & ASSOCIATES

FIGURE 16

| | |
|---|---|
| **Project: Kennedy High School Athletic Improvements**<br>**Project Location: Sacramento, California**<br>**WKA Number: 7915.05P** | **LOG OF SOIL BORING D109**<br>Sheet 1 of 1 |

| | | | | | |
|---|---|---|---|---|---|
| Date(s) Drilled | **6/12/13** | Logged By | **Joe Follettie** | Checked By | **MSM** |
| Drilling Method | **Solid Flight Augers** | Drilling Contractor | **V & W Drilling** | Total Depth of Drill Hole | **25.0 feet** |
| Drill Rig Type | **CME 75** | Diameter(s) of Hole, inches | **6"** | Approx. Surface Elevation, ft MSL | |
| Groundwater Depth [Elevation], feet | **4.5** | Sampling Method(s) | **California Modified** | Drill Hole Backfill | **Neat Cement** |
| Remarks | **w.o.h = weight of hammer** | | | Driving Method and Drop | **140 lb automatic hammer** |

| 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, slightly moist, silty fine to coarse sand (SM) | | | | | | |
| | | | Gray, very moist to wet, very loose, clayey silt (ML) | | | | | | |
| | 5 | | | | D109-1I | 1 | | | |
| | | | Brown, wet, very loose, sandy silt (ML) | | | | | | |
| | 10 | | | | D109-2I | w.o.h. | 32.2 | 87 | |
| | | | Brown, wet, very loose, silty fine sand (SM) | | | | | | |
| | 15 | | | | D109-3I | 4 | | | |
| | | | gray | | | | | | |
| | 20 | | | | D109-4I | 1 | | | |
| | 25 | | | | D109-5 | 2 | 26.1 | 96 | |
| | | | Boring terminated at 25 feet | | | | | | |

BORING LOG 7915.05P - KENNEDY HIGH SCHOOL ATHLETIC IMPROVEMENTS.GPJ WKA.GDT 8/14/13 9:29 AM

Wallace Kuhl & ASSOCIATES

**FIGURE 17**

# APPENDIX C

## Liquefaction Analysis Results

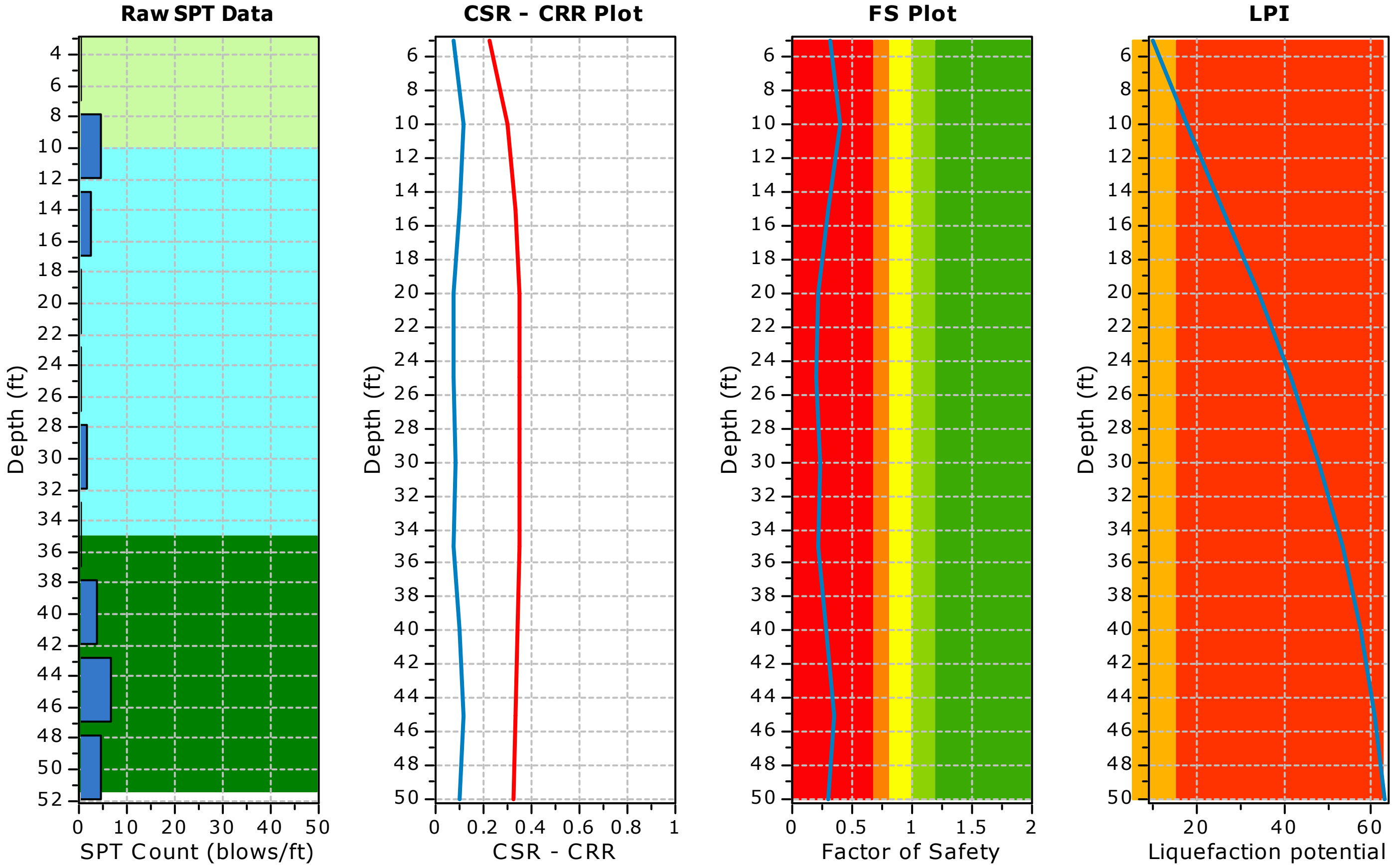
## SPT BASED LIQUEFACTION ANALYSIS REPORT

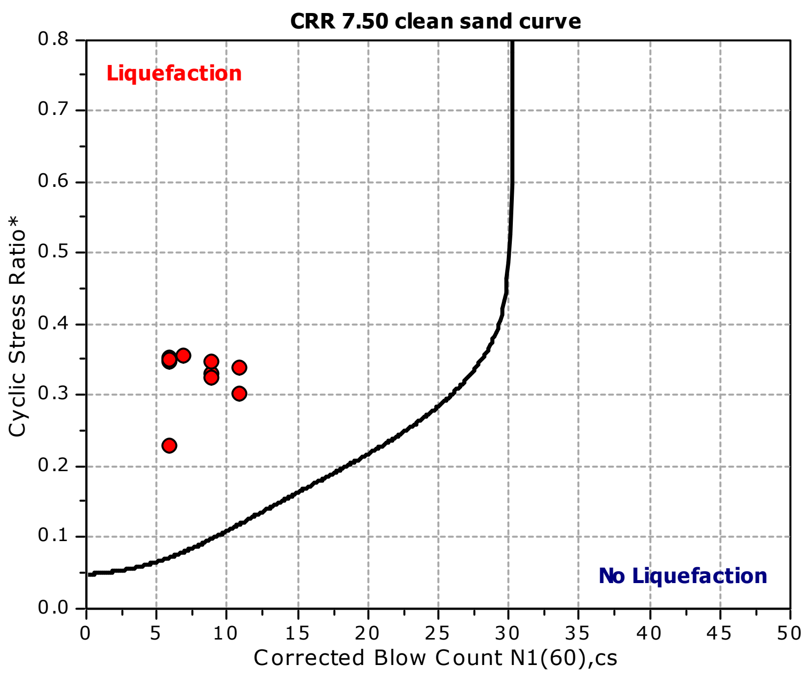
**Project title : JFK Athletic Improvements** **SPT Name: D106**

**Location : Sacramento, CA**

**:: Input parameters and analysis properties ::**

| | | | |
|---|---|---|---|
| Analysis method: | NCEER 1998 | G.W.T. (in-situ): | 4.00 ft |
| Fines correction method: | NCEER 1998 | G.W.T. (earthq.): | 4.00 ft |
| Sampling method: | Standard Sampler | Earthquake magnitude $M_w$: | 6.46 |
| Borehole diameter: | 65mm to 115mm | Peak ground acceleration: | 0.46 g |
| Rod length: | 3.30 ft | Eq. external load: | 0.00 tsf |
| Hammer energy ratio: | 0.80 | | |



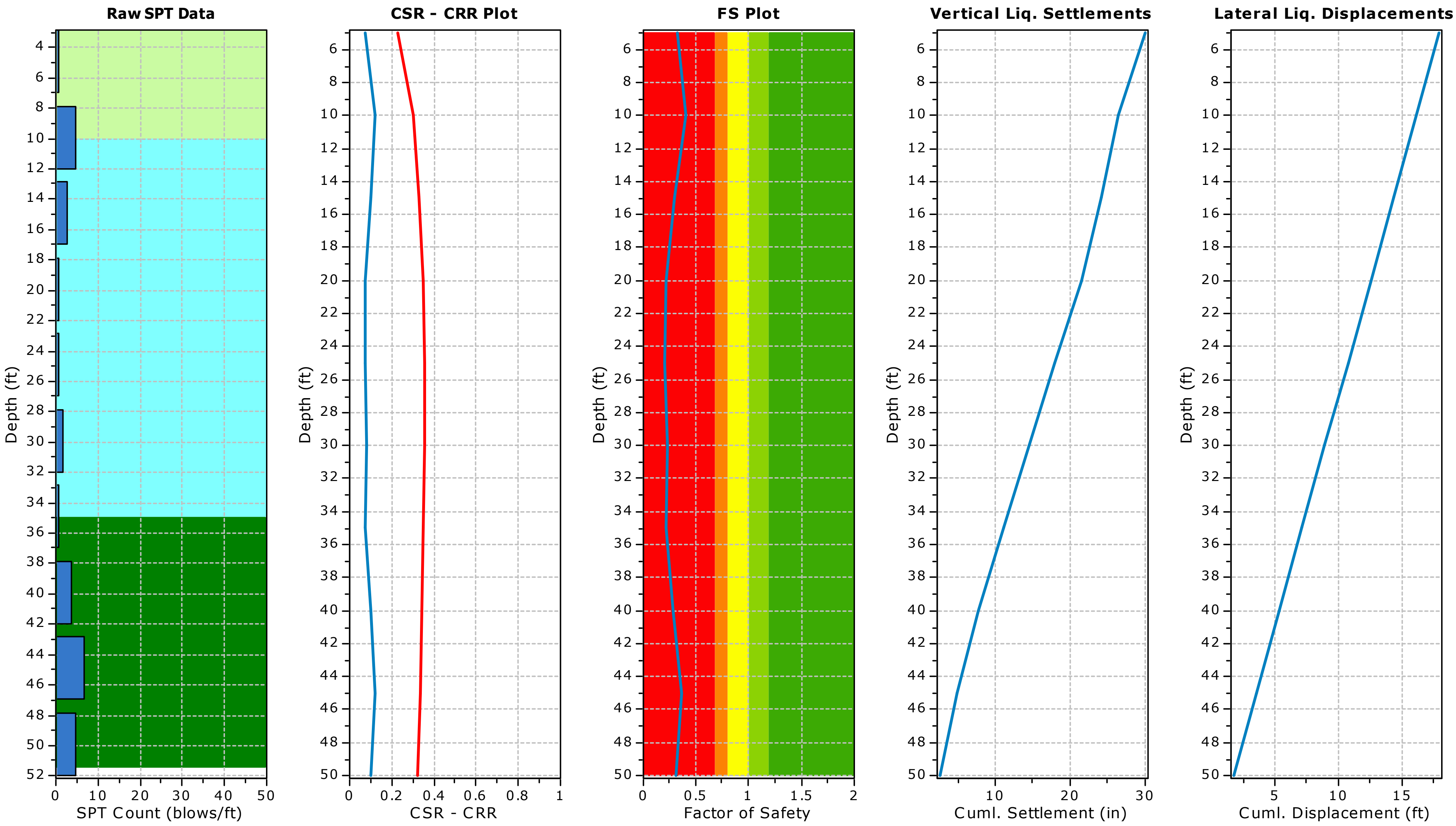

**F.S. color scheme**

- 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

**:: Overall Liquefaction Assessment Analysis Plots ::**



**:: Field input data ::**

| Test Depth (ft) | SPT Field Value (blows) | Fines Content (%) | Unit Weight (pcf) | Infl. Thickness (ft) | Can Liquefy |
|---|---|---|---|---|---|
| 5.00 | 1 | 52.00 | 107.00 | 5.00 | Yes |
| 10.00 | 5 | 52.00 | 117.00 | 5.00 | Yes |
| 15.00 | 3 | 55.30 | 117.00 | 5.00 | Yes |
| 20.00 | 1 | 51.00 | 117.00 | 5.00 | Yes |
| 25.00 | 1 | 54.70 | 117.00 | 5.00 | Yes |
| 30.00 | 2 | 52.00 | 117.00 | 5.00 | Yes |
| 35.00 | 1 | 52.20 | 117.00 | 5.00 | Yes |
| 40.00 | 4 | 52.00 | 117.00 | 5.00 | Yes |
| 45.00 | 7 | 47.90 | 117.00 | 5.00 | Yes |
| 50.00 | 5 | 52.00 | 117.00 | 5.00 | Yes |

**Abbreviations**

| | |
|---|---|
| Depth: | Depth at which test was performed (ft) |
| SPT Field Value: | Number of blows per foot |
| Fines Content: | Fines content at test depth (%) |
| Unit Weight: | Unit weight at test depth (pcf) |
| Infl. Thickness: | Thickness of the soil layer to be considered in settlements analysis (ft) |
| Can Liquefy: | User defined switch for excluding/including test depth from the analysis procedure |

**:: Cyclic Resistance Ratio (CRR) calculation data ::**

| Depth (ft) | SPT Field Value | Unit Weight (pcf) | $\sigma_v$ (tsf) | $u_o$ (tsf) | $\sigma'_{vo}$ (tsf) | $C_N$ | $C_E$ | $C_B$ | $C_R$ | $C_S$ | $(N_1)_{60}$ | Fines Content (%) | α | β | $(N_1)_{60cs}$ | $CRR_{7.5}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 1 | 107.00 | 0.27 | 0.03 | 0.24 | 1.55 | 0.80 | 1.00 | 0.75 | 1.00 | 1 | 52.00 | 5.00 | 1.20 | 6 | 0.073 |
| 10.00 | 5 | 117.00 | 0.56 | 0.19 | 0.37 | 1.42 | 0.80 | 1.00 | 0.85 | 1.00 | 5 | 52.00 | 5.00 | 1.20 | 11 | 0.120 |
| 15.00 | 3 | 117.00 | 0.85 | 0.34 | 0.51 | 1.31 | 0.80 | 1.00 | 0.85 | 1.00 | 3 | 55.30 | 5.00 | 1.20 | 9 | 0.099 |
| 20.00 | 1 | 117.00 | 1.15 | 0.50 | 0.65 | 1.22 | 0.80 | 1.00 | 0.95 | 1.00 | 1 | 51.00 | 5.00 | 1.20 | 6 | 0.073 |
| 25.00 | 1 | 117.00 | 1.44 | 0.66 | 0.78 | 1.13 | 0.80 | 1.00 | 0.95 | 1.00 | 1 | 54.70 | 5.00 | 1.20 | 6 | 0.073 |
| 30.00 | 2 | 117.00 | 1.73 | 0.81 | 0.92 | 1.06 | 0.80 | 1.00 | 1.00 | 1.00 | 2 | 52.00 | 5.00 | 1.20 | 7 | 0.081 |
| 35.00 | 1 | 117.00 | 2.02 | 0.97 | 1.06 | 1.00 | 0.80 | 1.00 | 1.00 | 1.00 | 1 | 52.20 | 5.00 | 1.20 | 6 | 0.073 |
| 40.00 | 4 | 117.00 | 2.31 | 1.12 | 1.19 | 0.95 | 0.80 | 1.00 | 1.00 | 1.00 | 3 | 52.00 | 5.00 | 1.20 | 9 | 0.099 |
| 45.00 | 7 | 117.00 | 2.61 | 1.28 | 1.33 | 0.90 | 0.80 | 1.00 | 1.00 | 1.00 | 5 | 47.90 | 5.00 | 1.20 | 11 | 0.120 |
| 50.00 | 5 | 117.00 | 2.90 | 1.44 | 1.46 | 0.85 | 0.80 | 1.00 | 1.00 | 1.00 | 3 | 52.00 | 5.00 | 1.20 | 9 | 0.099 |

**Abbreviations**

| | |
|---|---|
| $\sigma_v$: | Total stress during SPT test (tsf) |
| $u_o$: | Water pore pressure during SPT test (tsf) |
| $\sigma'_{vo}$: | Effective overburden pressure during SPT test (tsf) |
| $C_N$: | Overburden corretion factor |
| $C_E$: | Energy correction factor |
| $C_B$: | Borehole diameter correction factor |
| $C_R$: | Rod length correction factor |
| $C_S$: | Liner correction factor |
| $N_{1(60)}$: | Corrected $N_{SPT}$ to a 60% energy ratio |
| α, β: | Clean sand equivalent clean sand formula coefficients |
| $N_{1(60)cs}$: | Corected $N_{1(60)}$ value for fines content |
| $CRR_{7.5}$: | Cyclic resistance ratio for M=7.5 |

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

| Depth (ft) | Unit Weight (pcf) | $\sigma_{v,eq}$ (tsf) | $u_{o,eq}$ (tsf) | $\sigma'_{vo,eq}$ (tsf) | $r_d$ | α | CSR | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 107.00 | 0.27 | 0.03 | 0.24 | 0.99 | 1.00 | 0.335 | 1.46 | 0.229 | 1.00 | 0.229 | 0.318 |
| 10.00 | 117.00 | 0.56 | 0.19 | 0.37 | 0.98 | 1.00 | 0.440 | 1.46 | 0.300 | 1.00 | 0.300 | 0.400 |

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

| Depth (ft) | Unit Weight (pcf) | $\sigma_{v,eq}$ (tsf) | $u_{o,eq}$ (tsf) | $\sigma'_{vo,eq}$ (tsf) | $r_d$ | α | CSR | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 15.00 | 117.00 | 0.85 | 0.34 | 0.51 | 0.97 | 1.00 | 0.485 | 1.46 | 0.331 | 1.00 | 0.331 | 0.300 | ● |
| 20.00 | 117.00 | 1.15 | 0.50 | 0.65 | 0.96 | 1.00 | 0.507 | 1.46 | 0.346 | 1.00 | 0.346 | 0.210 | ● |
| 25.00 | 117.00 | 1.44 | 0.66 | 0.78 | 0.94 | 1.00 | 0.517 | 1.46 | 0.353 | 1.00 | 0.353 | 0.206 | ● |
| 30.00 | 117.00 | 1.73 | 0.81 | 0.92 | 0.92 | 1.00 | 0.518 | 1.46 | 0.354 | 1.00 | 0.354 | 0.228 | ● |
| 35.00 | 117.00 | 2.02 | 0.97 | 1.06 | 0.89 | 1.00 | 0.510 | 1.46 | 0.348 | 1.00 | 0.348 | 0.209 | ● |
| 40.00 | 117.00 | 2.31 | 1.12 | 1.19 | 0.85 | 1.00 | 0.494 | 1.46 | 0.337 | 0.98 | 0.346 | 0.287 | ● |
| 45.00 | 117.00 | 2.61 | 1.28 | 1.33 | 0.80 | 1.00 | 0.472 | 1.46 | 0.322 | 0.96 | 0.337 | 0.357 | ● |
| 50.00 | 117.00 | 2.90 | 1.44 | 1.46 | 0.75 | 1.00 | 0.446 | 1.46 | 0.304 | 0.94 | 0.325 | 0.306 | ● |

**Abbreviations**

| | |
|---|---|
| $\sigma_{v,eq}$: | Total overburden pressure at test point, during earthquake (tsf) |
| $u_{o,eq}$: | Water pressure at test point, during earthquake (tsf) |
| $\sigma'_{vo,eq}$: | Effective overburden pressure, during earthquake (tsf) |
| $r_d$ : | Nonlinear shear mass factor |
| α: | Improvement factor due to stone columns |
| CSR : | Cyclic Stress Ratio (adjusted for improvement) |
| MSF : | Magnitude Scaling Factor |
| $CSR_{eq,M=7.5}$: | CSR adjusted for M=7.5 |
| $K_{sigma}$: | Effective overburden stress factor |
| $CSR^*$: | CSR fully adjusted (user FS applied) *** |
| FS: | Calculated factor of safety against soil liquefaction |

*** User FS: 1.00

## :: Liquefaction potential according to Iwasaki ::

| Depth (ft) | FS | F | wz | Thickness (ft) | $I_L$ |
|---|---|---|---|---|---|
| 5.00 | 0.318 | 0.68 | 9.24 | 5.00 | 9.60 |
| 10.00 | 0.400 | 0.60 | 8.48 | 5.00 | 7.74 |
| 15.00 | 0.300 | 0.70 | 7.71 | 5.00 | 8.23 |
| 20.00 | 0.210 | 0.79 | 6.95 | 5.00 | 8.37 |
| 25.00 | 0.206 | 0.79 | 6.19 | 5.00 | 7.49 |
| 30.00 | 0.228 | 0.77 | 5.43 | 5.00 | 6.38 |
| 35.00 | 0.209 | 0.79 | 4.67 | 5.00 | 5.62 |
| 40.00 | 0.287 | 0.71 | 3.90 | 5.00 | 4.24 |
| 45.00 | 0.357 | 0.64 | 3.14 | 5.00 | 3.08 |
| 50.00 | 0.306 | 0.69 | 2.38 | 5.00 | 2.52 |

**Overall potential $I_L$ : 63.27**

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

## :: Vertical settlements estimation for saturated sands ::

| Depth (ft) | $D_{50}$ (in) | $q_c$/N | $e_v$ weight factor | $e_v$ (%) | Δh (ft) | s (in) |
|---|---|---|---|---|---|---|
| 5.00 | 0.00 | 5.00 | 1.00 | 5.80 | 5.00 | 3.480 |
| 10.00 | 0.00 | 5.00 | 1.00 | 3.82 | 5.00 | 2.289 |
| 15.00 | 0.00 | 5.00 | 1.00 | 4.50 | 5.00 | 2.698 |
| 20.00 | 0.00 | 5.00 | 1.00 | 5.80 | 5.00 | 3.480 |
| 25.00 | 0.00 | 5.00 | 1.00 | 5.80 | 5.00 | 3.480 |

**:: Vertical settlements estimation for saturated sands ::**

| Depth (ft) | $D_{50}$ (in) | $q_c$/N | $e_v$ weight factor | $e_v$ (%) | Δh (ft) | s (in) |
|---|---|---|---|---|---|---|
| 30.00 | 0.00 | 5.00 | 1.00 | 5.53 | 5.00 | 3.316 |
| 35.00 | 0.00 | 5.00 | 1.00 | 5.80 | 5.00 | 3.480 |
| 40.00 | 0.00 | 5.00 | 1.00 | 4.50 | 5.00 | 2.698 |
| 45.00 | 0.00 | 5.00 | 1.00 | 3.82 | 5.00 | 2.289 |
| 50.00 | 0.00 | 5.00 | 1.00 | 4.50 | 5.00 | 2.698 |

**Cumulative settlements: 29.909**

**Abbreviations**

$D_{50}$: Median grain size (in)
$q_c$/N: Ratio of cone resistance to SPT
$e_v$: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

**:: Lateral displacements estimation for saturated sands ::**

| Depth (ft) | $(N_1)_{60}$ | $D_r$ (%) | $\gamma_{max}$ (%) | $d_z$ (ft) | LDI | LD (ft) |
|---|---|---|---|---|---|---|
| 5.00 | 1 | 14.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 10.00 | 5 | 31.30 | 51.20 | 5.00 | 2.560 | 1.79 |
| 15.00 | 3 | 24.25 | 51.20 | 5.00 | 2.560 | 1.79 |
| 20.00 | 1 | 14.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 25.00 | 1 | 14.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 30.00 | 2 | 19.80 | 51.20 | 5.00 | 2.560 | 1.79 |
| 35.00 | 1 | 14.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 40.00 | 3 | 24.25 | 51.20 | 5.00 | 2.560 | 1.79 |
| 45.00 | 5 | 31.30 | 51.20 | 5.00 | 2.560 | 1.79 |
| 50.00 | 3 | 24.25 | 51.20 | 5.00 | 2.560 | 1.79 |

**Cumulative lateral displacements: 17.92**

**Abbreviations**

$D_r$: Relative density (%)
$\gamma_{max}$: Maximum amplitude of cyclic shear strain (%)
$d_z$: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

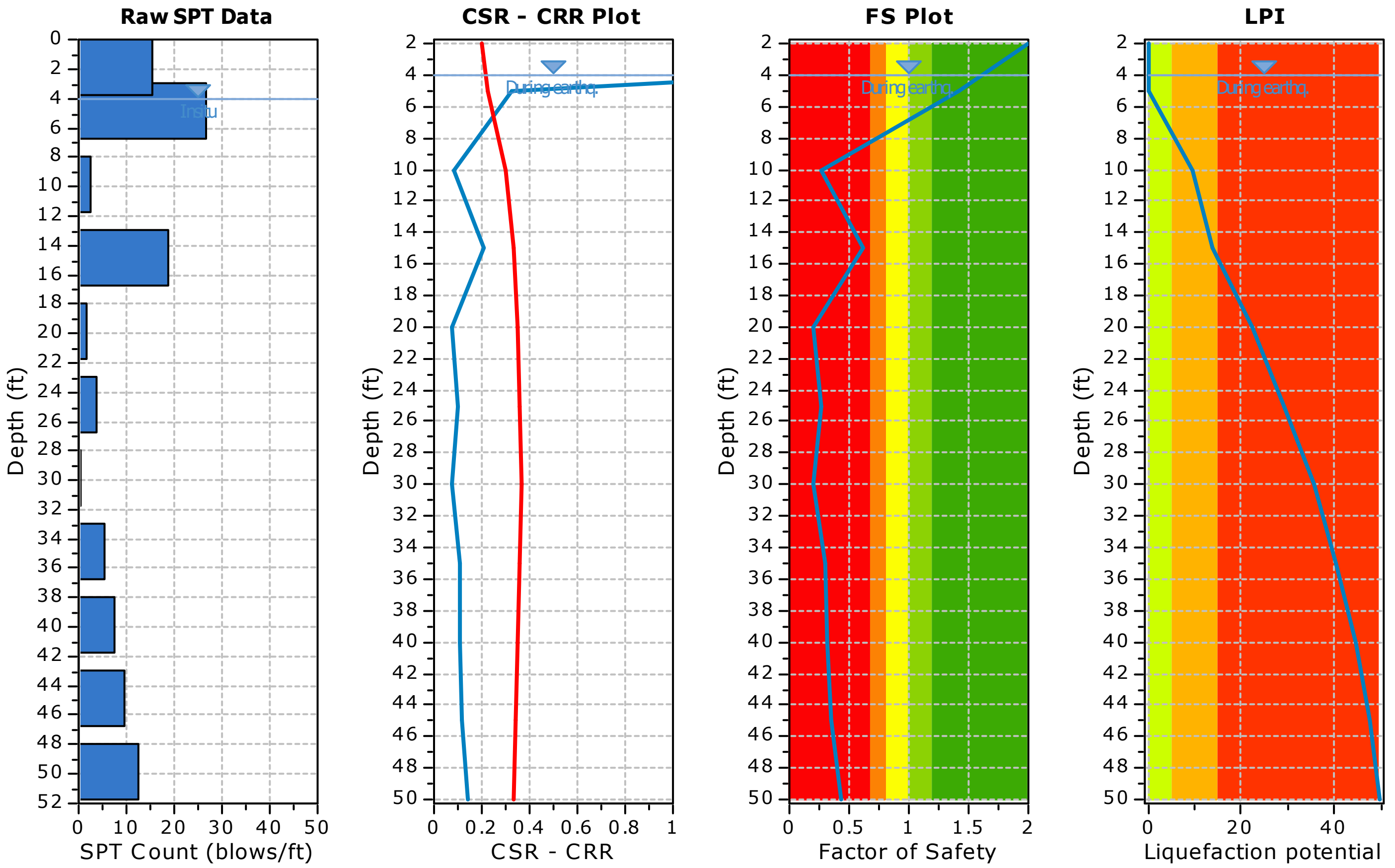
## SPT BASED LIQUEFACTION ANALYSIS REPORT

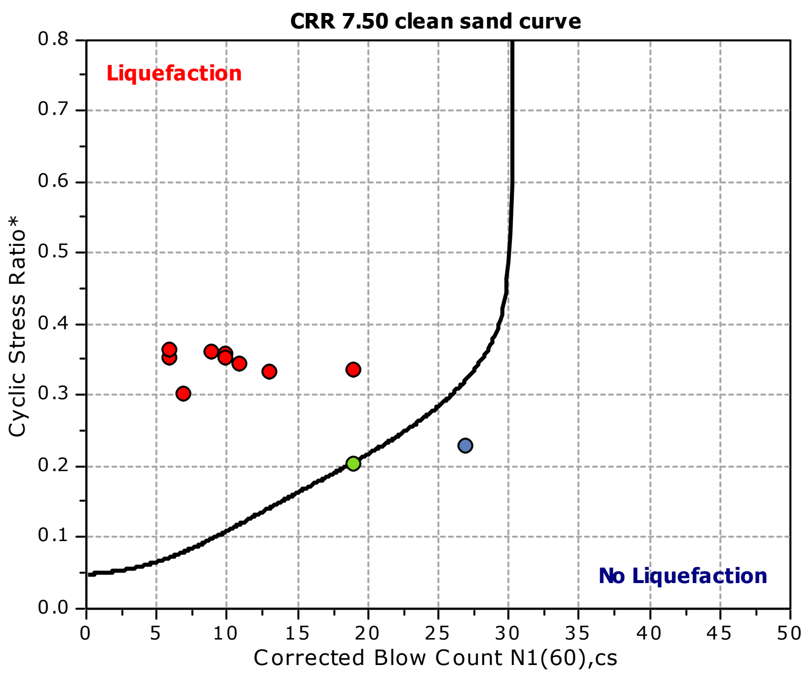
**Project title : JFK Athletic Improvements** **SPT Name: D1**

**Location : Sacramento, CA**

**:: Input parameters and analysis properties ::**

| | | | |
|---|---|---|---|
| Analysis method: | NCEER 1998 | G.W.T. (in-situ): | 4.00 ft |
| Fines correction method: | NCEER 1998 | G.W.T. (earthq.): | 4.00 ft |
| Sampling method: | Standard Sampler | Earthquake magnitude $M_w$: | 6.46 |
| Borehole diameter: | 200mm | Peak ground acceleration: | 0.46 g |
| Rod length: | 3.30 ft | Eq. external load: | 0.00 tsf |
| Hammer energy ratio: | 0.50 | | |



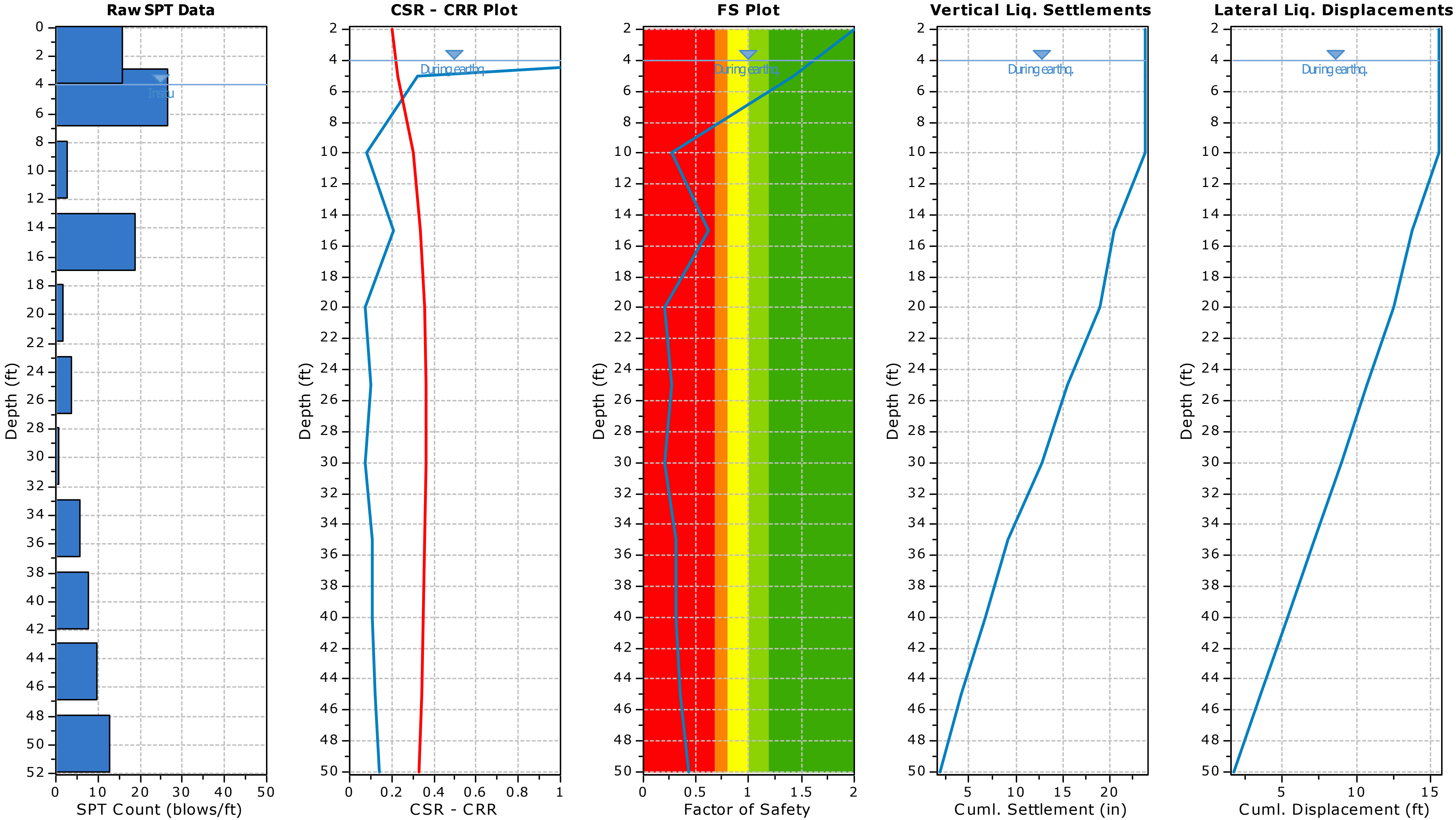

**F.S. color scheme**

- 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

## :: Overall Liquefaction Assessment Analysis Plots ::



**:: Field input data ::**

| Test Depth (ft) | SPT Field Value (blows) | Fines Content (%) | Unit Weight (pcf) | Infl. Thickness (ft) | Can Liquefy |
|---|---|---|---|---|---|
| 2.00 | 16 | 50.00 | 112.00 | 3.50 | Yes |
| 5.00 | 27 | 50.00 | 112.00 | 4.00 | Yes |
| 10.00 | 3 | 50.00 | 112.00 | 5.00 | Yes |
| 15.00 | 19 | 47.60 | 112.00 | 5.00 | Yes |
| 20.00 | 2 | 42.70 | 112.00 | 5.00 | Yes |
| 25.00 | 4 | 50.00 | 112.00 | 5.00 | Yes |
| 30.00 | 1 | 50.00 | 112.00 | 5.00 | Yes |
| 35.00 | 6 | 59.40 | 112.00 | 5.00 | Yes |
| 40.00 | 8 | 50.00 | 112.00 | 5.00 | Yes |
| 45.00 | 10 | 50.00 | 112.00 | 5.00 | Yes |
| 50.00 | 13 | 50.00 | 112.00 | 5.00 | Yes |

**Abbreviations**

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

**:: Cyclic Resistance Ratio (CRR) calculation data ::**

| Depth (ft) | SPT Field Value | Unit Weight (pcf) | $\sigma_v$ (tsf) | $u_o$ (tsf) | $\sigma'_{vo}$ (tsf) | $C_N$ | $C_E$ | $C_B$ | $C_R$ | $C_S$ | $(N_1)_{60}$ | Fines Content (%) | α | β | $(N_1)_{60cs}$ | $CRR_{7.5}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2.00 | 16 | 112.00 | 0.11 | 0.00 | 0.11 | 1.68 | 0.50 | 1.15 | 0.75 | 1.00 | 12 | 50.00 | 5.00 | 1.20 | 19 | 4.000 |
| 5.00 | 27 | 112.00 | 0.28 | 0.03 | 0.25 | 1.53 | 0.50 | 1.15 | 0.75 | 1.00 | 18 | 50.00 | 5.00 | 1.20 | 27 | 0.323 |
| 10.00 | 3 | 112.00 | 0.56 | 0.19 | 0.37 | 1.42 | 0.50 | 1.15 | 0.85 | 1.00 | 2 | 50.00 | 5.00 | 1.20 | 7 | 0.081 |
| 15.00 | 19 | 112.00 | 0.84 | 0.34 | 0.50 | 1.32 | 0.50 | 1.15 | 0.85 | 1.00 | 12 | 47.60 | 5.00 | 1.20 | 19 | 0.206 |
| 20.00 | 2 | 112.00 | 1.12 | 0.50 | 0.62 | 1.23 | 0.50 | 1.15 | 0.95 | 1.00 | 1 | 42.70 | 5.00 | 1.20 | 6 | 0.073 |
| 25.00 | 4 | 112.00 | 1.40 | 0.66 | 0.74 | 1.16 | 0.50 | 1.15 | 0.95 | 1.00 | 3 | 50.00 | 5.00 | 1.20 | 9 | 0.099 |
| 30.00 | 1 | 112.00 | 1.68 | 0.81 | 0.87 | 1.09 | 0.50 | 1.15 | 1.00 | 1.00 | 1 | 50.00 | 5.00 | 1.20 | 6 | 0.073 |
| 35.00 | 6 | 112.00 | 1.96 | 0.97 | 0.99 | 1.03 | 0.50 | 1.15 | 1.00 | 1.00 | 4 | 59.40 | 5.00 | 1.20 | 10 | 0.110 |
| 40.00 | 8 | 112.00 | 2.24 | 1.12 | 1.12 | 0.98 | 0.50 | 1.15 | 1.00 | 1.00 | 4 | 50.00 | 5.00 | 1.20 | 10 | 0.110 |
| 45.00 | 10 | 112.00 | 2.52 | 1.28 | 1.24 | 0.93 | 0.50 | 1.15 | 1.00 | 1.00 | 5 | 50.00 | 5.00 | 1.20 | 11 | 0.120 |
| 50.00 | 13 | 112.00 | 2.80 | 1.44 | 1.36 | 0.88 | 0.50 | 1.15 | 1.00 | 1.00 | 7 | 50.00 | 5.00 | 1.20 | 13 | 0.142 |

**Abbreviations**

$\sigma_v$: Total stress during SPT test (tsf)
$u_o$: Water pore pressure during SPT test (tsf)
$\sigma'_{vo}$: Effective overburden pressure during SPT test (tsf)
$C_N$: Overburden corretion factor
$C_E$: Energy correction factor
$C_B$: Borehole diameter correction factor
$C_R$: Rod length correction factor
$C_S$: Liner correction factor
$N_{1(60)}$: Corrected $N_{SPT}$ to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

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

| Depth (ft) | Unit Weight (pcf) | $\sigma_{v,eq}$ (tsf) | $u_{o,eq}$ (tsf) | $\sigma'_{vo,eq}$ (tsf) | $r_d$ | α | CSR | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|

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

| Depth (ft) | Unit Weight (pcf) | $\sigma_{v,eq}$ (tsf) | $u_{o,eq}$ (tsf) | $\sigma'_{vo,eq}$ (tsf) | $r_d$ | α | CSR | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | CSR* | FS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2.00 | 112.00 | 0.11 | 0.00 | 0.11 | 1.00 | 1.00 | 0.298 | 1.46 | 0.204 | 1.00 | 0.204 | 2.000 |
| 5.00 | 112.00 | 0.28 | 0.03 | 0.25 | 0.99 | 1.00 | 0.333 | 1.46 | 0.227 | 1.00 | 0.227 | 1.421 |
| 10.00 | 112.00 | 0.56 | 0.19 | 0.37 | 0.98 | 1.00 | 0.440 | 1.46 | 0.300 | 1.00 | 0.300 | 0.269 |
| 15.00 | 112.00 | 0.84 | 0.34 | 0.50 | 0.97 | 1.00 | 0.490 | 1.46 | 0.334 | 1.00 | 0.334 | 0.618 |
| 20.00 | 112.00 | 1.12 | 0.50 | 0.62 | 0.96 | 1.00 | 0.516 | 1.46 | 0.352 | 1.00 | 0.352 | 0.207 |
| 25.00 | 112.00 | 1.40 | 0.66 | 0.74 | 0.94 | 1.00 | 0.529 | 1.46 | 0.361 | 1.00 | 0.361 | 0.275 |
| 30.00 | 112.00 | 1.68 | 0.81 | 0.87 | 0.92 | 1.00 | 0.532 | 1.46 | 0.363 | 1.00 | 0.363 | 0.200 |
| 35.00 | 112.00 | 1.96 | 0.97 | 0.99 | 0.89 | 1.00 | 0.526 | 1.46 | 0.359 | 1.00 | 0.359 | 0.305 |
| 40.00 | 112.00 | 2.24 | 1.12 | 1.12 | 0.85 | 1.00 | 0.510 | 1.46 | 0.348 | 0.99 | 0.352 | 0.311 |
| 45.00 | 112.00 | 2.52 | 1.28 | 1.24 | 0.80 | 1.00 | 0.488 | 1.46 | 0.333 | 0.97 | 0.344 | 0.350 |
| 50.00 | 112.00 | 2.80 | 1.44 | 1.36 | 0.75 | 1.00 | 0.462 | 1.46 | 0.315 | 0.95 | 0.332 | 0.428 |

**Abbreviations**

| | |
|---|---|
| $\sigma_{v,eq}$: | Total overburden pressure at test point, during earthquake (tsf) |
| $u_{o,eq}$: | Water pressure at test point, during earthquake (tsf) |
| $\sigma'_{vo,eq}$: | Effective overburden pressure, during earthquake (tsf) |
| $r_d$ : | Nonlinear shear mass factor |
| α: | Improvement factor due to stone columns |
| CSR : | Cyclic Stress Ratio (adjusted for improvement) |
| MSF : | Magnitude Scaling Factor |
| $CSR_{eq,M=7.5}$: | CSR adjusted for M=7.5 |
| $K_{sigma}$: | Effective overburden stress factor |
| CSR*: | CSR fully adjusted (user FS applied) *** |
| FS: | Calculated factor of safety against soil liquefaction |

*** User FS: 1.00

## :: Liquefaction potential according to Iwasaki ::

| Depth (ft) | FS | F | wz | Thickness (ft) | $I_L$ |
|---|---|---|---|---|---|
| 2.00 | 2.000 | 0.00 | 9.70 | 3.00 | 0.00 |
| 5.00 | 1.421 | 0.00 | 9.24 | 3.00 | 0.00 |
| 10.00 | 0.269 | 0.73 | 8.48 | 5.00 | 9.44 |
| 15.00 | 0.618 | 0.38 | 7.71 | 5.00 | 4.50 |
| 20.00 | 0.207 | 0.79 | 6.95 | 5.00 | 8.41 |
| 25.00 | 0.275 | 0.73 | 6.19 | 5.00 | 6.84 |
| 30.00 | 0.200 | 0.80 | 5.43 | 5.00 | 6.61 |
| 35.00 | 0.305 | 0.69 | 4.67 | 5.00 | 4.94 |
| 40.00 | 0.311 | 0.69 | 3.90 | 5.00 | 4.10 |
| 45.00 | 0.350 | 0.65 | 3.14 | 5.00 | 3.11 |
| 50.00 | 0.428 | 0.57 | 2.38 | 5.00 | 2.08 |

**Overall potential $I_L$ : 50.03**

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

## :: Vertical settlements estimation for dry sands ::

| Depth (ft) | $(N_1)_{60}$ | $\tau_{av}$ | p | $G_{max}$ (tsf) | α | b | γ (%) | $\varepsilon_{15}$ | $N_c$ | $\varepsilon_{Nc}$ (%) | Δh (ft) | ΔS (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2.00 | 12 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 3.50 | 0.000 |

**:: Vertical settlements estimation for dry sands ::**

| Depth (ft) | $(N_1)_{60}$ | $\tau_{av}$ | p | $G_{max}$ (tsf) | α | b | γ (%) | $\varepsilon_{15}$ | $N_c$ | $\varepsilon_{Nc}$ (%) | Δh (ft) | ΔS (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|

**Cumulative settlemetns: 0.000**

**Abbreviations**

$\tau_{av}$: Average cyclic shear stress
p: Average stress
$G_{max}$: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain (%)
$\varepsilon_{15}$: Volumetric strain after 15 cycles
$N_c$: Number of cycles
$\varepsilon_{Nc}$: Volumetric strain for number of cycles $N_c$ (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

**:: Vertical settlements estimation for saturated sands ::**

| Depth (ft) | $D_{50}$ (in) | $q_c/N$ | $e_v$ weight factor | $e_v$ (%) | Δh (ft) | s (in) |
|---|---|---|---|---|---|---|
| 5.00 | 0.00 | 5.00 | 1.00 | 0.00 | 4.00 | 0.000 |
| 10.00 | 0.00 | 5.00 | 1.00 | 5.53 | 5.00 | 3.316 |
| 15.00 | 0.00 | 5.00 | 1.00 | 2.44 | 5.00 | 1.462 |
| 20.00 | 0.00 | 5.00 | 1.00 | 5.80 | 5.00 | 3.480 |
| 25.00 | 0.00 | 5.00 | 1.00 | 4.50 | 5.00 | 2.698 |
| 30.00 | 0.00 | 5.00 | 1.00 | 5.80 | 5.00 | 3.480 |
| 35.00 | 0.00 | 5.00 | 1.00 | 4.13 | 5.00 | 2.475 |
| 40.00 | 0.00 | 5.00 | 1.00 | 4.13 | 5.00 | 2.475 |
| 45.00 | 0.00 | 5.00 | 1.00 | 3.82 | 5.00 | 2.289 |
| 50.00 | 0.00 | 5.00 | 1.00 | 3.33 | 5.00 | 1.996 |

**Cumulative settlements: 23.672**

**Abbreviations**

$D_{50}$: Median grain size (in)
$q_c/N$: Ratio of cone resistance to SPT
$e_v$: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

**:: Lateral displacements estimation for saturated sands ::**

| Depth (ft) | $(N_1)_{60}$ | $D_r$ (%) | $\gamma_{max}$ (%) | $d_z$ (ft) | LDI | LD (ft) |
|---|---|---|---|---|---|---|
| 2.00 | 12 | 48.50 | 0.00 | 3.50 | 0.000 | 0.00 |
| 5.00 | 18 | 59.40 | 0.76 | 4.00 | 0.030 | 0.02 |
| 10.00 | 2 | 19.80 | 51.20 | 5.00 | 2.560 | 1.79 |
| 15.00 | 12 | 48.50 | 34.10 | 5.00 | 1.705 | 1.19 |
| 20.00 | 1 | 14.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 25.00 | 3 | 24.25 | 51.20 | 5.00 | 2.560 | 1.79 |
| 30.00 | 1 | 14.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 35.00 | 4 | 28.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 40.00 | 4 | 28.00 | 51.20 | 5.00 | 2.560 | 1.79 |
| 45.00 | 5 | 31.30 | 51.20 | 5.00 | 2.560 | 1.79 |
| 50.00 | 7 | 37.04 | 51.20 | 5.00 | 2.560 | 1.79 |

**:: Lateral displacements estimation for saturated sands ::**

| Depth (ft) | $(N_1)_{60}$ | $D_r$ (%) | $\gamma_{max}$ (%) | $d_z$ (ft) | LDI | LD (ft) |
|---|---|---|---|---|---|---|

**Cumulative lateral displacements: 15.55**

**Abbreviations**

$D_r$: Relative density (%)
$\gamma_{max}$: Maximum amplitude of cyclic shear strain (%)
$d_z$: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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