Appendix A

# Preliminary Geotechnical Investigation

July 19, 2022
J.N.: 3027.00

Mr. Mitchell Gardner
G3 Urban
15235 S. Western Avenue
Gardena, California 90249

**Subject:** **Update Letter, Proposed Industrial Development, 11709 Artesia Boulevard, Artesia, California.**

**Reference:** *Preliminary Geotechnical Investigation, Proposed Industrial Development, 11709 Artesia Blvd., Artesia, California*, prepared by Albus & Associates, Inc., dated December 7, 2021.

*Preliminary Geotechnical Investigation for Proposed Water Quality Improvements, Proposed Industrial Development, 11709 Artesia Blvd., Artesia, California*, prepared by Albus & Associates, Inc., dated May 5, 2022.

Dear Mr. Gardner,

Pursuant to your request, ***Albus & Associates, Inc***, is providing an update letter to our above-referenced reports. We understand the previous structures have been demolished and the site is now vacant.

This proposed development is generally the same as the proposed development discussed in our above-referenced reports. The conclusions and recommendations in the referenced reports remain valid and applicable to the current proposed project.

Per our referenced report dated May 5, 2022, infiltration of storm water is not feasible and the demolition of the previous structures has not altered that conclusion.

Additionally, the demolition of the previous structures has likely disturbed the upper 1 to 2 feet of the existing subsurface soils. Per our referenced report dated December 7, 2021, we have recommended the existing ground be prepared by removing and recompacting the existing artificial fill soils which extend to depths of 2 feet or more. Therefore, soils disturbed by demolition are anticipated to be included by this recommendation and no modifications to our recommendations are required. The exposed subgrade from demolition operations should conform the Site Clearing section (Section 6.1.3) of our referenced report dated December 7, 2021.

We appreciate this opportunity to be of service to you. If you should have any questions regarding the contents of this correspondence, please do not hesitate to call our office.

Sincerely,

***ALBUS & ASSOCIATES, INC.***

REGISTERED PROFESSIONAL ENGINEER
HYUN JIN KIM
No. 3106
GEOTECHNICAL
STATE OF CALIFORNIA

Paul Hyun Jin Kim
Associate Engineer
GE 3106

December 7, 2021
J.N.: 3027.00

Mr. Mitchell Gardner
G3 Urban
15235 S. Western Avenue
Gardena, California 90249

**Subject:** **Preliminary Geotechnical Investigation, Proposed Industrial Development, 11709 Artesia Blvd., Artesia, California**

Dear Mr. Gardner,

Pursuant to your request, ***Albus & Associates, Inc***. is pleased to present to you our preliminary geotechnical investigation report for the proposed development at the subject site. This report presents the results of our aerial photo and literature review, subsurface exploration, laboratory testing, and engineering analyses. Conclusions relevant to the feasibility of the proposed site development and recommendations for site development are also presented herein based on the findings of our work.

We appreciate this opportunity to be of service to you. If you should have any questions regarding the contents of this report, please do not hesitate to call.

Sincerely,

***ALBUS & ASSOCIATES, INC.***

David E. Albus
Principal Engineer

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**FIGURES AND PLATES**



**APPENDICES**

# 1.0 INTRODUCTION

## 1.1 PURPOSE AND SCOPE

The purposes of our preliminary geotechnical investigation were to evaluate geotechnical conditions within the project area and to provide conclusions and recommendations relevant to the design and construction of the proposed improvements at the subject site. The scope of this investigation included the following:

- Review of the historical aerial photographs;
- Review of published geologic and seismic data for the site and surrounding area;
- Exploratory drilling and soil sampling;
- Laboratory testing of selected soil samples;
- Engineering analyses of data obtained from our review, exploration, and laboratory testing;
- Evaluation of site seismicity, liquefaction potential, and settlement potential; and,
- Preparation of this report

## 1.2 SITE LOCATION AND DESCRIPTION

The site is located at the address of 11709 Artesia Boulevard within the city of Artesia, California. The site is bordered by Artesia Boulevard to the south, Fallon Avenue to the west, Alburtis Avenue to the east, and a commercial building and surface parking lots to the north. The location of the site and its relationship to the surrounding areas are shown in Figure 1, Site Location Map.

The site consists of approximately 3.33 acres of land with several industrial buildings, above-ground tanks, processing equipment, and above-ground piping. The remainder of the site consists of asphalt- and concrete-covered paving.

## 1.3 PROPOSED DEVELOPMENT

Based on the project brochure prepared by Colliers, we understand that the proposed development will likely consist of one or two new industrial buildings at-grade parking as well as associated interior driveways, parking, underground utilities, and decorative hardscape and landscape areas.

No grading or structural plans were available in preparing this report. However, we anticipate that minor rough grading of the site will be required to achieve future surface configuration. Structural loads are anticipated to typically consist of 250 kips for columns and 6 kips/ft for walls.


**SITE LOCATION MAP**

**G3 Urban**
**11709 Artesia Boulevard,**
**Artesia, California**

**NOT TO SCALE**

**FIGURE 1**

## 2.0 INVESTIGATION

### 2.1 RESEARCH

We have reviewed the referenced geologic publications and maps (see references). Data from these sources were utilized to develop some of the findings and conclusions presented herein.

We have also reviewed available historical aerial photographs. The aerial photos indicate that as early as 1967, the subject site was developed with industrial facilities. The site has remained relatively unchanged since 1967.

## 2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on November 2, 2021 and consisted of drilling three (3) soil borings to depths ranging from approximately 11.5 to 51.5 feet below the existing ground surface (bgs) and advancing four (4) Cone Penetration Tests (CPTs) to a depth of 50 feet bgs. The borings were drilled using a truck-mounted, continuous flight, hollow-stem-auger drill rig. A representative of Albus & Associates, Inc. logged the exploratory borings. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented in the Exploration Logs in Appendix A. The approximate locations of the exploratory excavations and CPT soundings completed by this firm are shown on the enclosed Geotechnical Map, Plate 1.

Bulk, relatively undisturbed, and Standard Penetration Test (SPT) samples were obtained at selected depths within the exploratory borings for subsequent laboratory testing. Relatively undisturbed samples were obtained using a 3-inch O.D., 2.5-inch I.D., California split-spoon soil sampler lined with brass rings. SPT samples were obtained from the boring using a standard, unlined SPT soil sampler. During each sampling interval, the sampler was driven 18 inches with successive drops of a 140-pound automatic hammer falling 30 inches. The number of blows required to advance the sampler was recorded for each six inches of advancement. The total blow count for the lower 12 inches of advancement per soil sample is recorded on the exploration log. Samples were placed in sealed containers or plastic bags and transported to our laboratory for analyses. The borings were backfilled with auger cuttings upon completion of sampling.

## 2.3 LABORATORY TESTING

Selected samples obtained from our subsurface exploration were tested in our soil laboratory. Tests consisted of maximum dry density and optimum moisture content, in-situ moisture content and dry density, expansion index, soluble sulfate content, direct shear, consolidation/collapse potential, grain-size distribution analysis, percent passing No. 200 sieve, direct shear strength, corrosivity testing (pH, chloride, and resistivity), and Atterberg limits. A description of laboratory test criteria and test results are presented in Appendix B.

# 3.0 SUBSURFACE CONDITIONS

## 3.1 SOIL CONDITIONS

Descriptions of the earth materials encountered during our investigation are summarized below and are presented in detail on the Exploration Logs presented in Appendix A.

Soil materials encountered at the subject site consisted of approximately 2 feet of artificial fill over alluvial soils. The artificial fill is predominately comprised of grayish brown sandy silt and silty sand. These fill materials typically were observed to be damp to moist and medium dense.

Underlying the artificial fills are native soils consisting of young alluvial fan deposits ($Qyf_a$). The alluvial fan deposit materials were encountered to the maximum depth explored of 51.5 feet and are comprised of grayish brown to light gray, interlayered silty sand and sand that are damp to wet and loose to very dense. Occasional lenses and layers of sandy silt are also present that are generally very moist to wet and firm to very stiff.

## 3.2 GROUNDWATER

Groundwater was encountered during this firm's subsurface exploration at a depth of 14 feet. The CDMG Special Report 019 suggests that historic high groundwater for the subject site is below 10 feet.

## 3.3 FAULTING

Based on our review of the referenced publications and seismic data, no active faults are known to project through or immediately adjacent to the subject sites and the site does not lie within an "Earthquake Fault Zone" as defined by the State of California in Earthquake Fault Zoning Act. Table 3.1 summarizes the known seismically active faults within 10 miles of the sites based on the 2008 USGS National Seismic Hazard Maps.

**TABLE 3.1**
**Summary of Faults**

| Name | Dist. (miles) | Slip Rate (mm/yr.) | Preferred Dip (degrees) | Slip Sense | Rupture Top (km) | Fault Length (km) |
|---|---|---|---|---|---|---|
| Puente Hills (Coyote Hills) | 2.67 | 0.7 | 26 | thrust | 2.8 | 17 |
| Puente Hills (Santa Fe Springs) | 2.82 | 0.7 | 29 | thrust | 2.8 | 11 |
| Newport Inglewood Connected alt 2 | 6.87 | 1.3 | 90 | strike slip | 0 | 208 |
| Newport-Inglewood, alt 1 | 6.95 | 1 | 88 | strike slip | 0 | 65 |
| Newport Inglewood Connected alt 1 | 6.95 | 1.3 | 89 | strike slip | 0 | 208 |
| Puente Hills (LA) | 7.01 | 0.7 | 27 | thrust | 2.1 | 22 |
| Elsinore;W+GI+T+J+CM | 8.40 | n/a | 84 | strike slip | 0 | 241 |
| Elsinore;W+GI | 8.40 | n/a | 81 | strike slip | 0 | 83 |
| Elsinore;W+GI+T | 8.40 | n/a | 84 | strike slip | 0 | 124 |
| Elsinore;W+GI+T+J | 8.40 | n/a | 84 | strike slip | 0 | 199 |
| Elsinore;W | 8.40 | 2.5 | 75 | strike slip | 0 | 46 |

# 4.0 ANALYSES

## 4.1 SEISMICITY

2019 CBC requires seismic parameters in accordance with ASCE 7-16. Unless noted otherwise, all section numbers cited in the following refer to the sections in ASCE 7-16.

Per Section 20.3 the project site was designated as Site Class D. We used the OSHPD seismic hazard tool to obtain the basic mapped acceleration parameters, including short periods ($S_S$) and 1-second period ($S_1$) $MCE_R$ Spectral Response Accelerations. Section 11.4.8 requires site-specific ground hazard analysis for structures on Site Class E with $S_S$ greater than or equal to 1.0 or Site Class D or E with $S_1$ greater than or equal to 0.2. Based on the mapped values of $S_S$ and $S_1$ the project site falls within this category, requiring site-specific hazard analysis in accordance with Section 21.2.

However, "A ground motion hazard analysis is not required for structures where: Structures on Site Class D sites with $S_1$ greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$." Assuming this exception is met for this project, a ground motion hazard analysis is not required and mapped seismic values can be used. Should this exception not be met, a ground motion hazard analysis is required to determine the Design response spectra for the proposed structures at this site. Both mapped and site-specific seismic design parameters are provided in this report, as presented in Section 6.2. Details of a ground motion hazard analysis are explained below.

According to Section 21.2.3 (Supplement 1), the site-specific Risk Targeted Maximum Considered Earthquake (MCER) spectral response acceleration at any period is the lesser of the probabilistic and the deterministic response accelerations, subject to the exception specified in the same section. The probabilistic response spectrum was developed using the computer program OpenSHA (Field et al., 2013), which implements Method 1 as described in Section 21.2.1.1. Fault Models 3.1 and 3.2 from the Third Uniform California Earthquake Rupture Forecast (UCERF3) were used as the earthquake rupture forecast models for the PSHA. In addition to known fault sources, background seismicity was also included in the PSHA. The ground motion Prediction Equations (GMPEs) selected for use in this analysis are those developed for the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) West 2 project. Four GMPEs - Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014) were used to perform the analysis.

In accordance with Section 21.2.2 (Supplement 1), the deterministic spectral response acceleration at each period was calculated as the 84th percentile, 5% damped response acceleration, using NGA-West2 GMPE Worksheet. For this, the information from at least three causative faults with the greatest contribution per deaggregation analysis were used and the larger acceleration spectrum among these was selected as the deterministic response spectrum. The deterministic spectrum was adjusted per requirements in Section 21.2.2 (Supplement 1) where applicable. Both probabilistic and deterministic spectra were subjected to the maximum direction scale factors specified in Section 21.2 to produce the maximum acceleration spectra.

Design response spectrum was developed by subjecting the site-specific $MCE_R$ response spectrum to the provisions outlined in Section 21.3. This process included comparison with 80% code-based design spectrum determined in accordance with Section 11.4.6. The short period and long period site coefficients (Fa and Fv, respectively) were determined per Section 21.3 in conjunction with Table 11.4-1. Site-specific design acceleration parameters ($S_{MS}$, $S_{M1}$, $S_{DS}$, and $S_{D1}$) were calculated according to Section 21.4.

Per Section 11.2 (definitions on Page 79 of ASCE7-16) for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues, Maximum Considered Earthquake Geometric Mean ($MCE_G$) peak ground acceleration $PGA_M$ shall be used. The site-specific $PGA_M$ is calculated per Section 21.5.3, as the lesser of the probabilistic $PGA_M$ (Section 21.5.1) and deterministic $PGA_M$ (Section 21.5.2), but no less than 80% site modified peak ground acceleration, $PGA_M$, obtained from OSHPD seismic hazard tool. From our analyses, we obtain a $PGA_M$ of 0.662g.

## 4.2 STATIC SETTLEMENT

Analyses were performed to estimate the maximum static settlement due to the anticipated maximum foundation loads. The analyses were based on the results of subsurface exploration and laboratory testing conducted at the site. Settlements were calculated based on the elastic method using the sampler blow counts to estimate the elastic property of the soils. Stresses induced by the footings were based on a Boussinesq distribution.

Settlement of the proposed buildings will depend on the magnitude of the structural loads. Assuming a column load of 250 kips, bearing pressure of 4,000 psf, and footing depth of 2 foot, total static settlement of a column footing is estimated to be 0.9 inches. A continuous footing that supports a load of 6 kips/ft, bearing pressure of 3,000 psf, 2 feet wide, and 2 feet deep is estimated to have a total static settlement of 0.5 inches.

## 4.3 LIQUEFACTION

Engineering research of soil liquefaction potential (Youd et al., 2001) indicates that generally three basic factors must exist concurrently in order for liquefaction to occur. These factors include:

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.
- A relatively loose silty and/or sandy soil.
- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

The site is located within a State-designated zone of potentially liquefiable soils. As a result, we conservatively have evaluated the potential for liquefaction.

The liquefaction susceptibility of the onsite soils was evaluated by analyzing the potential concurrent occurrence of the above-mentioned three basic factors. The liquefaction evaluation for the site was completed under the guidance of Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 2008).

Our liquefaction analyses were based on the field test data results from our CPT data. The liquefaction analyses were performed utilizing the CLiq software by GeoLogismiki. Among the methods available in this program for analysis of liquefaction potential, Robertson (NCEER 2001, 2009) was used for the current project. The seismic event was defined by peak ground acceleration PGA of 0.73 and mean moment magnitude of 6.83. The historic-high groundwater level, which is used as the design level for evaluation of liquefaction potential, is taken at a depth of 10 feet below ground surface, as discussed in Section 3.2.

Based on our analyses, several sublayers below the assumed shallowest groundwater level of 10 feet have a factor of safety less than 1.3 and are therefore considered prone to liquefaction during the design earthquake event. Liquefaction analyses are provided in Appendix C.

### 4.4 SEISMIC-INDUCED SETTLEMENT

To quantify the consequences of liquefaction at the site, seismic-induced settlement has been evaluated using the four CPT soundings. Robertson (NCEER 2001 and 2009) method was used for this evaluation.

Analyses were performed to evaluate the potential for seismic settlement from saturated liquefied and unsaturated dry soils. The calculated seismic-induced settlements of saturated soil using various methods for CPT analysis are ranging from 3.7 to 5.3 inches. Liquefaction induced-settlement analyses are provided in Appendix C.

Seismic-induced settlement can occur both above and below the groundwater table during a strong seismic event. We have estimated the dry seismic settlement using the Robertson and Shao (2010) Method. The total seismic dry settlement we calculated ranges from 0 to 0.2 inches.

## 5.0 CONCLUSIONS

### 5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible provided the recommendations presented in this report are incorporated into the design and construction of the project. Furthermore, it is also our opinion that the proposed development will not adversely impact the stability of adjoining properties.

### 5.2 GEOLOGIC HAZARDS

#### 5.2.1 Ground Rupture

No known active faults are known to project through the subject sites nor do the sites lie within the boundaries of an “Earthquake Fault Zone” as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The closest known active fault is the Puente Hills (Coyote Hills) fault located approximately 2.7 miles to the northeast. Therefore, potential for ground rupture due to an earthquake beneath the sites is considered low.

#### 5.2.2 Ground Shaking

The site is situated in a seismically active area that has historically been affected by generally moderate to occasionally high levels of ground motion. The site lies in relatively close proximity to several seismically active faults; therefore, during the life of the proposed improvements, the property will probably experience similar moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the Southern California region. Design and construction in accordance with the current California Building Code (CBC) requirements are anticipated to address the issues related to potential ground shaking.

#### 5.2.3 Landsliding

Geologic hazards associated with landsliding are not anticipated at the site since the site is relatively level.

#### 5.2.4 Liquefaction

Our analyses indicate liquefaction could in soils located below a depth of 10 feet if groundwater were to rise to shallowest historic levels concurrent with a strong ground motion. Liquefaction could lead to a total seismic settlement (saturated and dry) of the ground surface of up to 5.3 inches due to seismic consolidation during liquefaction. The differential settlement due to seismic settlement would likely be on the order of ½ of the total seismic settlement or approximately 2.7 inches over 30 feet. Lateral spreading is not a significant risk at the site in consideration of the relatively flat site topography and lack of an nearby channel face or slope.

Based on the State of California Special Publication (SP) 117A, the seismic-induced settlement at the site does not fall within the Level of Liquefaction Hazard of “Large-scale Displacements.” “Large scale Displacements” are defined as those that ***exceed*** 1-3 feet horizontally and 4-6 inches vertically. Therefore, the Level of Liquefaction Hazard is classified as a “Localized Failure.” One of the suitable mitigation alternatives presented in the SP 117A for Localized Failures is the use of reinforced shallow foundations and improved structural design to withstand predicted vertical and lateral ground displacements.

The SP 117A also stated that hazards from liquefaction should be mitigated to the extent required to reduce seismic risk to “acceptable levels.” The acceptable level of risk means, “that level that provides reasonable protection of the public safety” [California Code of Regulations Title 14, Section 3721 (a)]. The use of well-reinforced foundations, such as post-tensioned slabs, grade beams with structural slabs, or mat foundations have been proven to adequately provide basal support for similar structures during comparable liquefaction events. Specific recommendations to mitigate risks associated with liquefaction are provided in Section 6.3.

### 5.3 STATIC SETTLEMENT

As discussed in Section 4.2, analyses were performed to evaluate potential for static settlement of the underlying alluvium. Provided site grading is performed in accordance with the recommendations provided herein and based on the anticipated foundation loads, total and differential static settlement is not anticipated to exceed 1 inch and ½-inch over 30 feet, respectively, for the proposed commercial buildings. The estimated magnitudes of static settlements are considered within tolerable limits for the proposed structure.

## 5.4 EXCAVATION AND MATERIAL CHARACTERISTICS

Onsite earth materials are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment. The site earth materials are generally considered suitable for reuse as fill provided, they are cleared on deleterious debris and oversized rocks (greater than 4 inches in greatest dimension). Site materials are generally near the optimum moisture content. As such, fill soils derived from onsite soils will likely require the addition of nominal amounts of water and mixing in preparation for reuse as compacted fill.

Temporary construction slopes will be required to complete removal of unsuitable soils and for construction of underground utilities. Such excavations will require laybacks where they are surcharged or where they exceed 4 feet in height. Specific recommendations are provided in Section 6.1.8.

If encountered, portions of concrete debris and asphalt can likely be reduced in size (4 inches minus) and incorporated within fill soils during earthwork operations.

If onsite disposal systems, clarifiers, and other underground improvements are present beneath the site, these improvements will require proper abandonment or removal per the City guidelines.

## 5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate that the near surface soils will shrink about 5 to 10 percent when removed and replaced as compacted fill. Subsidence due to reprocessing of removal bottoms is anticipated to be about 0.05 feet. The estimates of shrinkage and subsidence are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during the grading process.

## 5.6 SOIL EXPANSION

Based on our laboratory test results and the USCS visual manual classification, the near-surface soils are generally anticipated to possess a **Low** expansion potential. Additional testing for soil expansion will be required prior to construction of foundations and other concrete work to confirm these conditions.

# 6.0 RECOMMENDATIONS

## 6.1 EARTHWORK

### 6.1.1 General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Artesia, California in addition to the recommendations presented herein.

### 6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of grading, we recommend a meeting be held between the developer, City Inspector, grading contractor, civil engineer, and geotechnical consultant to discuss the proposed grading and construction logistics. We also recommend a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site grading and foundation construction. This is to observe compliance with the design specifications and recommendations and to allow for design changes in the event that subsurface conditions differ from those anticipated. If conditions are encountered that appear to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

### 6.1.3 Site Clearing

Areas to be graded should be cleared of vegetation, existing asphalt and concrete, underground improvements to be abandoned and deleterious materials. Existing underground utility lines within the project area that will be protected in place and that fall within a 1 to 1 (H:V) plane projected down from the edges of footings may be subject to surcharge loads. Under such conditions, this office should be made aware of these conditions for evaluation of potential surcharging. Supplemental recommendations may be required to protect such improvements in place.

The project geotechnical consultant should be notified at the appropriate times to provide observation services during clearing operations to verify compliance with the above recommendations. Voids created by clearing and excavation should be left open for observation by the geotechnical consultant. Should any unusual soil conditions or subsurface structures be encountered during site clearing or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations as needed.

Temporary construction equipment (office trailers, power poles, etc.) should be positioned to allow adequate room for clearing and recommended ground preparation to be performed for proposed structures, pavements, and hardscapes.

### 6.1.4 Ground Preparation

In general, the artificial fill is considered unsuitable for support of the proposed development. Based on our exploratory borings, the depth of the artificial fill material is anticipated to be about 2 feet in depth. However, deeper fills are expected in the areas of the previous site improvements (i.e. utility lines) and where the current buildings and structures are located. All artificial fill soils should be removed to expose the underlying alluvial soils within the limits of the structures (buildings and site walls) and paving.

Removal of unsuitable materials should extend laterally beyond the limits of the proposed buildings a distance equal to the depth of removal (i.e. 1:1 projection) but not less than 5 feet. Removals within pavements and footings for site walls may be limited to the edge of foundations or pavement where lateral restrictions to removals are present such as property lines.

All removal and overexcavations should be evaluated by the geotechnical consultant during grading to confirm the exposed conditions are as anticipated and to provide supplemental recommendations if required.

### 6.1.5 Scarification

Prior to placement of compacted fill, the exposed ground should be scarified to a depth of 6 to 8 inches, moisture conditioned to at least 110 percent of the optimum moisture content, then compacted to at least 90 percent of the laboratory standard. The laboratory standard for maximum dry density and optimum moisture content for each soil type should be determined in accordance with ASTM D 1557.

### 6.1.6 Fill Placement

Materials excavated from the site may be reused as fill provided, they are free of deleterious materials and particles greater than 4 inches in maximum dimension (oversized materials). Asphaltic and concrete debris generated during site demolition or encountered within the existing fill can be incorporated within new fill soils during earthwork operations provided they are reduced to no more than 4 inches in maximum dimension. Such materials should be mixed thoroughly with fill soils to prevent nesting. All fill should be placed in lifts no greater than 8 inches in loose thickness, moisture conditioned to at least 110% of optimum moisture content, then compacted in place to at least 90 percent of the laboratory standard. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultant has approved the preceding lift.

### 6.1.7 Import Materials

If import materials are required to achieve the proposed finish grades, the proposed import soils should have an Expansion Index (EI, ASTM D 4829) less than 20 and possess negligible soluble sulfate concentrations. Import sources should be indicated to the geotechnical consultant prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

### 6.1.8 Temporary Excavations

Temporary construction slopes in site materials that are not surcharged may be cut vertically up to a height of 4 feet. Temporary excavations greater than 4 feet but no greater than 10 feet in height that are not surcharged should be laid back at a maximum gradient of 1:1 (H:V) or properly shored.

Excavations should not be left open for prolonged periods of time. The project geotechnical consultant should observe all temporary cuts to confirm anticipated conditions and to provide alternate recommendations if conditions dictate. All excavations should conform to the requirements of Cal/OSHA. The current removal requirement is 12 inches below existing grade and not expected to undermine the existing foundations. However, if deeper removals are required during grading and where insufficient room exists for recommended lay back cuts, shoring or slot cutting methods may be required. Additional recommendations for such conditions can be provided at that time based on the observed materials and subsequent lab testing.

## 6.2 SEISMIC DESIGN PARAMETERS

### 6.2.1 Mapped Seismic Design Parameters

For design of the project in accordance with Chapter 16 of the 2019 CBC, the mapped seismic parameters may be taken as presented in the tables below.

**TABLE 6.1**
**2019 CBC Mapped Seismic Design Parameters**

| Parameter | Value |
|---|---|
| Site Class | D |
| Mapped $MCE_R$ Spectral Response Acceleration, short periods, $S_S$ | 1.552 |
| Mapped $MCE_R$ Spectral Response Acceleration, at 1-sec. period, $S_1$ | 0.552 |
| Site Coefficient, $F_a$ | 1.0 |
| Site Coefficient, $F_v$ | 1.7445* |
| Adjusted $MCE_R$ Spectral Response Acceleration, short periods, $S_{MS}$ | 1.552 |
| Adjusted $MCE_R$ Spectral Response Acceleration, at 1-sec. period, $S_{M1}$ | 0.968 |
| Design Spectral Response Acceleration, short periods, $S_{DS}$ | 1.034 |
| Design Spectral Response Acceleration, at 1-sec. period, $S_{D1}$ | 0.645 |
| Long-Period Transition Period, $T_L$ (sec.) | 8 |
| Seismic Design Category for Risk Categories I-IV | II |

$MCE_R$ = Risk-Targeted Maximum Considered Earthquake

*According to Section 11.4.8 in ASCE 7-16, "a ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following structures on Site Class D and E sites with $S_1$ greater than or equal to 0.2." However, "A ground motion hazard analysis is not required for structures where: Structures on Site Class D sites with $S_1$ greater than or equal to 0.2, provided the value of the seismic response coefficient $C_s$ is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$." The $F_v$ value of 1.7 above from Table 11.4-2 assumes that this exception is met and that a ground motion hazard analysis is not required. Should this exception not be met, the site-specific seismic design parameters provided in the next section should be used.

### 6.2.2 Site-Specific Seismic Design Parameters

In addition to the Code Spectra parameters presented in Table 6.1, we have performed a site-specific ground motion hazard analysis in accordance with Chapter 21 of ASCE 7-16 to obtain site-specific seismic design acceleration parameters, the risk-targeted maximum considered earthquake response spectrum, and the design earthquake response spectrum. The site-specific seismic design parameters are presented below.

**TABLE 6.2**
**2019 CBC Site Specific Seismic Design Parameters**

| Parameter | Value |
|---|---|
| Site Class | D |
| Site Coefficient, $F_a$ | 1.0 |
| Site Coefficient, $F_v$ | 2.5 |
| Adjusted MCE Spectral Response Acceleration, short periods, $S_{MS}$ | 1.833 |
| Adjusted MCE Spectral Response Acceleration, at 1-sec. period, $S_{M1}$ | 1.822 |
| Design Spectral Response Acceleration, short periods, $S_{DS}$ | 1.222 |
| Design Spectral Response Acceleration, at 1-sec. period, $S_{D1}$ | 1.214 |

MCE = Maximum Considered Earthquake

## 6.3 CONVENTIONAL FOUNDATION DESIGN

### 6.3.1 General

As discussed in Section 5.2.4, the site is prone to liquefaction. To mitigate this condition, we recommend structures be supported by a foundation system consisting of a mat, post-tensioned slab, or spread footings tied together with a structural slab with grade beams. For this project, we have assumed the foundation system will utilize spread footings tied together with a structural slab with grade beams. Specific recommendations for this system is provided in the following sections. Recommendations for other systems can be provided upon request.

The following design parameters are provided to assist the project structural engineer to design foundations for structures at the site. These design parameters are based on typical site materials encountered during subsurface exploration and are provided for preliminary design and estimating purposes. The project geotechnical consultant should provide final design parameters following observation and testing of site materials during grading. Depending on actual materials encountered during site grading, the design parameters presented herein may require modification.

### 6.3.2 Soil Expansion

The recommendations presented herein are based on soils with a **Low** expansion potential. Following site grading, additional testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations. If site soils with higher expansion potentials are encountered or imported to the site, the recommendations contained herein may require modification.

### 6.3.3 Static and Seismic Settlement

Based on anticipated foundation loads and provided that the recommendations for ground preparation in this report are followed, total and differential static settlement are anticipated to be less than 1 inch and ½ inch over 30 feet, respectively. These values are considered within tolerable limits of proposed structures and site improvements. Design of the structures should consider these maximum anticipated settlements.

### 6.3.4 Allowable Bearing Value

Foundations may utilize a bearing value of 2,000 pounds per square foot (psf) for continuous and pad footings a minimum width of 12 inches and founded at a minimum depth of 12 inches below the lowest adjacent grade. This value may be increased by 260 psf and 700 psf for each additional foot in width and depth, respectively, up to a maximum value of 4,000 psf. Recommended allowable bearing values include both dead and live loads may be increased by one-third for wind and seismic forces.

### 6.3.5 Lateral Resistance

For foundations that are founded in the native alluvial soils or compacted fill, a passive earth pressure of 300 pounds per square foot per foot of depth (psf/ft) up to a maximum value of 1,500 pounds per square foot (psf) may be used to determine lateral bearing for footings. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.33 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. No increase in the coefficient of friction should be used when designing for wind and seismic forces.

The above values are based on footings placed directly against compacted fill or competent native soils. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of the laboratory standard.

### 6.3.6 Footing Dimensions and Reinforcement

Exterior and interior pad footings should be founded at a minimum depth of 12 inches below the lowest adjacent grade and have a minimum width of 12 inches. All continuous footings should have a minimum depth of 12 inches below lowest adjacent grade and a minimum width of 12 inches.

All continuous footings should be reinforced with a minimum of four No. 5 bars, two top and two bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein.

All isolated pad footings should be tied in both directions with a concrete grade beam to the nearest foundation. Grade beams should be at least 12 inches wide by 12 inches deep and be reinforced with four No. 5 bars, two top and two bottom. Reinforcing for the grade beams should tie into the adjacent footings.

### 6.3.7 Slabs on Grade

Interior concrete slabs constructed on grade should be a minimum 6 inches thick and should be reinforced with No. 3 bars spaced 18 inches each way. Care should be taken to ensure the placement of reinforcement at mid-slab height. The structural engineer may recommend a greater slab thickness and reinforcement based on proposed use and loading conditions and such recommendations should govern if greater than the recommendations presented herein. No. 4 tie bars should be provided between the slab and connecting grade beams at a spacing of 18 inches.

For consideration of point loading that may occur on the slab, a subgrade of modulus, $K_{V1}$, of 100 pci may be used.

Concrete floor slabs in areas to receive carpet, tile, or other moisture sensitive coverings should be underlain with a moisture vapor barrier 10-mil Visqueen, or equal. The membrane should be properly lapped, sealed, and protected with at least 2 inches of sand having an SE of 30 or more. This vapor barrier system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Special consideration should be given to slabs in areas to receive ceramic tile or other rigid, crack-sensitive floor coverings. Design and construction of such areas should mitigate hairline cracking as recommended by the structural engineer.

Block-outs should be provided around interior columns to permit relative movement and mitigate distress to the floor slabs due to differential settlement that will occur between column footings and adjacent floor subgrade soils as loads are applied.

Prior to placing concrete, subgrade soils below slab-on-grade areas should be thoroughly moistened to provide a moisture content that is equal to or greater than 110% of the optimum moisture content to a depth of 12 inches.

#### 6.3.8 Foundation Observations

Foundation excavation should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended above. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

### 6.4 RETAINING AND SCREENING WALLS

#### 6.4.1 General

The following preliminary design and construction recommendations are provided for general retaining and screen walls supported by engineered compacted fill or competent native soils. Final wall designs specific to the site development should be provided for review once completed. The structural engineer and architect should provide appropriate recommendations for sealing at all joints and applying moisture-proofing material on the back of the walls.

#### 6.4.2 Allowable Bearing Value and Lateral Resistance

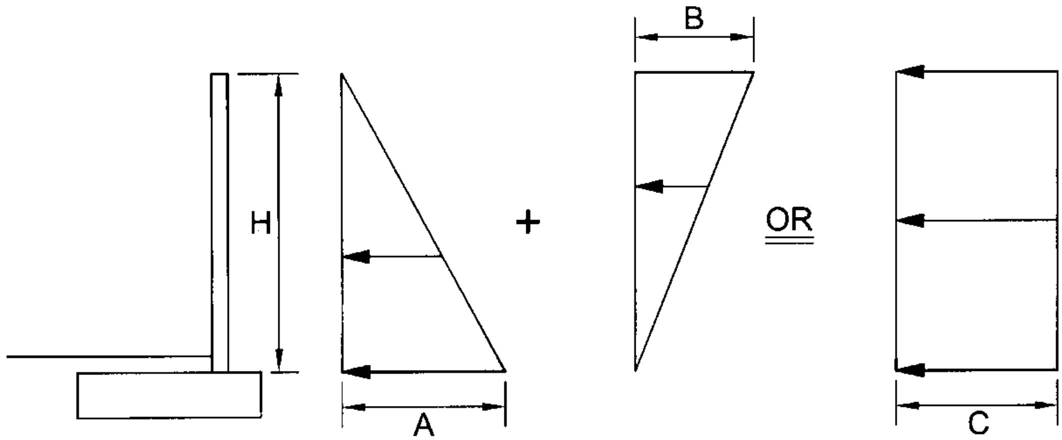
Design of retaining and screen walls may utilize the bearing and lateral resistance values provided in Sections 6.3.4 and **Error! Reference source not found.**.

#### 6.4.3 Active Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.3. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.42 g for 10% probability of exceedance in 50 years. As indicated in Section 1803.5.12 of the 2019 CBC, retaining walls supporting 10 feet of backfill or less are not required to be designed for seismic earth pressures. The values provided in the following table do not consider hydrostatic pressure. Retaining walls should also be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

**TABLE 6.3**

**SEISMIC EARTH PRESSURES**
**Pressure Diagram**




Static Component

Seismic Component

Total Force

**Pressure Values**
**Walls Up To 10 Feet High**

| Value | Backfill Condition | |
|---|---|---|
| | Level | 2H:1V Slope |
| **A** | 33H | 54H |
| **B** | 12.5H | 12.5H |
| **C** | 23H | 33.5H |
| Note: H is in feet and resulting pressure is in psf. Design may utilize either the sum of the static component and the seismic component force diagrams or the total force diagram above. SEAOSC has suggested using a load factor of 1.7 for the static component and 1.0 for the seismic component. The actual load factors should be determined by the structural engineer. | | |

### 6.4.4 Drainage and Moisture-Proofing

Retaining walls should be constructed with a perforated pipe and gravel subdrain to prevent entrapment of water in the backfill. The perforated pipe should consist of 4-inch-diameter, ABS SDR-

35 or PVC Schedule 40 with the perforations laid down. The pipe should be embedded in ¾- to 1½-inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend at least one foot up the wall above the footing and drainage outlet. Drainage gravel and piping should not be placed below outlets and weepholes. Filter fabric should consist of Mirafi 140N, or equal. Outlet pipes should be directed to positive drainage devices.

The use of weepholes may be considered in locations where aesthetic issues from potential nuisance water are not a concern. Weepholes should be 2 inches in diameter and provided at least every 6 feet on center. Where weepholes are used, perforated pipe may be omitted from the gravel subdrain.

Retaining walls supporting backfill should also be coated with a moisture-proofing compound or covered with such material to inhibit infiltration of moisture through the walls. Moisture-proofing material should cover any portion of the back of wall that will be in contact with soil and should lap over and onto the top of footing. A drainage panel should be provided between the soil backfill and water proofing. The panel should extend from the top of the backdrain gravel up to within 12 inches of finish grade. The top of footing should be finished smooth with a trowel where moisture proofing-materials are applied to inhibit the infiltration of water through the wall. The project structural engineer should provide specific recommendations for moisture-proofing, water stops, and joint details.

### 6.4.5 Footing Reinforcement and Wall Jointing

All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. Walls should be provided with cold joints spaced no more than 20 feet apart. Wall finishes and capping materials should not extend across the cold joint. The structural engineer may require different reinforcement or jointing and should dictate if greater than the recommendations provided herein. Where recommended removals are limited due to space restrictions, greater reinforcement and closer jointing may be recommended. Specific recommendations should be provided by the geotechnical consultant during grading based on as-built conditions exposed in the field.

### 6.4.6 Foundation Observations

Footing excavations should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

## 6.5 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 7 feet in each direction. Flatwork having a minimum dimension more than 7 feet should be reinforced with No. 3 bars spaced 18 inches center to center each way or 6-inch by 6-inch, W4 by W4 welded wire mesh. Special jointing detail should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to at least 110 percent of the optimum moisture content to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete. The geotechnical consultant should observe and verify the density and moisture content of subgrade soils prior to pouring concrete to ensure that the required compaction and pre-moistening recommendations have been met.

Drainage from flatwork areas should be directed to local area drains and/or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. The concrete flatwork should also be sloped at a minimum gradient of 0.5 percent away from building foundations and retaining walls.

## 6.6 CONCRETE MIX DESIGN

Laboratory testing of near-surface soils for soluble sulfate content indicates soluble sulfate concentration of 0.002%. We recommend following the procedures provided in ACI 318, Section 19.3.1, Table 19.3.1.1 for **S0** sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

## 6.7 CORROSION

Results of preliminary testing of soils for pH, chloride, and minimum resistivity indicate the site is potentially **Highly Corrosive** to metals that are in contact or close proximity to onsite soils. As such, specific recommendations should be obtained from a corrosion specialist if construction will include metals that will be near or in direct contact with site soils.

## 6.8 PRELIMINARY PAVEMENT DESIGN

### 6.8.1 Preliminary Pavement Structural Sections

Based on the soil conditions present at the site and an estimated traffic index, preliminary pavement sections are provided in the table below. A laboratory tested "R-value" of 33 was used for the near-surface soil in this preliminary pavement design. The sections provided below are for planning purposes only and should be re-evaluated subsequent to site grading. Final pavement sections should be based on actual R-value testing of in-place soils and analysis of anticipated traffic.

### 6.8.2 Subgrade Preparation

Prior to placement of paving elements, subgrade soils should be scarified 6 inches, moisture-conditioned to above the optimum moisture content then compacted to at least 90 percent of the maximum dry density determined in accordance with ASTM D1557. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding engineered compacted soil or aggregate base materials.

### 6.8.3 Aggregate Base

Aggregate base materials should be Crushed Aggregate Base or Crushed Miscellaneous Base conforming to Section 200-2 of the Standard Specification for Public Works Construction (Greenbook) or Class 2 Aggregate Base conforming to the Caltrans' Standard Specifications. The materials should be moisture conditioned to slightly over the optimum moisture content then compacted to at least 95 percent of ASTM D 1557.

**TABLE 6.4**
**PRELIMINARY PAVEMENT STRUCTURAL SECTIONS**

| Location | Traffic Index | AC (inches) | PCC (inches) | AB (inches) |
|---|---|---|---|---|
| Truck Drive Areas | 7.5 | 4.0 | -- | 11.0 |
| | | -- | 8.0 | -- |
| Parking Drives | 5.0 | 3.0 | -- | 5.0 |
| | | -- | 5.0 | -- |
| Parking Stalls | -- | 3.0 | -- | 5.0 |

### 6.8.4 Asphaltic Concrete

Paving asphalt should be PG 64-10 conforming to the requirements of Section 203-1 of the Greenbook. Asphalt concrete materials should conform to Section 203-6 and construction should conform to Section 302 of the Greenbook.

### 6.8.5 Portland Cement Concrete

Portland cement concrete used to construct concrete paving should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 3,250 pounds per square inch (psi) at 28 days. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. Such joints should be properly sealed. Where traffic will traverse over cold joints without keyways or dowels or edges of concrete paving, the edges should be thickened by 20% of the design thickness toward the edge over a horizontal distance of 5 feet.

## 6.9 POST GRADING CONSIDERATIONS

### 6.9.1 Site Drainage and Irrigation

The ground immediately adjacent to foundations should be provided with positive drainage away from the structures in accordance with 2019 CBC, Section 1804.4. Based on soil and climatic conditions, the ground slope within 10 feet of the buildings may be reduced to 2%. No rain or excess water should be allowed to pond against structures such as walls, foundations, flatwork, etc.

Excessive irrigation water can be detrimental to the performance of the proposed site development. Water applied in excess of the needs of vegetation will tend to percolate into the ground. Such percolation can lead to nuisance seepage and shallow perched groundwater. Seepage can form on slope faces, on the faces of retaining walls, in streets, or other low-lying areas. These conditions could lead to adverse effects such as the formation of stagnant water that breeds insects, distress or damage of trees, surface erosion, slope instability, discoloration and salt buildup on wall faces, and premature failure of pavement. Excessive watering can also lead to elevated vapor emissions within buildings that can damage flooring finishes or lead to mold growth inside the home.

Key factors that can help mitigate the potential for adverse effects of overwatering include the judicious use of water for irrigation, use of irrigation systems that are appropriate for the type of vegetation and geometric configuration of the planted area, the use of soil amendments to enhance moisture retention, use of low-water demand vegetation, regular use of appropriate fertilizers, and seasonal adjustments of irrigation systems to match the water requirements of vegetation. Specific recommendations should be provided by a landscape architect or other knowledgeable professional.

### 6.9.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.8 of this report. Trench excavations must also conform to the requirements of Cal/OSHA.

Trench backfill materials and compaction criteria should conform to the requirements of the local municipalities. As a minimum, utility trench backfill should be compacted to at least 90 percent of the laboratory standard. Materials placed within the pipe zone (6 inches below and 12 inches above the pipe) should consist of particles no greater than ¾ inches and have a SE of at least 30. The materials within the pipe zone should be moisture-conditioned and compacted by hand-operated compaction equipment. Above the pipe zone (>1 foot above pipe), the backfill may consist of general fill materials. Trench backfill should be moisture-conditioned to slightly over the optimum moisture content, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. For trenches with sloped walls, backfill material should be placed in lifts no greater than 8 inches in loose thickness, and then compacted by rolling with a sheepsfoot roller or similar equipment. The project geotechnical consultant should perform density testing along with probing to verify that adequate compaction has been achieved.

Within shallow trenches (less than 18 inches deep) where pipes may be damaged by heavy compaction equipment, imported clean sand having a SE of 30 or greater may be utilized. The sand should be placed in the trench, thoroughly watered, and then compacted with a vibratory compactor. For utility trenches located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base or crossing footing trenches, concrete or slurry should be used as trench backfill.

## 6.10 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend ***Albus & Associates, Inc.*** be engaged to review any future development plans, including civil plans (grading plans), foundation plans, and proposed structural loads, prior to construction. This is to verify that the assumptions of this report are valid and that the preliminary conclusions and recommendations contained in this report have been properly interpreted and are incorporated into the project plans and specifications. If we are not provided the opportunity to review these documents, we take no responsibility for misinterpretation of our preliminary conclusions and recommendations.

We recommend that a geotechnical consultant be retained to provide soil engineering services during construction of the project. These services are to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly from the assumed development described herein, the project geotechnical consultant should review our preliminary design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report or subsequent design reports, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

## 7.0 LIMITATIONS

This report is based on the proposed development and geotechnical data as described herein. The materials encountered on the project site and utilized in our laboratory testing for this investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil and bedrock materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty.

This report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

This report has been prepared for the exclusive use of **G3 Urban Company** and their project consultants in the planning and design of the proposed development. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

This report is subject to review by the controlling governmental agency.

Respectfully submitted,

***ALBUS & ASSOCIATES, INC.***

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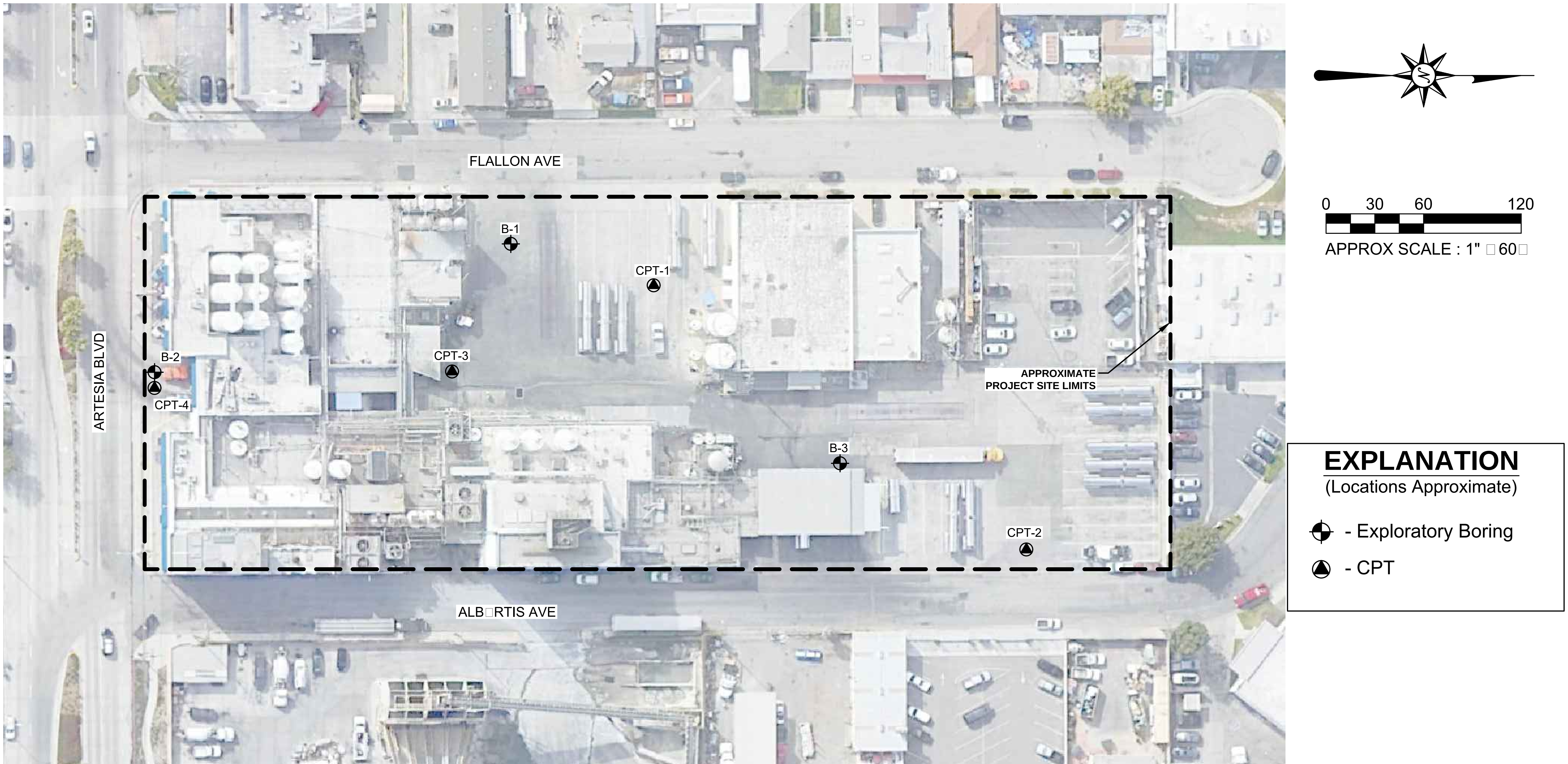



Google 2021

ALBUS & ASSOCIATES

GEOTECHNICAL MAP

| Job No.: 3027.00 | Date: 12/07/2021 | Plate: 1 |
|---|---|---|

# APPENDIX A

## EXPLORATION LOGS AND CONE PENETRATION TEST RESULTS

# Field Identification Sheet

Light gray

Gray

Dark gray

Gray Brown

Light brown

Brown

Dark Brown

Olive brown

Olive

Yellow

Yellowish brown

Yellowish red

Red

Reddish Brown

Tan

**Description Order:**

Description, Color, Moisture, Density, Grain Size, Additional Description

| Description | % | Example |
|---|---|---|
| | 0-5 | Sand |
| trace | 5-15 | Sand trace Silt |
| with | 15-30 | Sand with Silt |
| | 30+ | Silty Sand |

More Examples

Sand with Silt trace Clay

Sand trace Silt and Clay

Sand with Silt and Clay

Gravelly Sand with Silt trace Clay

Silty Clay with Sand trace Gravel

**Moisture**

| | |
|---|---|
| Dry | absence of water |
| Damp | below optimum |
| Moist | near optimum |
| Very Moist | above optimum |
| Wet | free water visible |

**Density (Navfac)**

| Coarse grained soils | SPT | CA |
|---|---|---|
| Very Loose | 0-3 | 0-5 |
| Loose | 3-8 | 5-13 |
| Medium Dense | 8-14 | 13-22 |
| Dense | 14-25 | 22-40 |
| Very Dense | 25> | 40> |

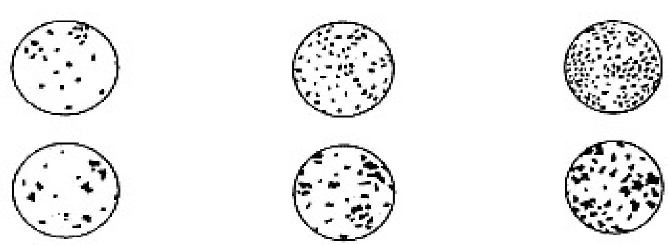
| Fine grained soils | | |
|---|---|---|
| Very Soft | 2< | 0-3 |
| Soft | 2-4 | 3-6 |
| Medium Stiff | 4-8 | 6-13 |
| Stiff | 8-15 | 13-24 |
| Very Stiff | 15-30 | 24-48 |
| Hard | 30> | 48> |

**Grain Size**

| Description | | Sieve Size | Approx. Size |
|---|---|---|---|
| Boulders | | >12" | Larger than basketball |
| Cobbles | | 3-12" | Fist to basketball |
| Gravel | coarse | 3/4-3" | Thumb to Fist |
| | fine | #4-3/4" | Pea to Thumb |
| Sand | coarse | #10-4 | Rock Salt to Pea |
| | medium | #40-10 | Sugar to Rock Salt |
| | fine | #200-40 | Flour to Sugar |
| Fines | | Pass #200 | Smaller than Flour |

**Additional Description (ie. roots, pinhole pores, debris, etc.)**

Trace 5%    Moderate 15%    Abundant 30%



# EXPLORATION LOG

| Project: | | Location: |
|---|---|---|
| Address: | | Elevation: |
| Job Number: | Client: | Date: |
| Drill Method: | Driving Weight: | Logged By: |

| Depth (feet) | Lith-ology | Material Description | Water | Samples: Blows Per Foot | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| | | **EXPLANATION** | | | | | | | |
| | | Solid lines separate geologic units and/or material types. | | | | | | | |
| 5 | | Dashed lines indicate unknown depth of geologic unit change or material type change. | | | | | | | |
| | | **Solid black rectangle** in Core column represents California Split Spoon sampler (2.5in ID, 3in OD). | | | | | | | |
| | | **Double triangle** in core column represents SPT sampler. | | | | | | | |
| 10 | | **Vertical Lines** in core column represents Shelby sampler. | | | | | | | |
| | | **Solid black rectangle** in Bulk column respresents large bag sample. | | | | | | | |
| 15 | | **Other Laboratory Tests:**<br>Max = Maximum Dry Density/Optimum Moisture Content<br>EI = Expansion Index<br>SO4 = Soluble Sulfate Content<br>DSR = Direct Shear, Remolded<br>DS = Direct Shear, Undisturbed<br>SA = Sieve Analysis (1" through #200 sieve)<br>Hydro = Particle Size Analysis (SA with Hydrometer) | | | | | | | |
| 20 | | 200 = Percent Passing #200 Sieve<br>Consol = Consolidation<br>SE = Sand Equivalent<br>Rval = R-Value<br>ATT = Atterberg Limits | | | | | | | |

*Albus & Associates, Inc.*

Plate A-1

# EXPLORATION LOG

| | | |
|---|---|---|
| Project: | | Location: B-1 |
| Address: 180 Flallon Avenue, Artesia, CA 90701 | | Elevation: 54.4 |
| Job Number: 3027.00 | Client: G3 Urban | Date: 11/2/2021 |
| Drill Method: Hollow-Stem Auger | Driving Weight: 140 lbs / 30 in | Logged By: ddalbus |

| Depth (feet) | Material Description | Water | Blows Per Foot | Moisture Content (%) | Dry Density (pcf) | Other Lab Tests |
|---|---|---|---|---|---|---|
| | Concrete | | | | | Max EI SO4 DS RVal pH Resist Ch Consol ATT |
| | **ARTIFICIAL FILL (Af)** Sandy Silt (ML): Grayish brown, moist, fine grained sand | | 14 | 22.4 | 100.7 | |
| | **ALLUVIUM (Qal)** Sandy Silt (ML): Grayish brown, moist, stiff, fine grained sand, moderate pinhole pores | | 15 | 19.1 | 104.5 | |
| 5 | @ 4 ft, more fine grained sand present | | | | | |
| | Sand trace Silt (SP): Light gray, moist, loose, fine to medium grained sand | | 9 | 11.9 | 100.5 | |
| 10 | Sandy Silt / Silty Sand (ML/SM): Grayish brown, very moist, stiff / medium dense, fine grained sand | | 17 | 20.1 | 99.8 | |
| | | ▽ | | | | |
| 15 | Sand trace Silt (SP): Grayish brown, wet, medium dense, fine to medium grained sand | | 14 | | | |
| 20 | Sandy Silt (SM): Grayish brown, wet, medium stiff, fine grained sand | | 4 | | | 200 |
| 25 | Sand trace Silt (SP): Light gray, wet, medium dense, fine to medium grained sand | | 16 | | | |
| | Total Depth 26.5 feet<br>Groundwater 14 feet<br>Boring backfilled with bentonite and capped with concrete | | | | | |

# EXPLORATION LOG

| Project: | | Location: B-2 |
|---|---|---|
| Address: 180 Flallon Avenue, Artesia, CA 90701 | | Elevation: 53.4 |
| Job Number: 3027.00 | Client: G3 Urban | Date: 11/2/2021 |
| Drill Method: Hollow-Stem Auger | Driving Weight: 140 lbs / 30 in | Logged By: ddalbus |

| Depth (feet) | Lith-ology | Material Description | Water | Samples: Blows Per Foot | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| | | Concrete | | | | | | | |
| | | **ARTIFICIAL FILL (Af)**<br>Sandy Silt trace Clay (ML): Grayish brown, damp to moist, fine grained sand | | 17 | | | 24.2 | 97.9 | Consol |
| 5 | | **ALLUVIUM (Qal)**<br>Sandy Silt trace Clay (ML): Grayish brown, damp to moist, medium dense, fine grained sand, trace pinhole pores, decayed rootlets | | 15 | | | 19.4 | 100.5 | |
| | | Silty Sand / Sandy Silt (SM/ML): Grayish brown, moist, medium dense / stiff, fine grained sand | | 12 | | | 15.7 | 90.7 | |
| | | Silty Sand (SM): Light grayish brown, very moist, loose, fine grained sand | | | | | | | |
| 10 | | Sandy Silt trace Clay (ML): Grayish brown, very moist, stiff, fine grained sand | | 18 | | | 12.6 | 98.8 | Consol |
| | | Silty Sand (SM): Grayish brown, very moist, medium dense, fine grained sand | | | | | | | |
| | | Sand trace Silt (SP): Grayish brown, very moist, medium dense, fine to medium grained sand | ▽ | | | | | | |
| 15 | | Silty Sand (SM): Grayish brown, wet, medium dense, fine grained sand, significant fines<br>@ 16.3 ft, Gray | | 12 | | | | | |
| 20 | | Silty Sand (SM): Grayish brown, wet, loose, fine grained sand | | 6 | | | | | 200 |
| 25 | | Sand trace Silt (SP): Light grayish brown, wet, dense, fine to medium grained sand | | 19 | | | | | |

***Albus & Associates, Inc.***

# EXPLORATION LOG

| Project: | | Location: B-2 |
|---|---|---|
| Address: 180 Flallon Avenue, Artesia, CA 90701 | | Elevation: 53.4 |
| Job Number: 3027.00 | Client: G3 Urban | Date: 11/2/2021 |
| Drill Method: Hollow-Stem Auger | Driving Weight: 140 lbs / 30 in | Logged By: ddalbus |

| Depth (feet) | Lith-ology | Material Description | Water | Samples: Blows Per Foot | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| | | Silty Sand trace Clay (SM): Grayish brown, wet, medium dense, fine grained sand | | 8 | | | | | 200 |
| 35 | | Sand trace Silt (SP): Grayish brown, wet, dense, fine to medium grained sand | | 25 | | | | | |
| 40 | | | | 13 | | | | | 200 |
| | | Sandy Silt (ML): Gray, wet, very stiff, fine grained sand | | | | | | | |
| 45 | | Sand (SP): Grayish brown, wet, very dense, fine to medium grained sand | | 35 | | | | | |
| 50 | | | | 11 | | | | | |
| | | Sandy Silt (ML): Grayish brown, wet, very stiff, fine grained sand | | | | | | | |
| | | Total Depth 51.5 feet<br>Groundwater 14 feet<br>Boring backfilled with bentonite and capped with concrete | | | | | | | |

# EXPLORATION LOG

| | |
|---|---|
| Project: | Location: B-3 |
| Address: 180 Flallon Avenue, Artesia, CA 90701 | Elevation: 57 |
| Job Number: 3027.00 — Client: G3 Urban | Date: 11/2/2021 |
| Drill Method: Hollow-Stem Auger — Driving Weight: 140 lbs / 30 in | Logged By: ddalbus |

| Depth (feet) | Lith-ology | Material Description | Water | Samples: Blows Per Foot | Samples: Core | Samples: Bulk | Laboratory Tests: Moisture Content (%) | Laboratory Tests: Dry Density (pcf) | Laboratory Tests: Other Lab Tests |
|---|---|---|---|---|---|---|---|---|---|
| | | concrete with rebar | | | | | | | |
| | | **ARTIFICIAL FILL (Af)**<br>Silty Sand / Sandy Silt (SM/ML): Grayish brown, moist, fine grained sand | | 15 | | | 20.9 | 102.9 | |
| | | **ALLUVIUM (Qal)**<br>Silty Sand / Sandy Silt (SM/ML): Grayish brown, moist, medium dense / stiff, fine grained sand, trace pinhole pores | | 13 | | | 19.2 | 101.8 | |
| 5 | | @ 4 ft, loose / stiff | | 11 | | | 15.3 | 97 | Consol |
| | | Silty Sand (SM): Grayish brown, moist, loose, fine grained sand | | | | | | | |
| 10 | | @ 10 ft, very moist, medium dense | | 17 | | | 15.4 | 93.2 | |
| 15 | | Sand trace Silt and Clay (SP): Grayish brown, wet, loose, fine grained sand | | 4 | | | | | |
| | | Silty Sand (SM): Gray, wet, loose, fine to medium grained sand | | | | | | | |
| 20 | | Silty Sand with Clay (SM): Grayish brown, wet, loose, fine grained sand | | 4 | | | | | 200 |
| 25 | | Sand (SP): Grayish brown, wet, medium dense, fine to medium grained sand | | 11 | | | | | |
| | | Total Depth 26.5 feet<br>Groundwater 14 feet<br>Boring backfilled with bentonite and capped with concrete | | | | | | | |

| | |
|---|---|
| ***Albus & Associates, Inc.*** | Plate A-5 |

# SUMMARY

# OF

# CONE PENETRATION TEST DATA

Project:

**180 Flallon Avenue**
**Artesia, CA**
**November 2, 2021**

Prepared for:


**Mr. Danny Albus**
**Albus & Associates**
**1011 N. Armando Street**
**Anaheim, CA  92806-2606**
**Office (714) 630-1626 / Fax (714) 630-1916**


Prepared by:


KTE

**KEHOE TESTING & ENGINEERING**

5415 Industrial Drive
Huntington Beach, CA  92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

# TABLE OF CONTENTS

# SUMMARY
# OF
# CONE PENETRATION TEST DATA

## 1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 180 Flallon Avenue in Artesia, California. The work was performed by Kehoe Testing & Engineering (KTE) on November 2, 2021. The scope of work was performed as directed by Albus & Associates personnel.

## 2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at four locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

| LOCATION | DEPTH OF CPT (ft) | COMMENTS/NOTES: |
|---|---|---|
| **CPT-1** | 50 | |
| **CPT-2** | 50 | |
| **CPT-3** | 50 | |
| **CPT-4** | 50 | |

**TABLE 2.1 - Summary of CPT Soundings**

## 3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone with a cone net area ratio of 0.83. The following parameters were recorded at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

## 4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

**KEHOE TESTING & ENGINEERING**

Steven P. Kehoe
President

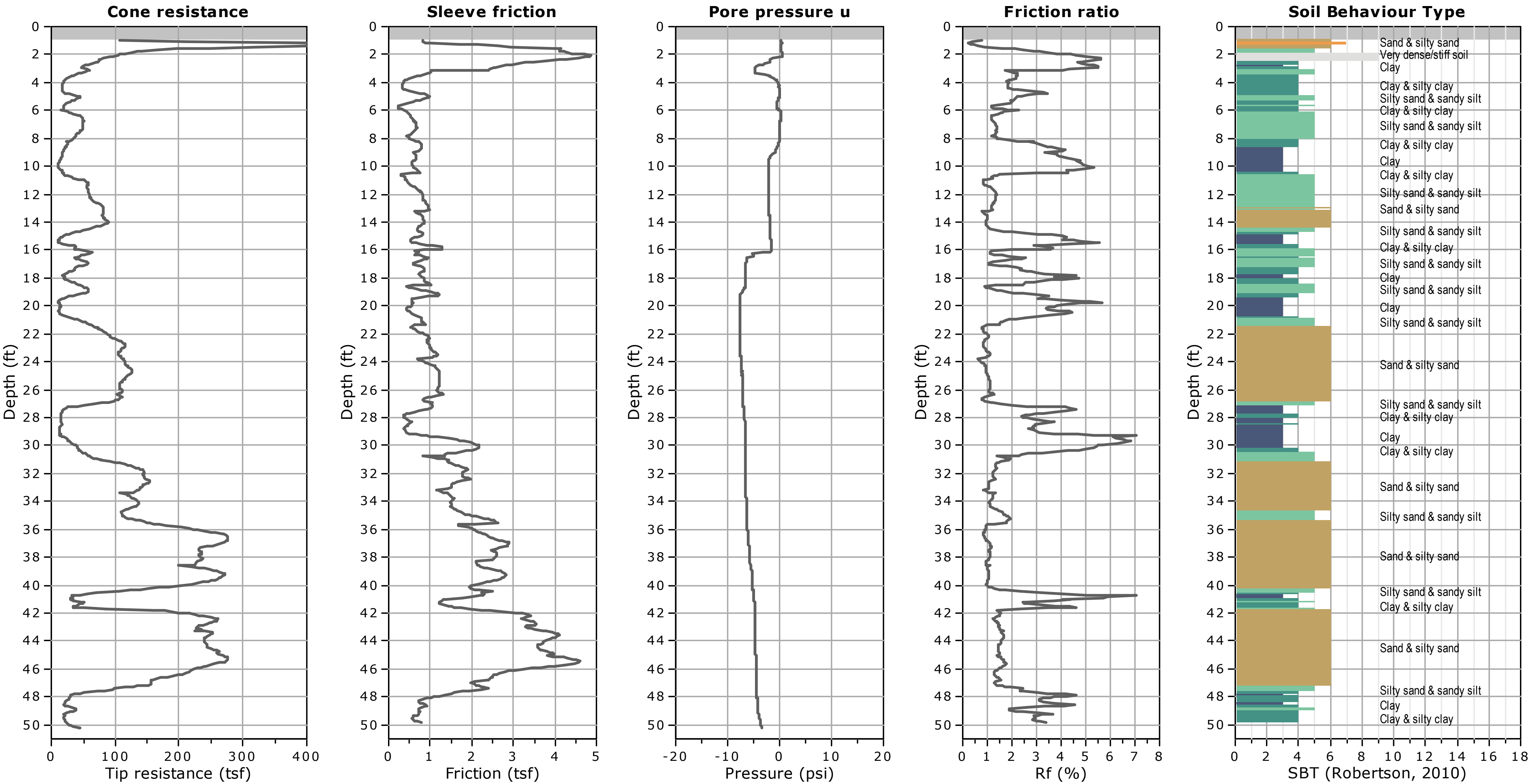
11/04/21-hh-3536

# APPENDIX

**Kehoe Testing and Engineering**
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

**Project:** **Albus & Associates**
**Location:** **180 Fallon Ave, Artesia, CA**

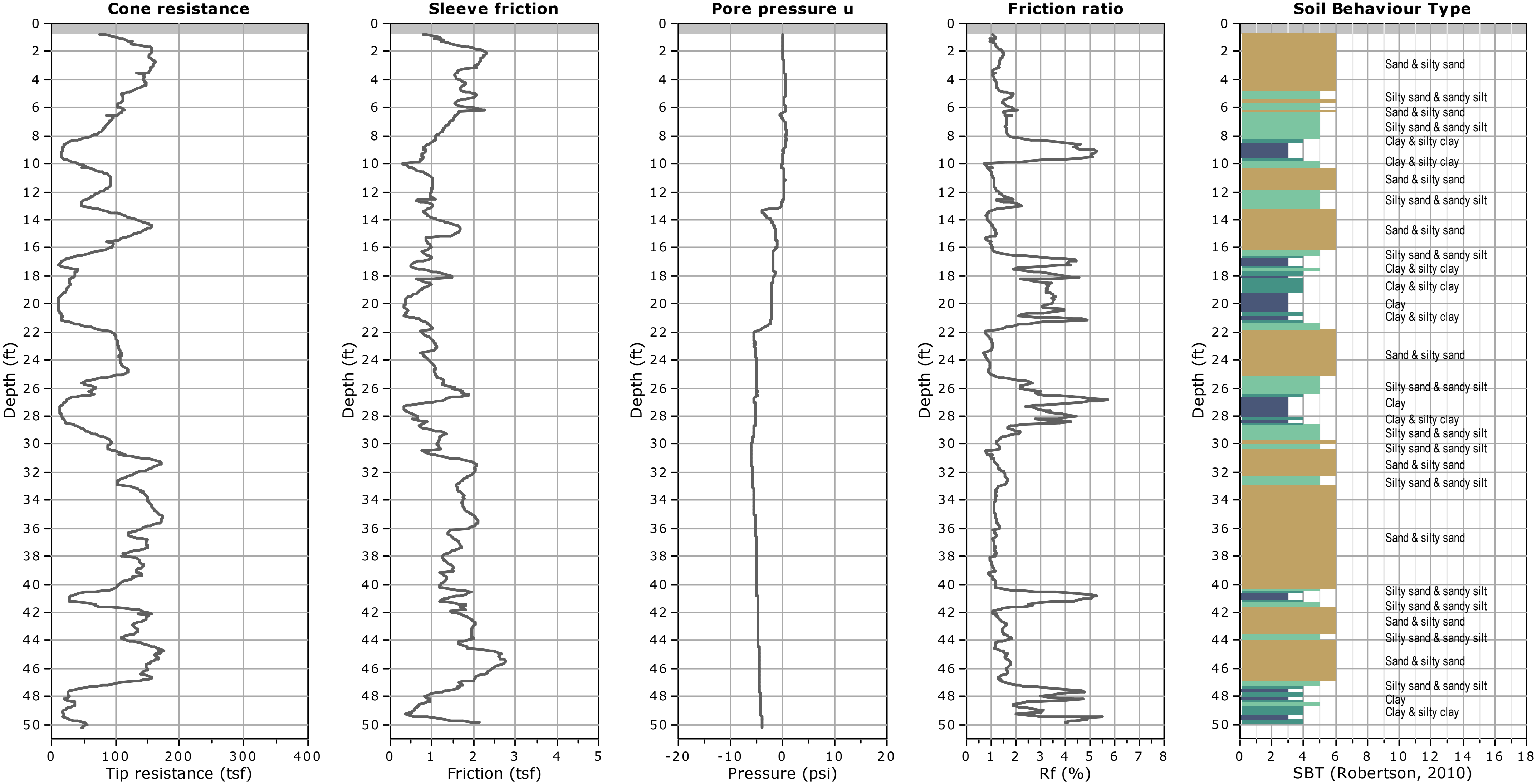
**CPT-1**
Total depth: 50.20 ft, Date: 11/2/2021



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

**Project:** Albus & Associates
**Location:** 180 Fallon Ave, Artesia, CA

**CPT-2**
Total depth: 50.22 ft, Date: 11/2/2021



Kehoe Testing and Engineering
714-901-7270
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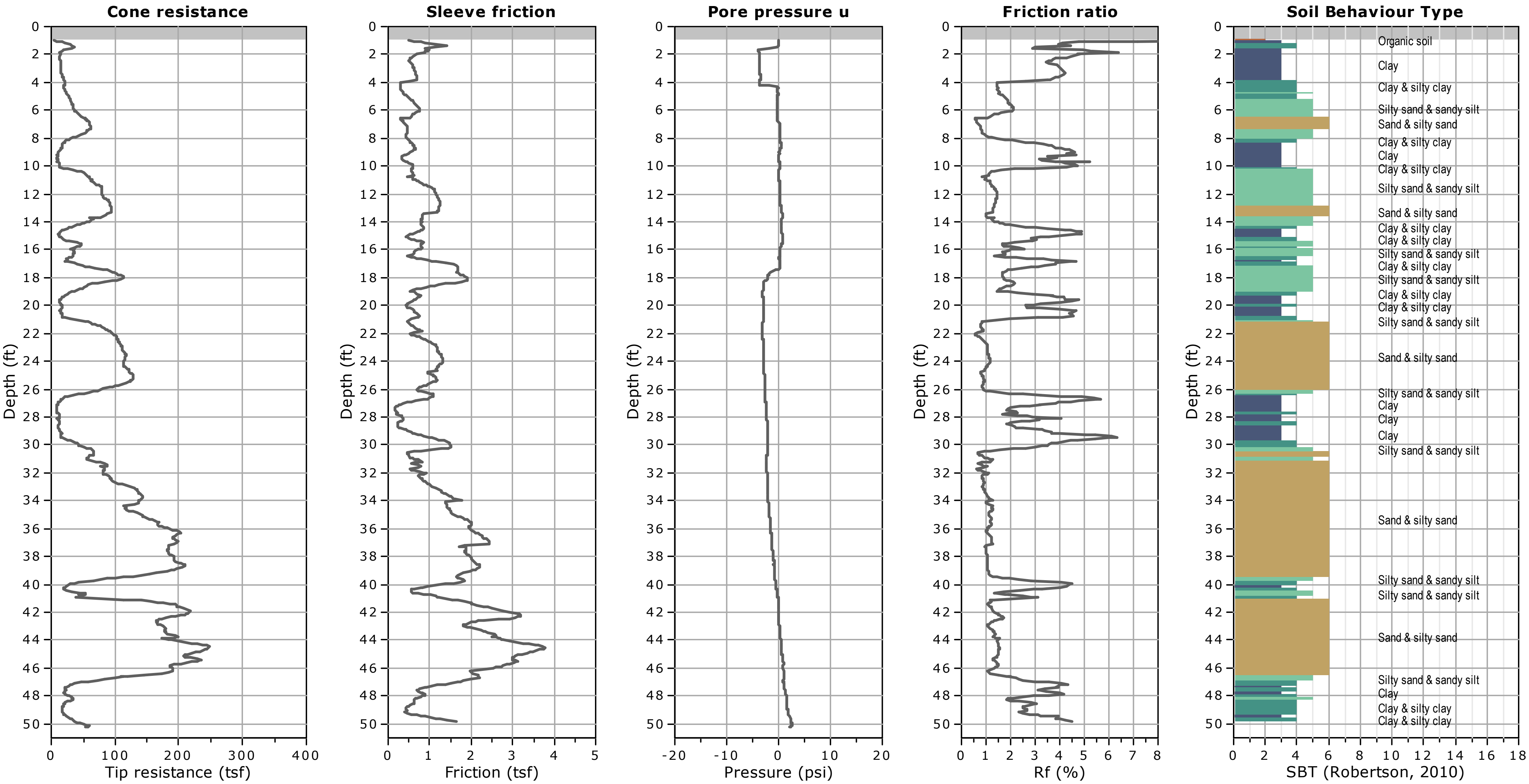
www.kehoetesting.com

**Project:** Albus & Associates
**Location:** 180 Fallon Ave, Artesia, CA

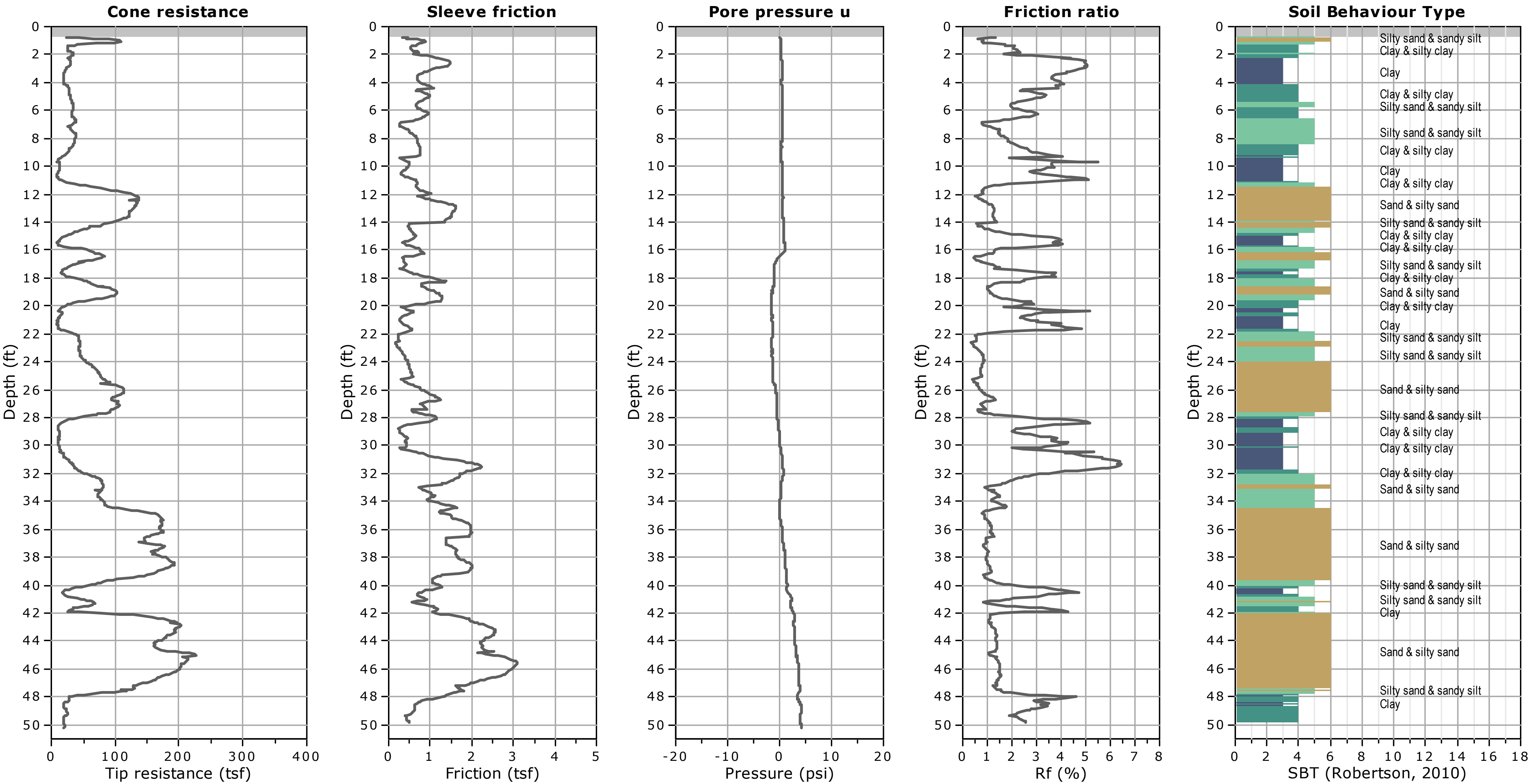
**CPT-3**
Total depth: 50.24 ft, Date: 11/2/2021



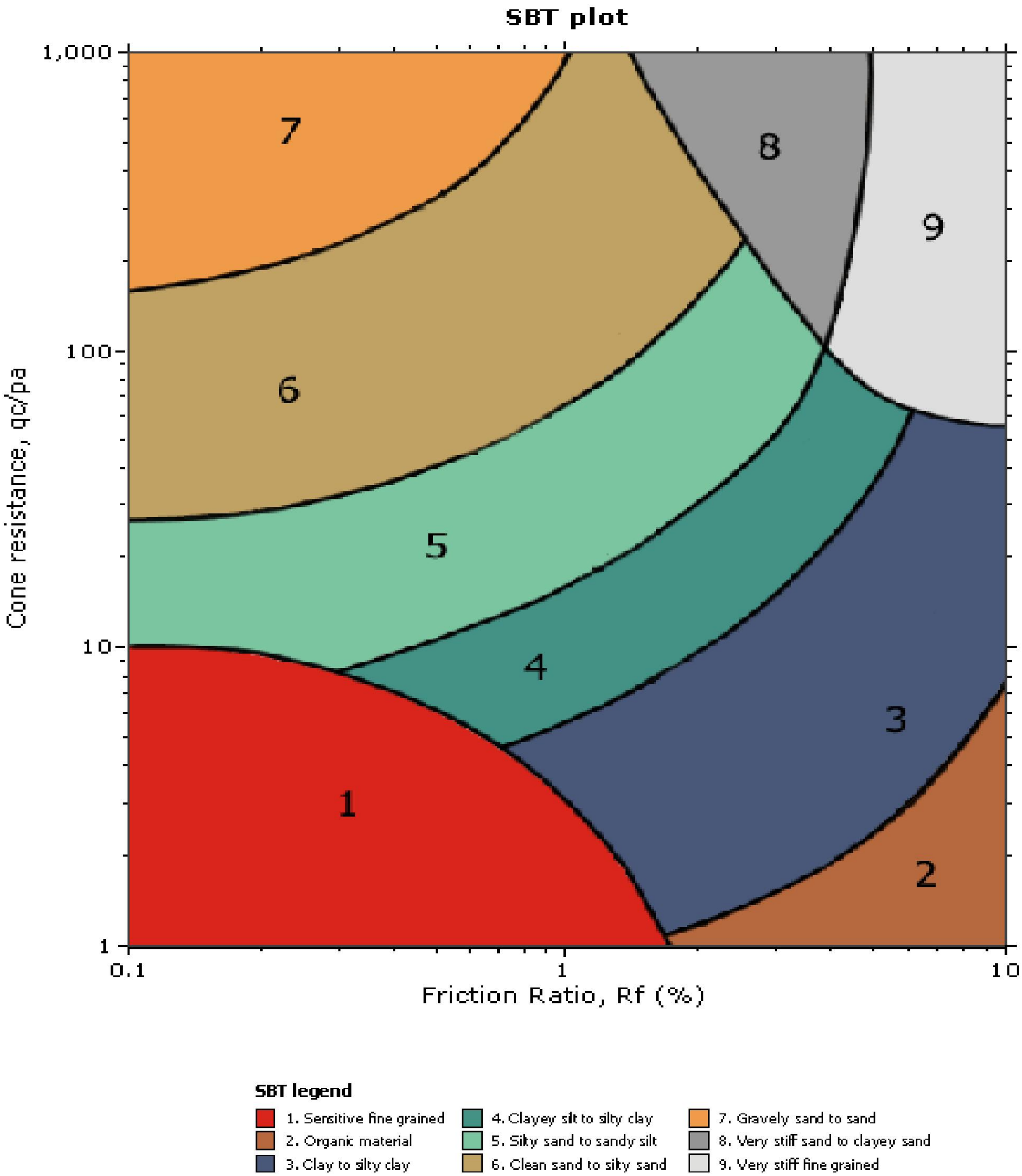
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

**Project:** Albus & Associates
**Location:** 180 Fallon Ave, Artesia, CA

**CPT-4**
Total depth: 50.22 ft, Date: 11/2/2021



**Kehoe Testing & Engineering**
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com



# APPENDIX B

## LABORATORY TEST RESULTS

# LABORATORY TESTING PROGRAM

## Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D 2488). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented on the Exploration Logs provided in Appendix A.

## In-Situ Moisture Content and Dry Density

Moisture content and dry density of in-place soil materials were determined in representative strata. Test data are summarized on the Exploration Logs, Appendix A.

## Atterberg Limits

Atterberg Limits (Liquid Limit, Plastic Limit, and Plasticity Index) were performed in accordance with Test Method ASTM D 4318. Pertinent test values are presented within Table B-1.

## Maximum Dry Density and Optimum Moisture Content

Maximum dry density and optimum moisture content were performed on a representative sample of the site materials obtained from our field explorations. The test was performed in accordance with ASTM D 1557. Pertinent test values are given in Table B-1.

## Expansion Potential

Expansion index testing was performed on a selected sample. The test was performed in accordance with ASTM D4829. The test result and expansion potential are presented in Table B-1.

## Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample and intact samples obtained from one our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The bulk sample was remolded to 90 percent of maximum dry density and at the optimum moisture content. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-5.

## Consolidation

Consolidation tests were performed for selected soil samples in general conformance with ASTM D 2435. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample. Loads were applied in geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The specific test samples were inundated at selected loads to evaluate the effects of a sudden increase in moisture content (hydro-consolidation potential). Results of the tests are graphically presented on Plates B-1 to B-4.

**Soluble Sulfate Content**

A chemical analysis was performed on a selected sample to determine soluble sulfate content. This test was performed in our soil laboratory in accordance with California Test Method No 417. The test result is included on Table B-1.

**Corrosion**

Select samples were tested for minimum resistivity, chloride, pH in accordance with California Test Method 643. Results of these tests are provided in Table B-1.

**R-Value**

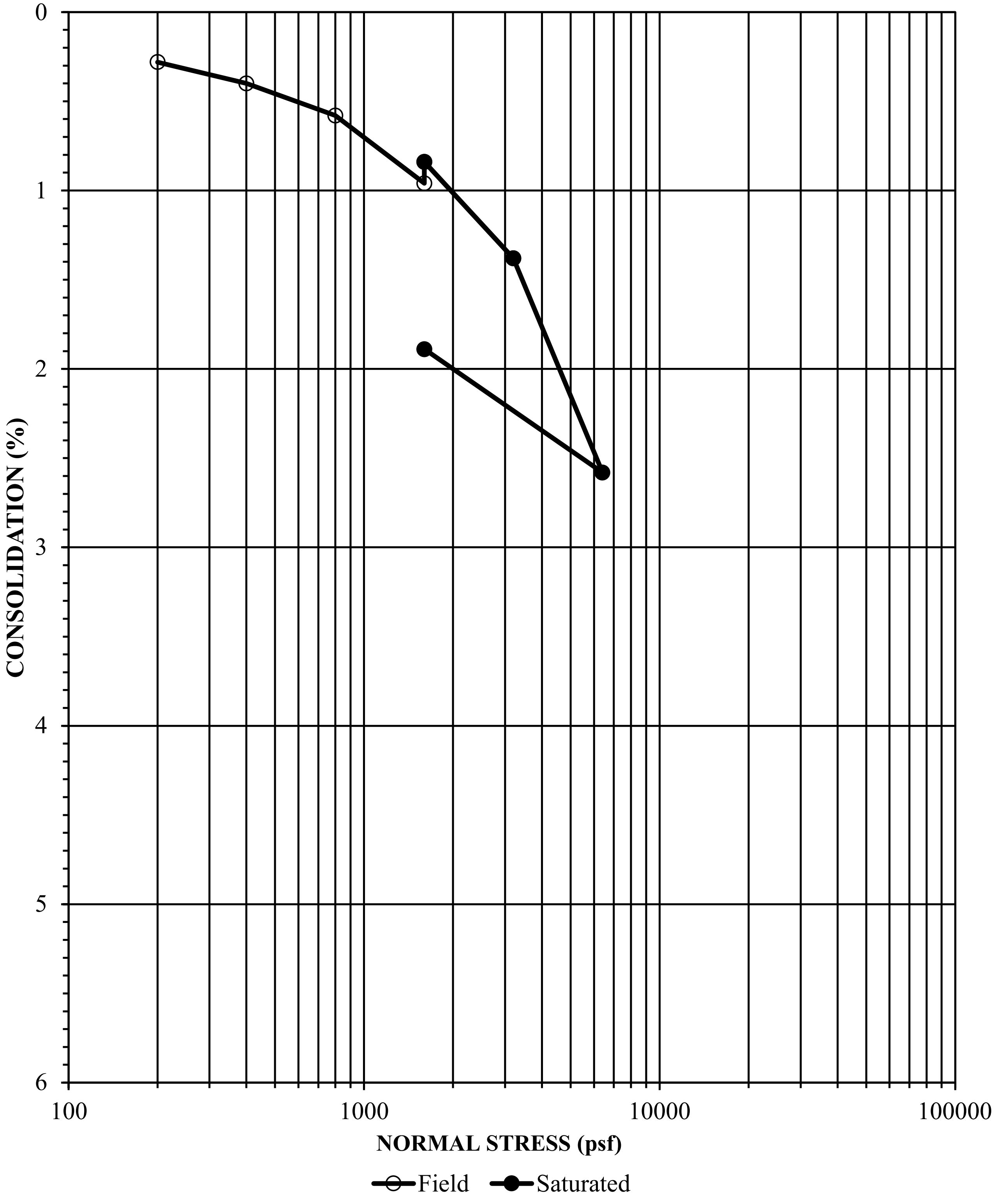
A sample of soil was tested for R-value in accordance with California Test Method (CTM) 301. The results are summarized in Plate B-6.

**TABLE B-1**
**SUMMARY OF LABORATORY TEST RESULTS**

| Boring No. | Sample Depth (ft.) | Soil Description | Test Results | |
|---|---|---|---|---|
| B-1 | 0-5 | Sandy Silt trace Clay | Maximum Dry Density (pcf):<br>Optimum Moisture (%):<br>Expansion Index:<br>Expansion Potential:<br>Soluble Sulfate Content (%):<br>Sulfate Exposure:<br>R-Value (By Exudation):<br>Resistivity (ohm-cm):<br>Chloride (ppm):<br>pH: | 113.5<br>15.5<br>23<br>Low<br>0.002<br>Negligible<br>43<br>2500<br>160<br>9.45 |
| B-1 | 2 | Sandy Silt | Liquid Limit (%):<br>Plasticity Index (%): | 32.7<br>9 |
| B-1 | 20 | Sandy Silt | Passing No. 200 Sieve (%): | 53.4 |
| B-2 | 20 | Silty Sand | Passing No. 200 Sieve (%): | 46.6 |
| B-2 | 30 | Silty Sand | Passing No. 200 Sieve (%): | 38.5 |
| B-2 | 40 | Sand trace Silt | Passing No. 200 Sieve (%): | 7.2 |
| B-3 | 20 | Silty Sand with Clay | Passing No. 200 Sieve (%): | 53.2 |

Note: Additional laboratory test results are provided on the boring logs provided in Appendix A.

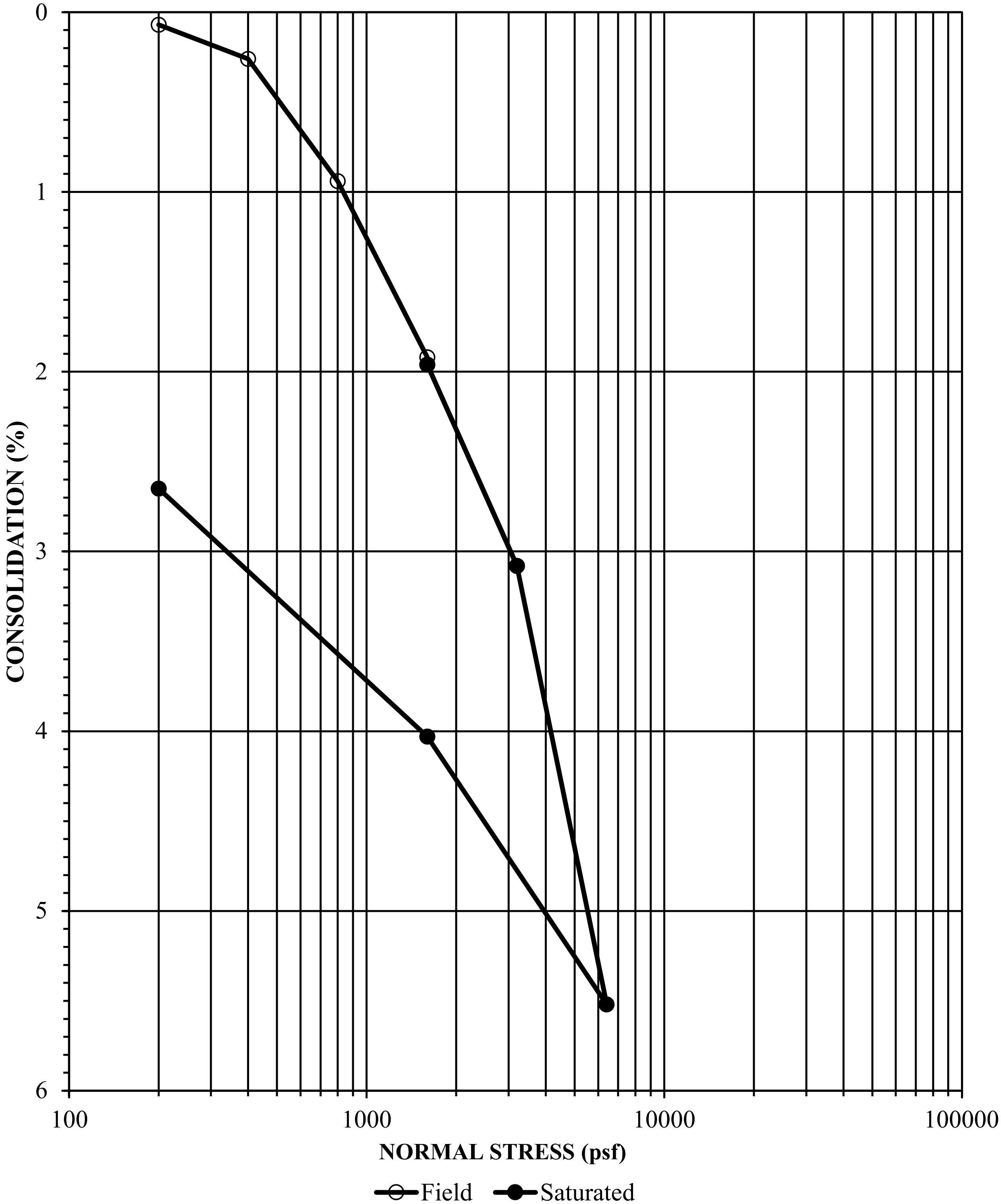
## CONSOLIDATION




| Job Number | Location | Depth | Description |
|---|---|---|---|
| 3027.00 | B-1 | 2 | Sandy Silt (ML) |

| Initial Dry Density (pcf) | Initial Moisture Content (%) | Final Moisture Concent (%) |
|---|---|---|
| 99 | 23.4 | 24.8 |

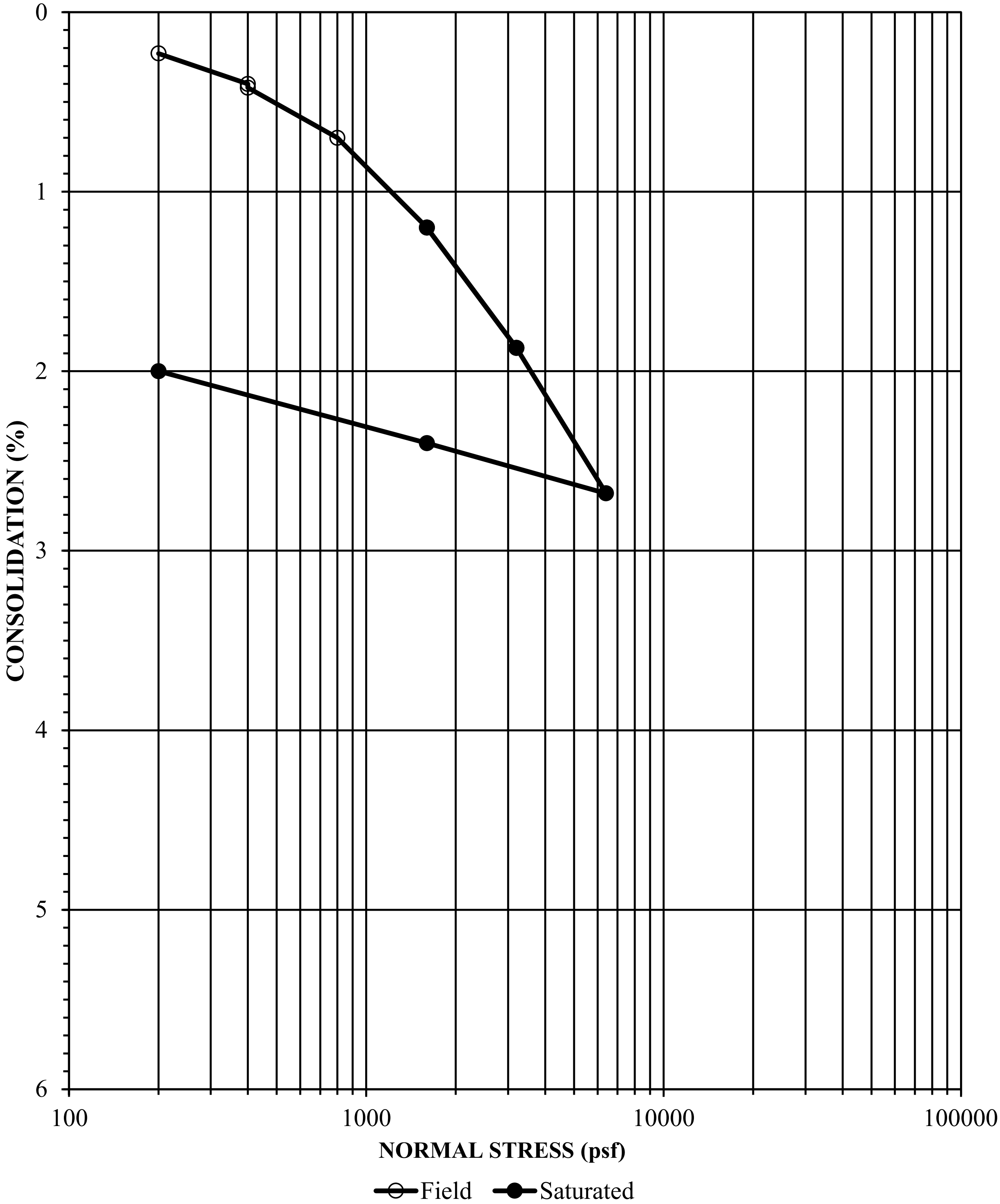
## CONSOLIDATION




| Job Number | Location | Depth | Description |
|---|---|---|---|
| 3027.00 | B-2 | 2 | Sandy Silt (ML) |

| Initial Dry Density (pcf) | Initial Moisture Content (%) | Final Moisture Concent (%) |
|---|---|---|
| 100.1 | 22.3 | 25.5 |

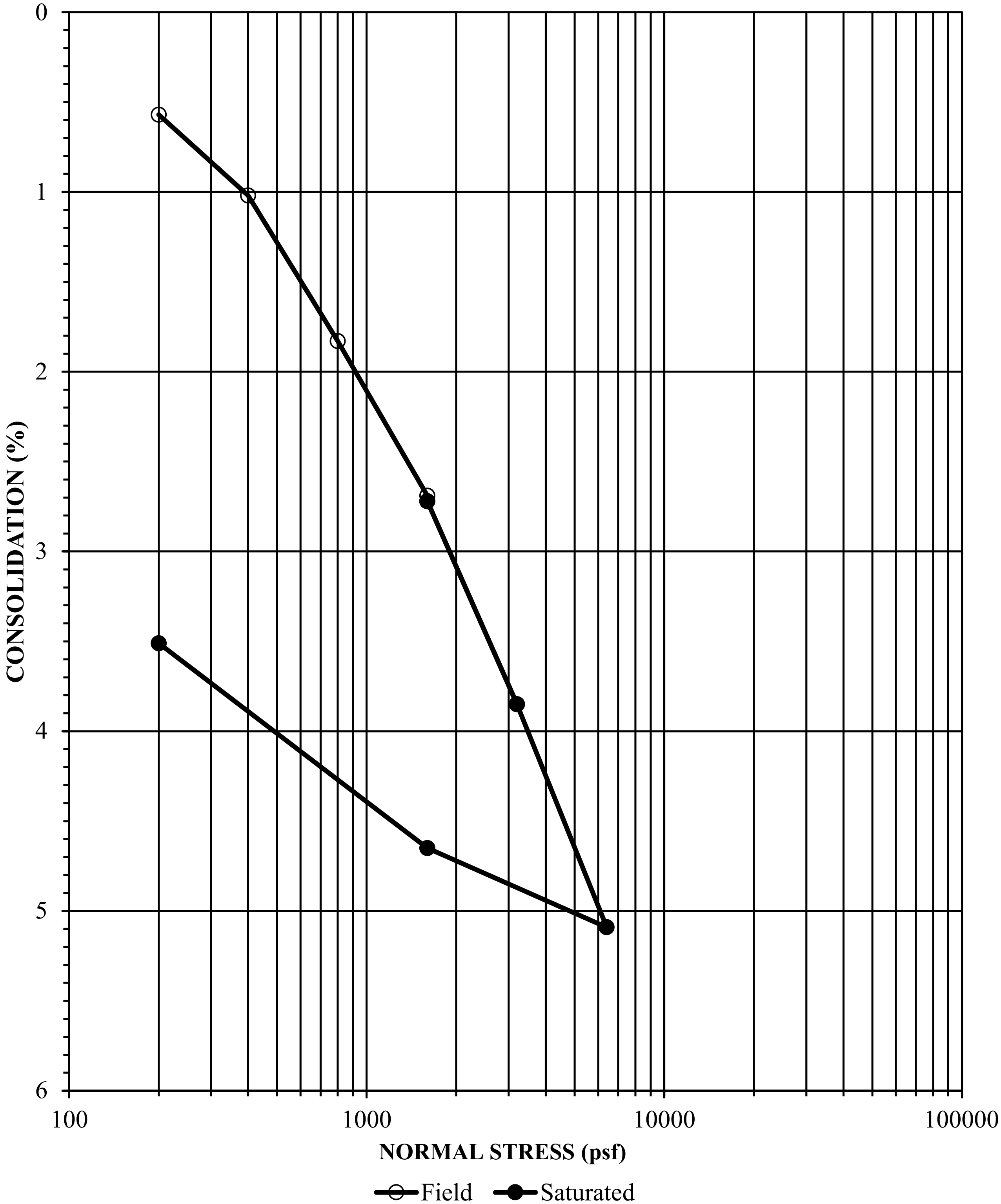
## CONSOLIDATION




| Job Number | Location | Depth | Description |
|---|---|---|---|
| 3027.00 | B-2 | 10 | Sandy Silt (ML) |

| Initial Dry Density (pcf) | Initial Moisture Content (%) | Final Moisture Concent (%) |
|---|---|---|
| 94.7 | 17.1 | 28.3 |

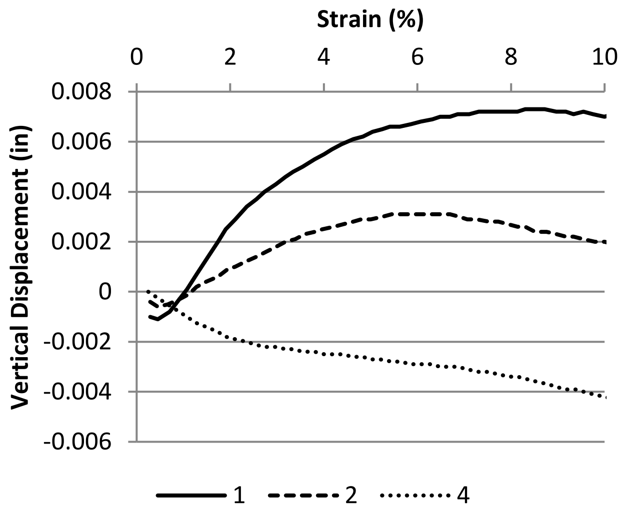
# CONSOLIDATION

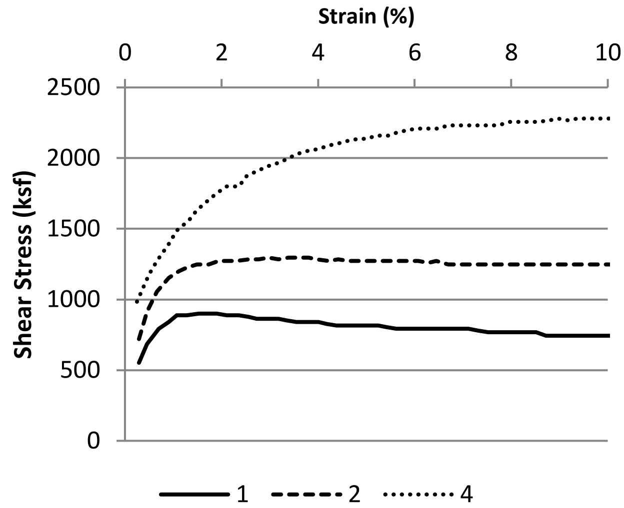


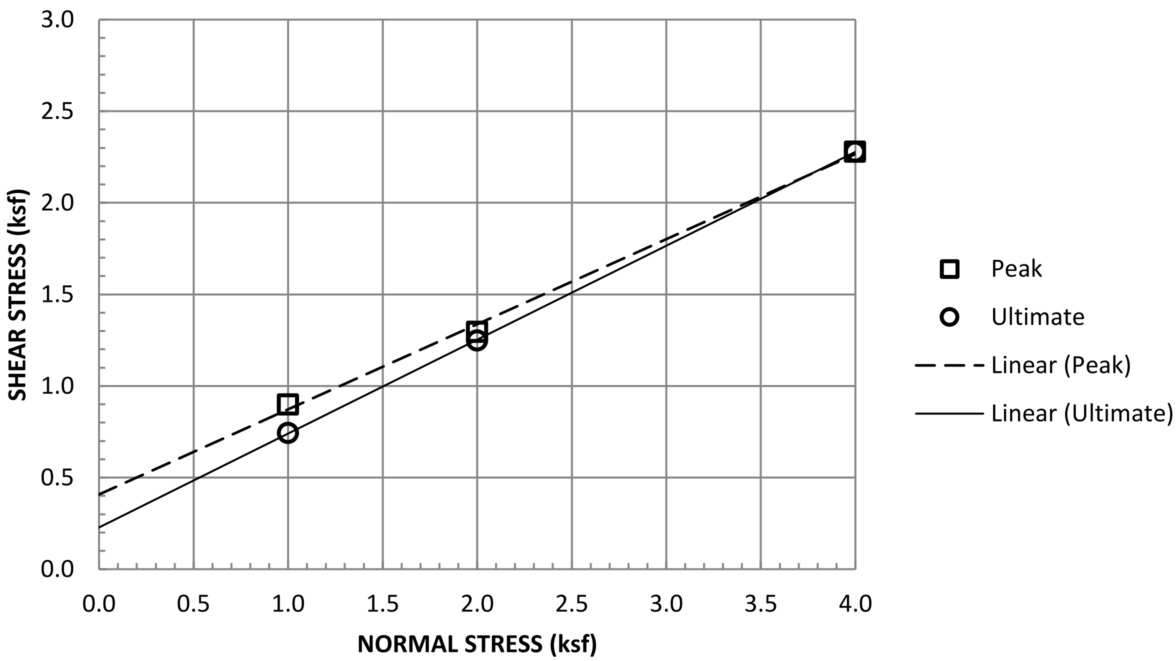

| Job Number | Location | Depth | Description |
|---|---|---|---|
| 3027.00 | B-3 | 6 | Silty Sand (SM) |

| Initial Dry Density (pcf) | Initial Moisture Content (%) | Final Moisture Concent (%) |
|---|---|---|
| 98.9 | 9 | 18.3 |

# DIRECT SHEAR




| | | | |
|---|---|---|---|
| Sample Type: | Remolded, Saturated | | |
| Normal Stress (ksf) | 1 | 2 | 4 |
| Peak Shear Stress (ksf) | 0.9 | 1.296 | 2.28 |
| Peak Displacement (in) | 0.007 | 0.003 | 0.004 |
| Ultimate Shear Stress (ksf) | 0.744 | 1.248 | 2.28 |
| Ultimate Displacement (in) | 0.25 | 0.25 | 0.25 |
| Initial Dry Density (pcf) | 95.9 | 95.5 | 95.5 |
| Initial Moisture Content (%) | 23.2 | 23.2 | 23.2 |
| Final Moisture Content (%) | 32 | 32 | 32.7 |
| Strain Rate (in/min) | 0.05 | | |

| Job Number | Location | Depth | Description |
|---|---|---|---|
| 3027.00 | B-1 | 0-5 | Sandy Silt |

# 'R' VALUE CA 301

Client: Albus

Date: 11/9/21

By: LD

Client's Job No.: 3027.00

Sample : B-1 @ 0 - 5'

GLA Reference: 2005-011

Soil Type: Brown, Sandy Clay

| TEST SPECIMEN | | A | B | C | D |
|---|---|---|---|---|---|
| Compactor Air Pressure | psi | **200** | **300** | **250** | |
| Initial Moisture Content | % | **17.3** | **17.3** | **17.3** | |
| Water Added | ml | **0** | **-15** | **-8** | |
| Moisture at Compaction | % | 17.3 | 15.9 | 16.5 | |
| Sample & Mold Weight | gms | **3174** | **3192** | **3163** | |
| Mold Weight | gms | **2103** | **2098** | **2099** | |
| Net Sample Weight | gms | 1071 | 1094 | 1064 | |
| Sample Height | in. | **2.5** | **2.511** | **2.466** | |
| Dry Density | pcf | 110.6 | 113.9 | 112.2 | |
| Pressure | lbs | **2835** | **7550** | **4620** | |
| Exudation Pressure | psi | 226 | 601 | 368 | |
| Expansion Dial | x 0.0001 | **27** | **115** | **70** | |
| Expansion Pressure | psf | 117 | 498 | 303 | |
| Ph at 1000lbs | psi | **38** | **22** | **30** | |
| Ph at 2000lbs | psi | **82** | **48** | **60** | |
| Displacement | turns | **4.69** | **4.05** | **4.29** | |
| R' Value | | 34 | 59 | 49 | |
| Corrected 'R' Value | | **34** | **59** | **49** | |

| FINAL 'R' VALUE | |
|---|---|
| By Exudation Pressure (@ 300 psi): | **43** |
| By Epansion Pressure : | **33** |
| TI = 5 | |

Geo-Logic ASSOCIATES

# APPENDIX C

## LIQUEFACTION ANALYSIS

**Albus & Associates, Inc.**
Geotechnical Consultants
1011 N. Armando Street, Anaheim, CA
albus-keefe.net

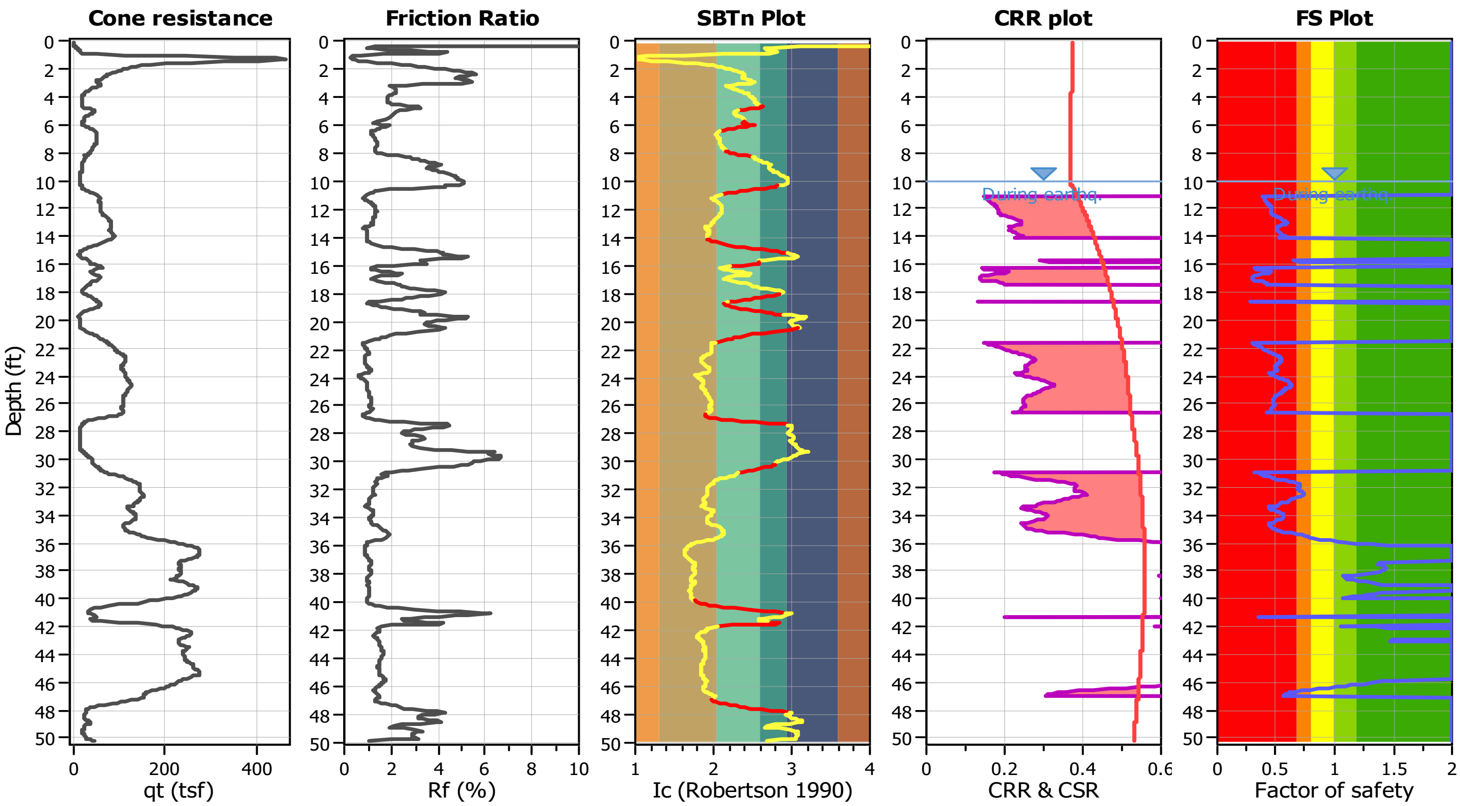
# LIQUEFACTION ANALYSIS REPORT

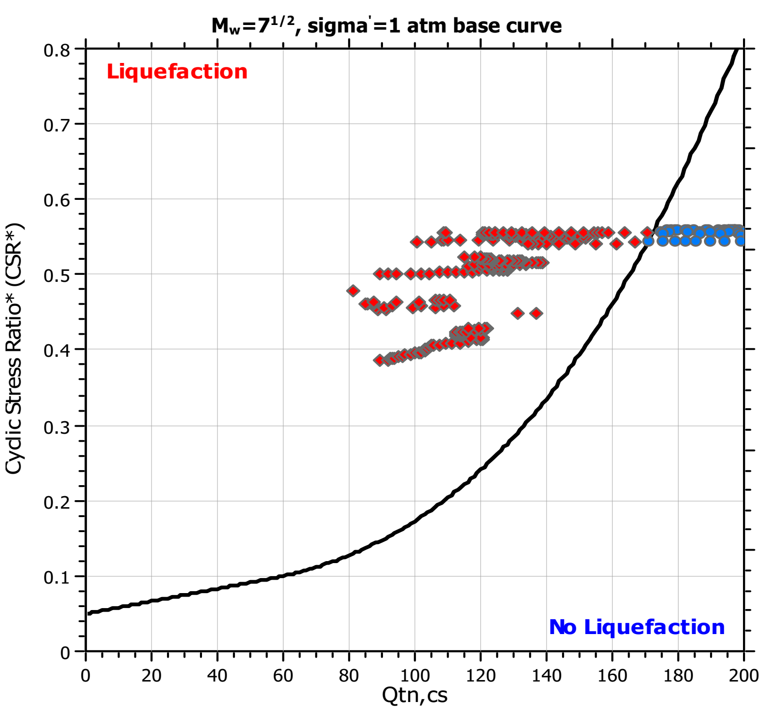
**Project title : 3027.00** **Location : Artesia, CA**

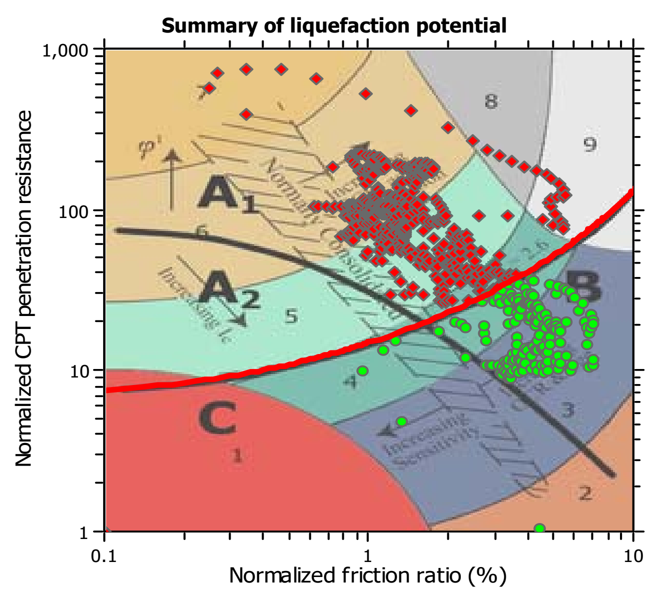
**CPT file : CPT-1**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | G.W.T. (in-situ): | 14.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | NCEER (1998) | G.W.T. (earthq.): | 10.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 6.83 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.73 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

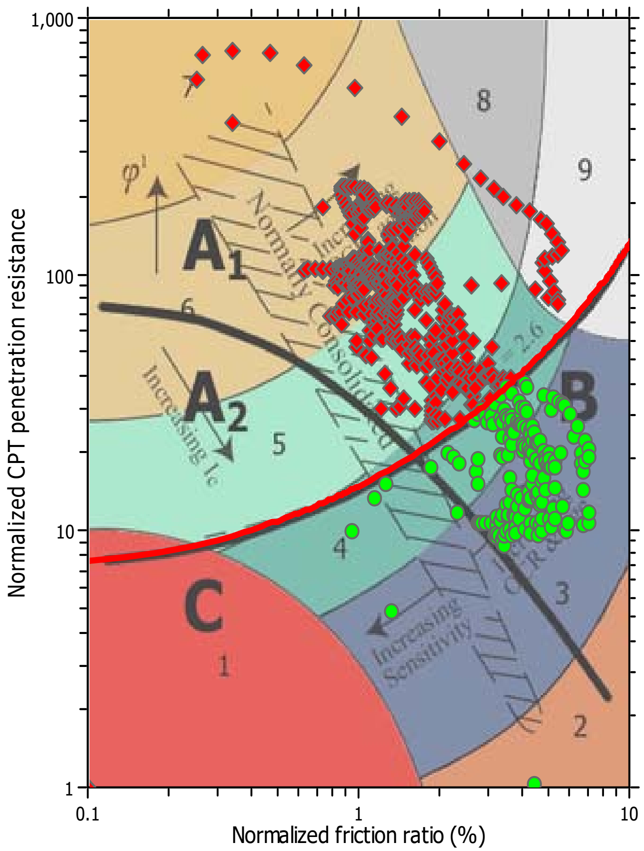


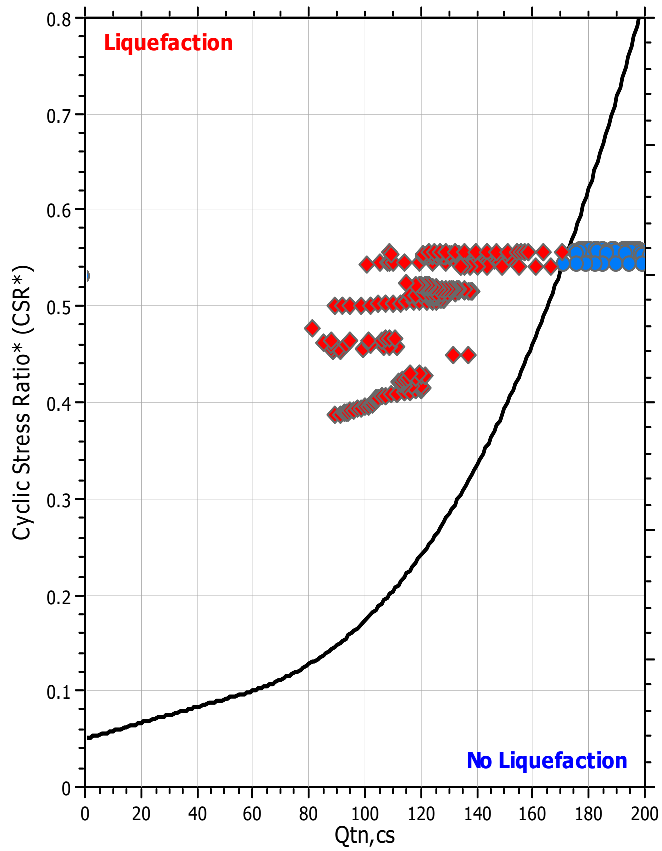



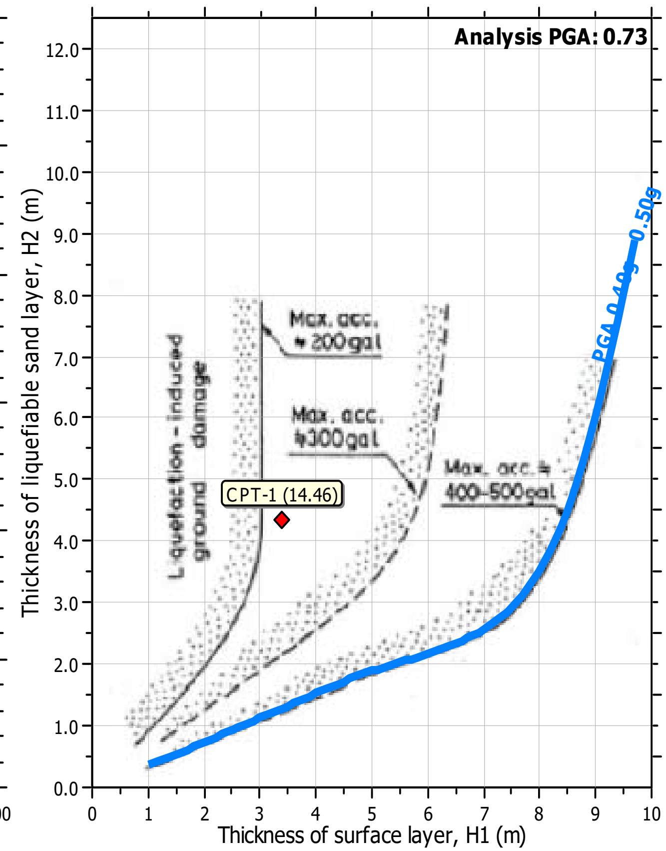




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction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

# Liquefaction analysis summary plots



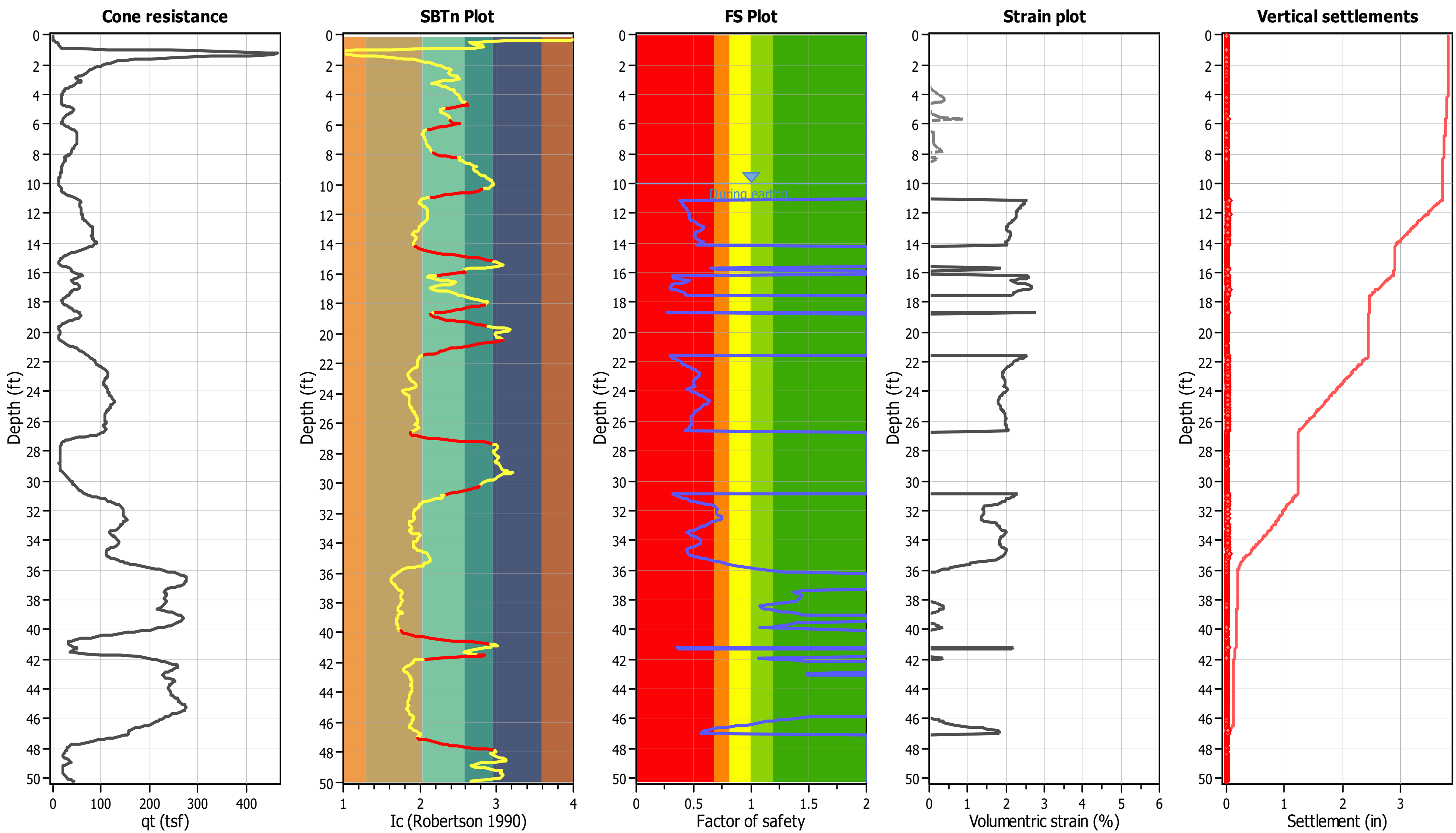




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | NCEER (1998) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_o$ applied: | Yes |
| Earthquake magnitude $M_w$: | 6.83 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.73 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 14.00 ft | Fill height: | N/A | Limit depth: | N/A |

# Estimation of post-earthquake settlements




**Abbreviations**

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

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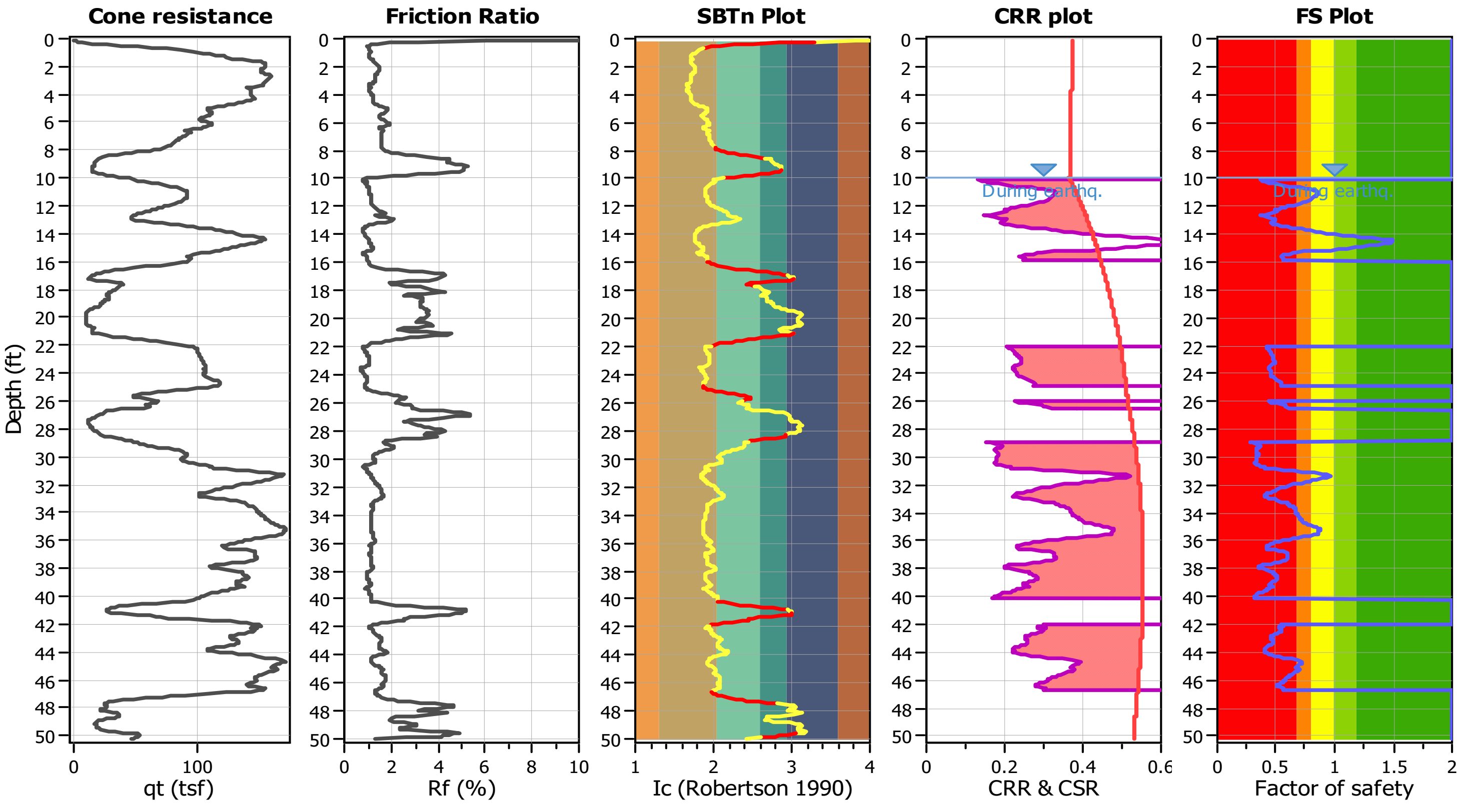
# LIQUEFACTION ANALYSIS REPORT

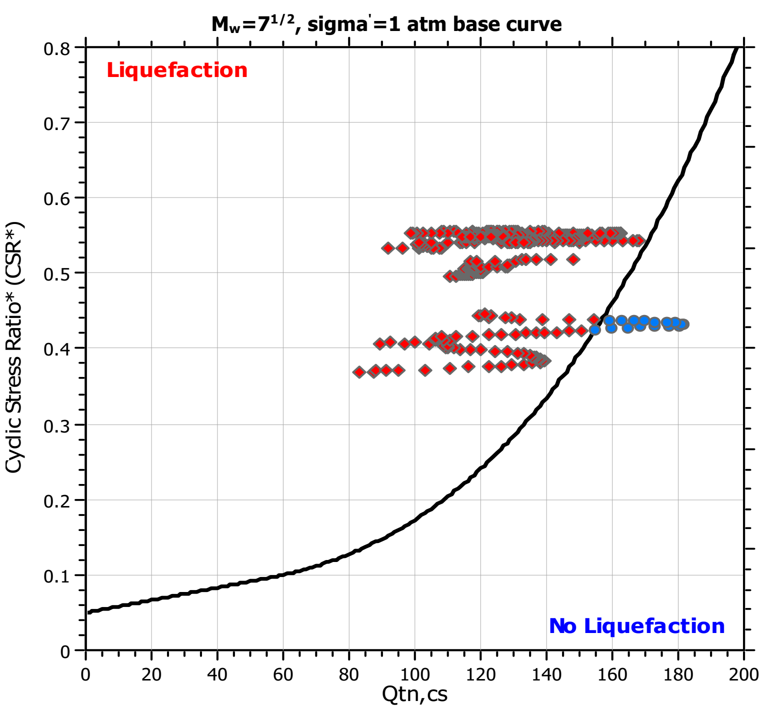
**Project title : 3027.00** **Location : Artesia, CA**

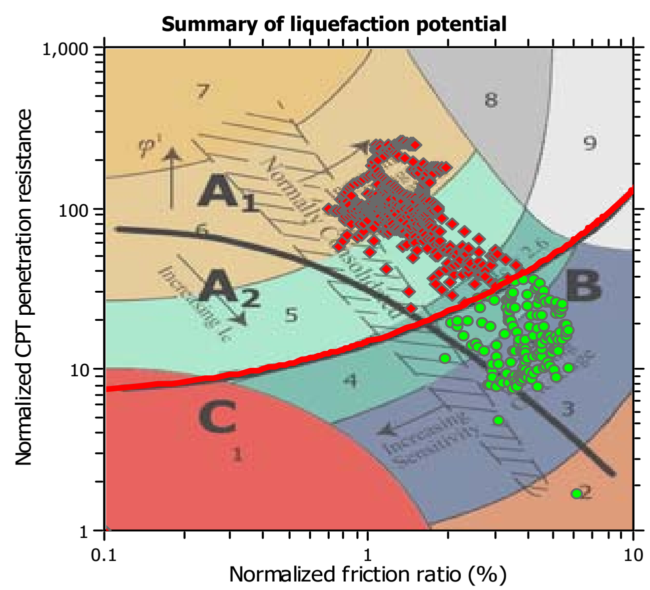
**CPT file : CPT-2**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | G.W.T. (in-situ): | 14.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | NCEER (1998) | G.W.T. (earthq.): | 10.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 6.83 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.73 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

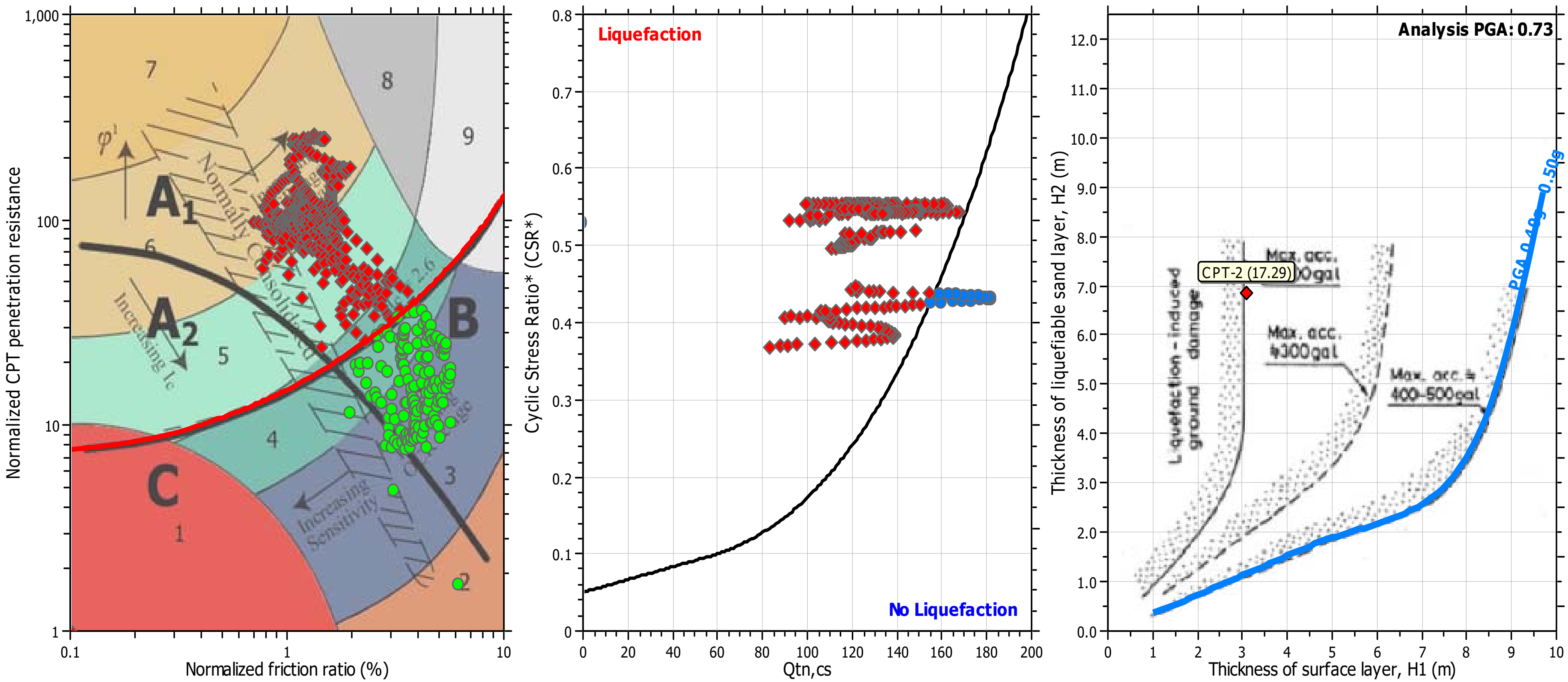



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction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

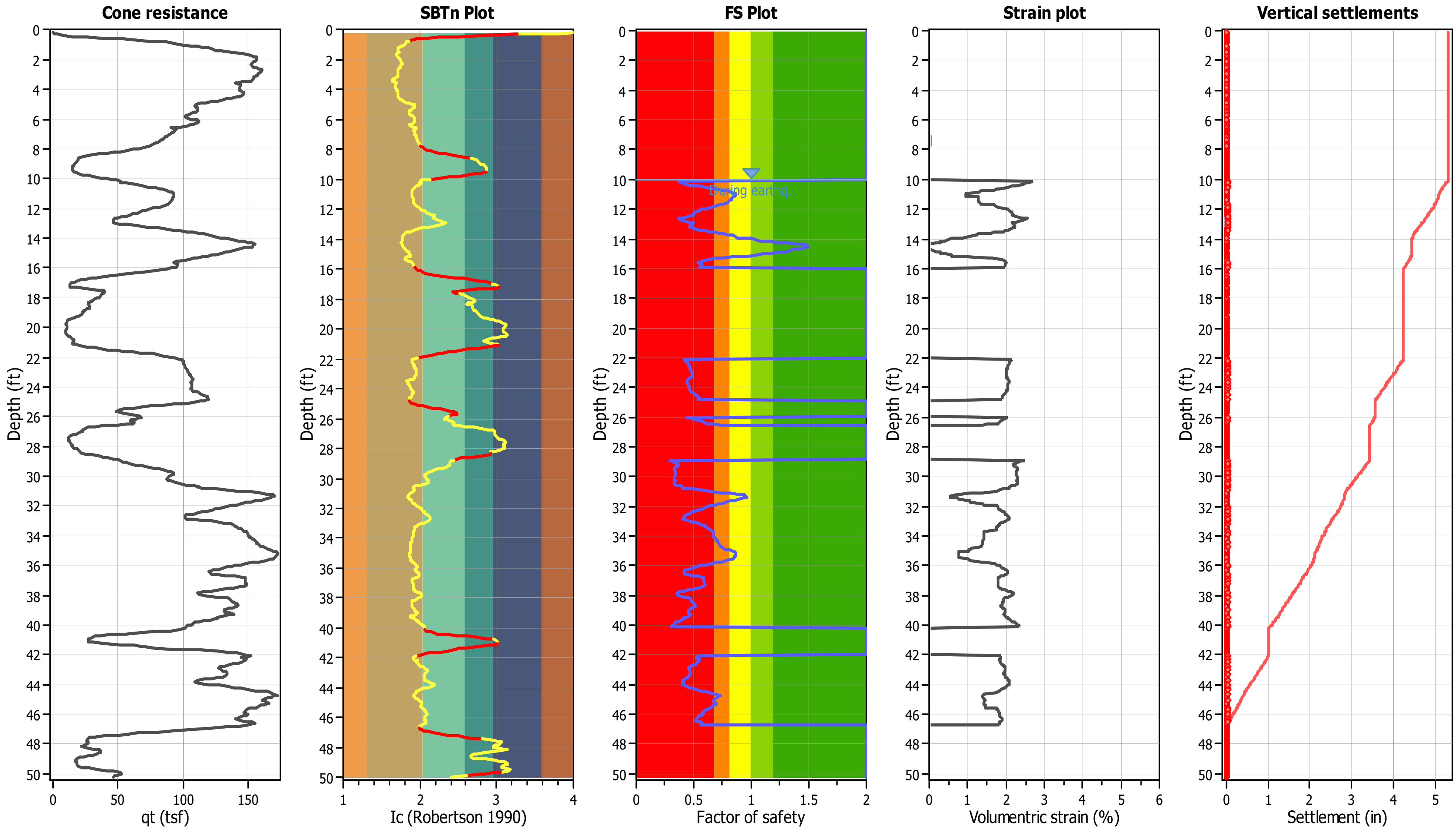
## Liquefaction analysis summary plots




### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | NCEER (1998) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 6.83 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.73 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 14.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Estimation of post-earthquake settlements




**Abbreviations**

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

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# LIQUEFACTION ANALYSIS REPORT

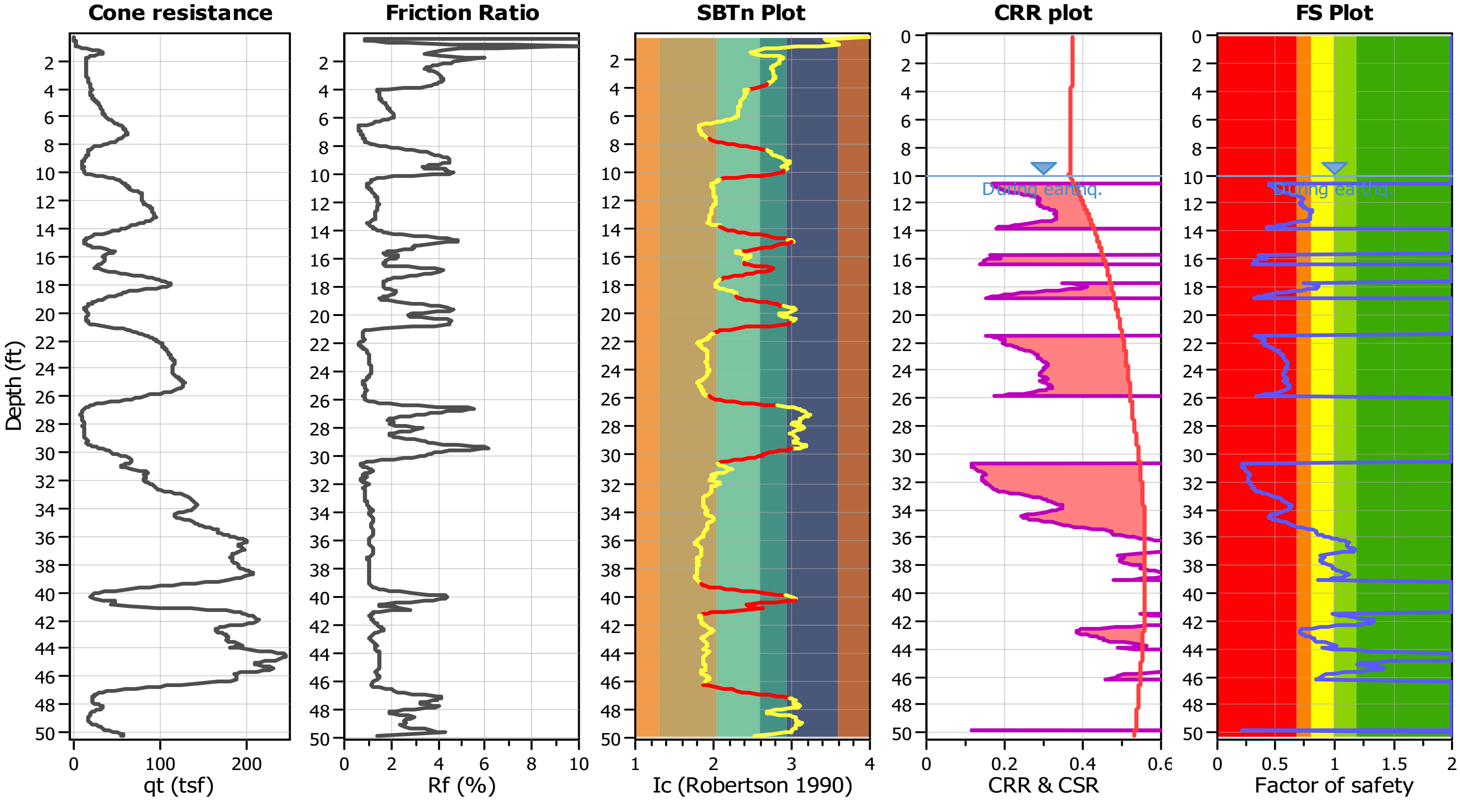
**Project title : 3027.00** **Location : Artesia, CA**

**CPT file : CPT-3**

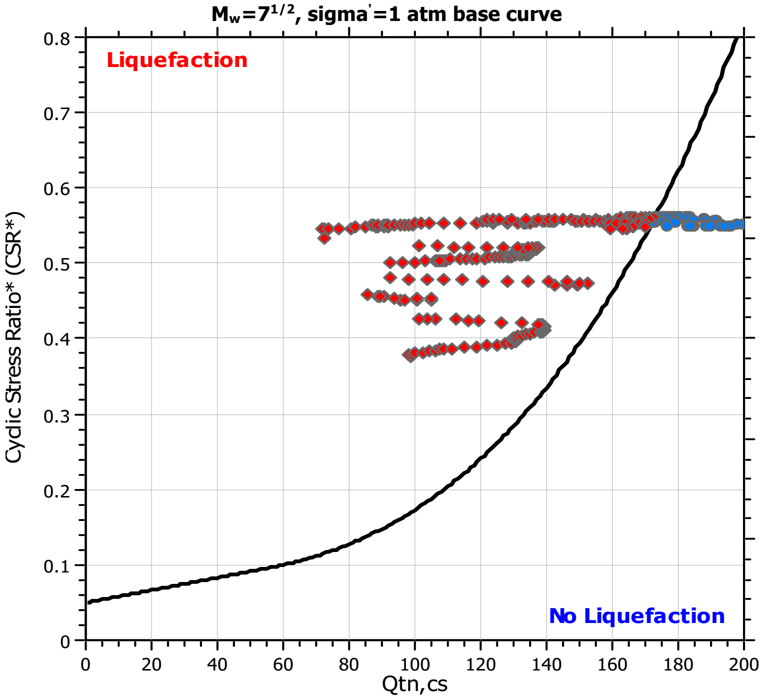
### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | G.W.T. (in-situ): | 14.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | NCEER (1998) | G.W.T. (earthq.): | 10.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 6.83 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.73 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

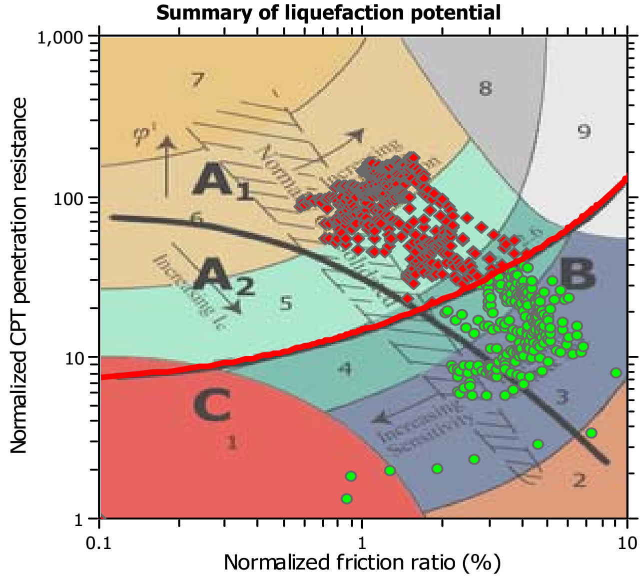
**Cone resistance** **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**




**$M_w=7^{1/2}$, sigma'=1 atm base curve**

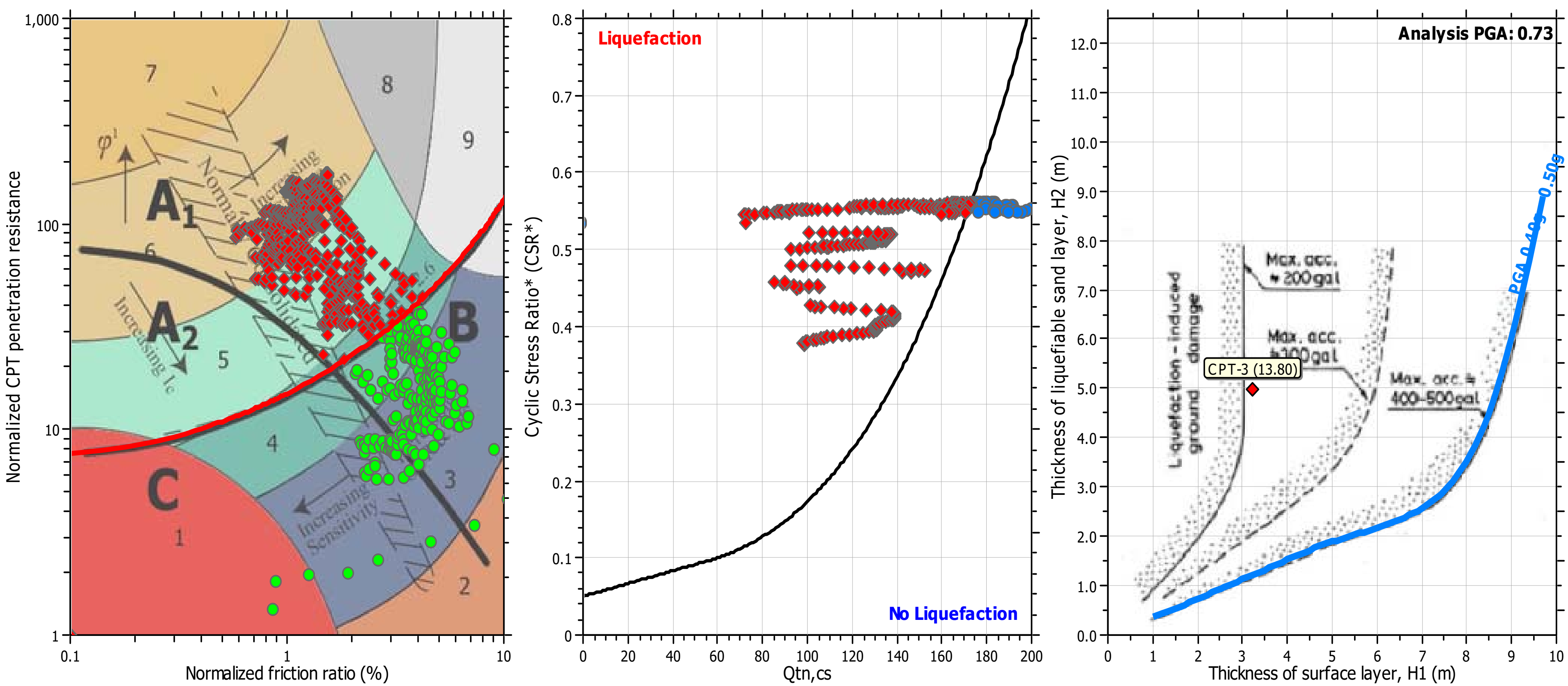



**Summary of liquefaction potential**




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

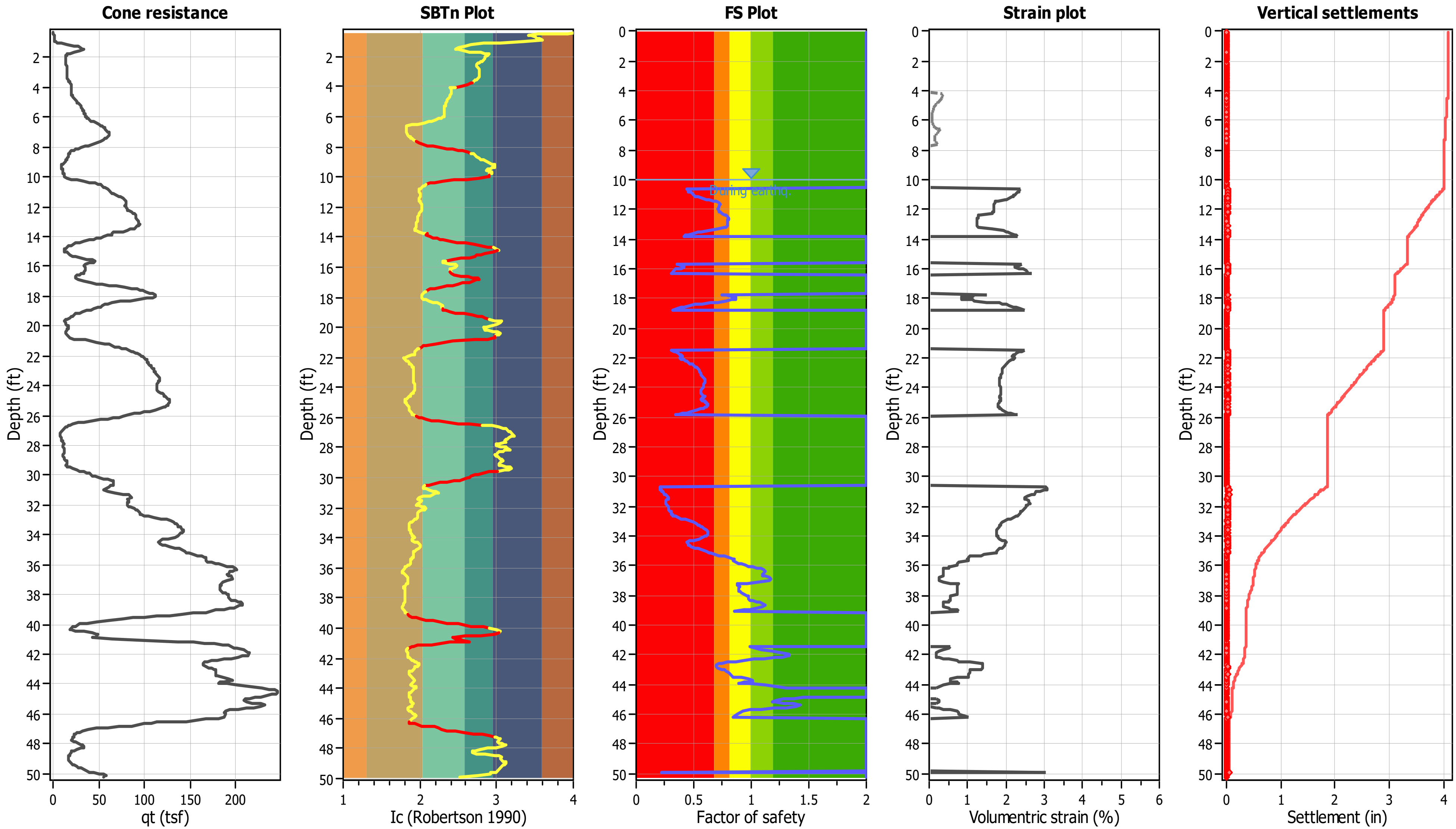
## Liquefaction analysis summary plots




### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | NCEER (1998) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 6.83 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.73 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 14.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Estimation of post-earthquake settlements




**Abbreviations**

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| $q_t$: | Total cone resistance (cone resistance $q_c$ corrected for pore water effects) |
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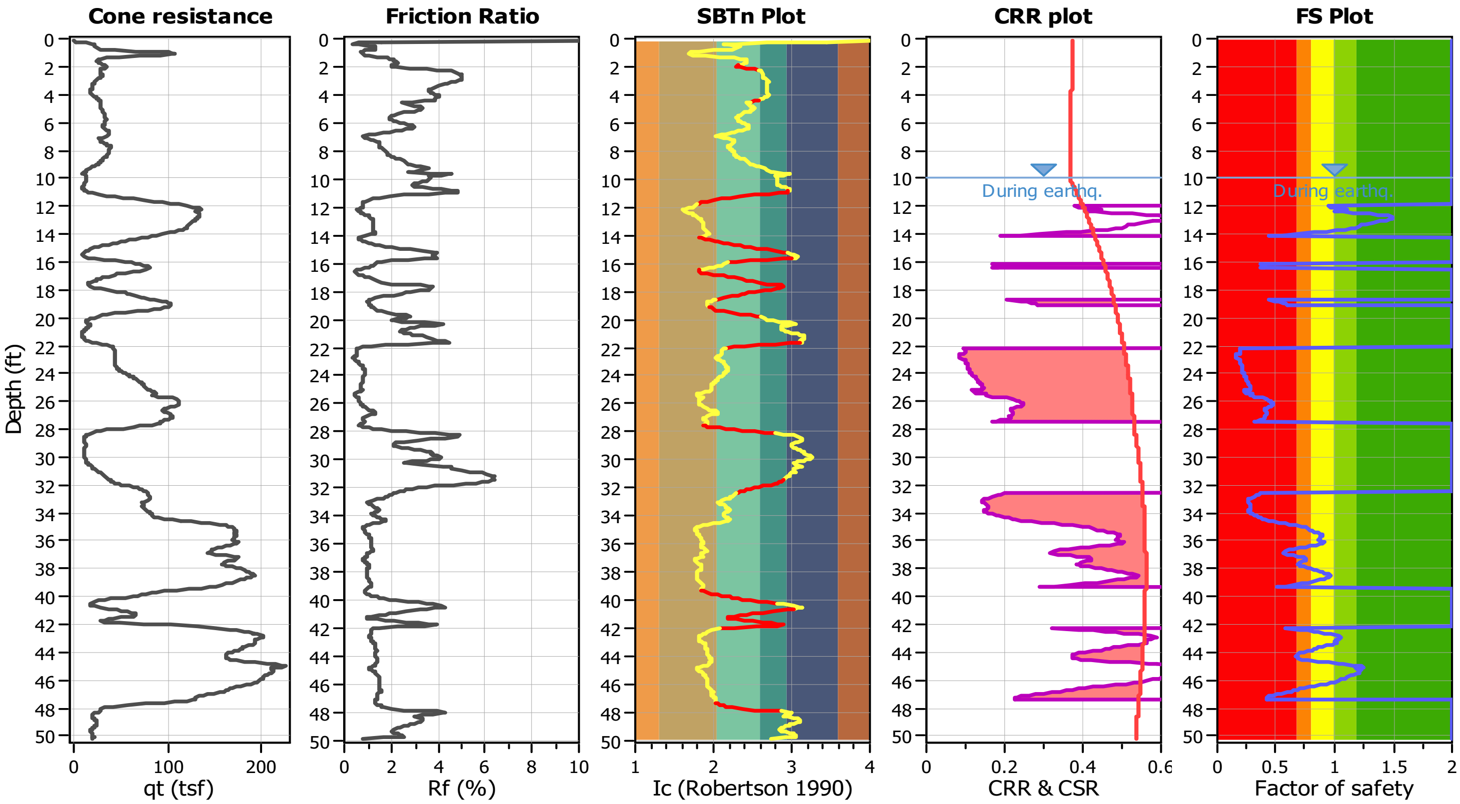
## LIQUEFACTION ANALYSIS REPORT

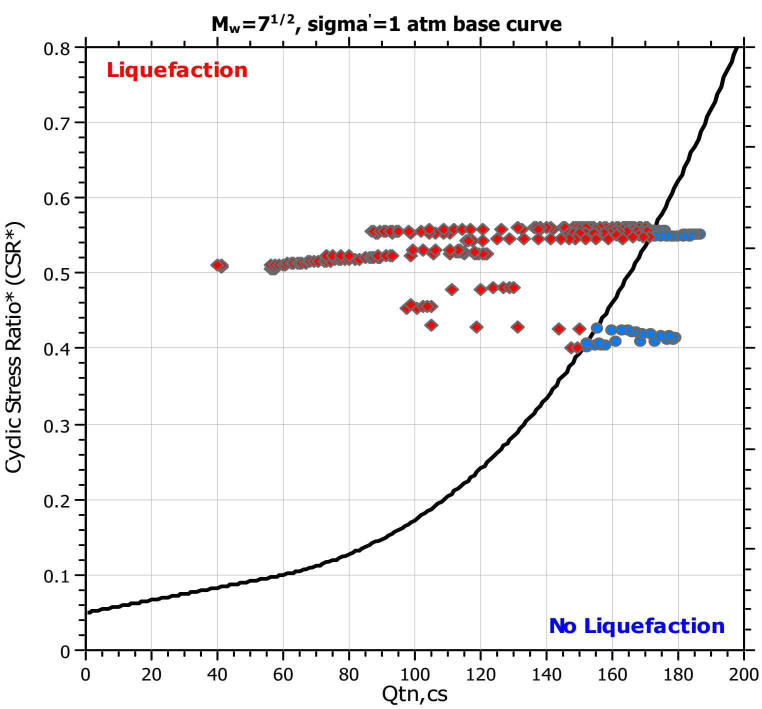
**Project title : 3027.00** **Location : Artesia, CA**

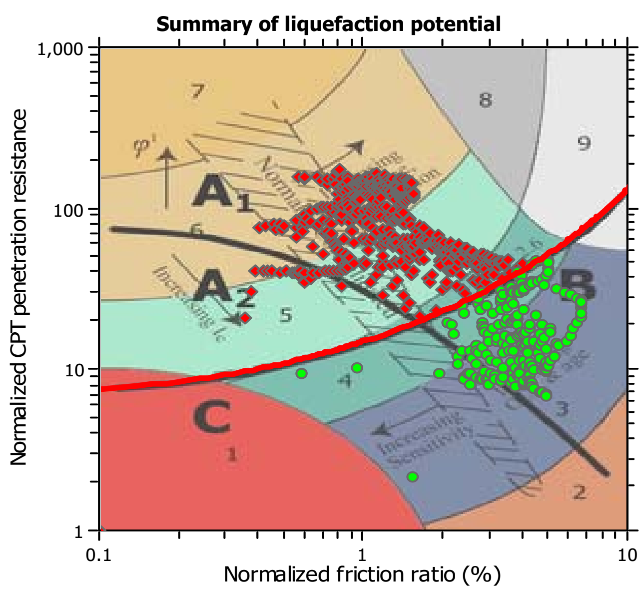
**CPT file : CPT-4**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | G.W.T. (in-situ): | 14.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | NCEER (1998) | G.W.T. (earthq.): | 10.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 6.83 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.73 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

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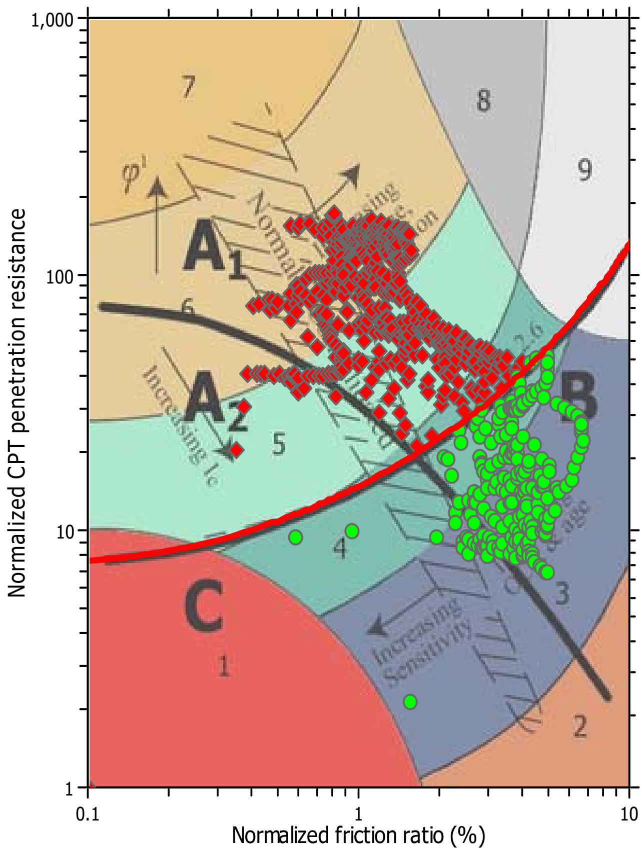
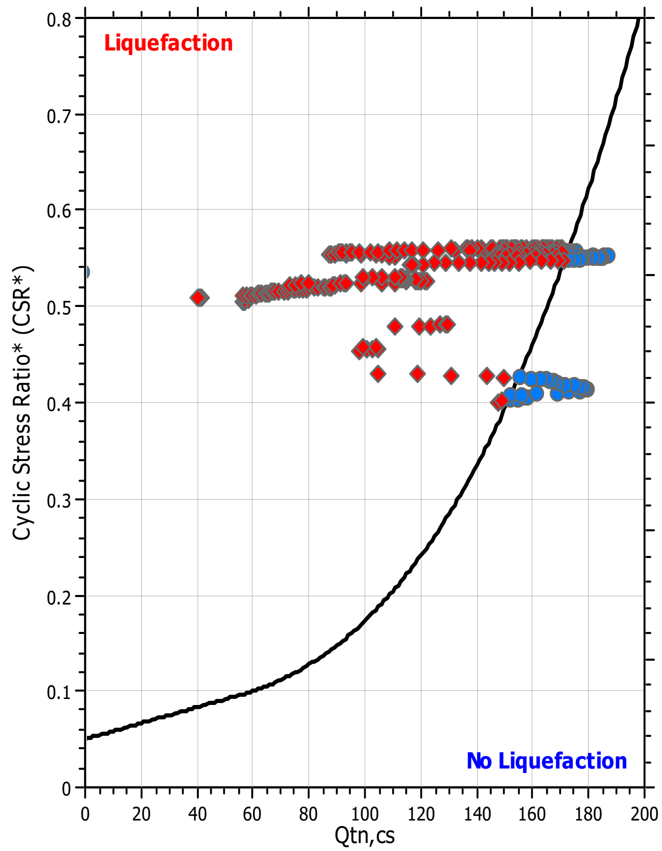
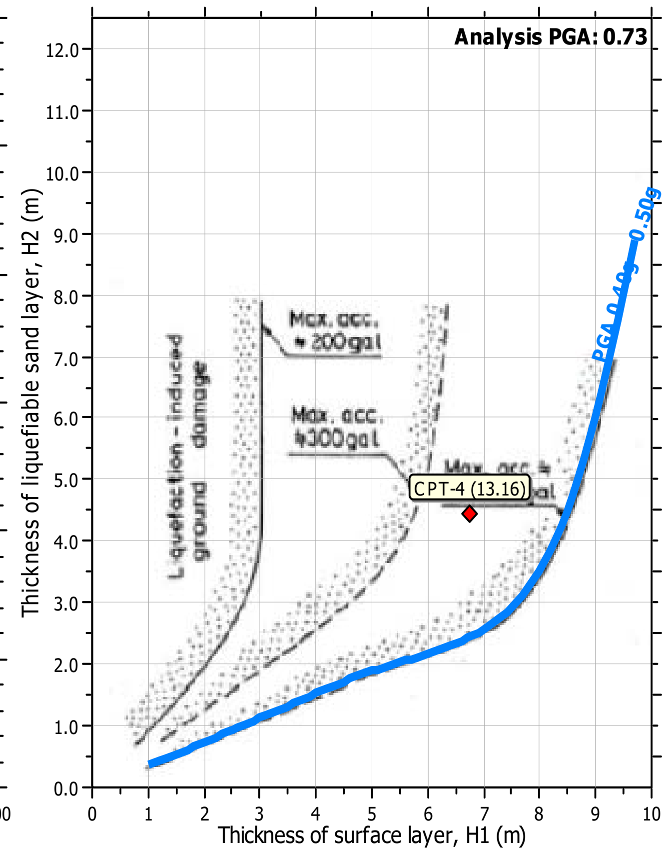
## Liquefaction analysis summary plots




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | NCEER (1998) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | NCEER (1998) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_{\sigma}$ applied: | Yes |
| Earthquake magnitude $M_w$: | 6.83 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.73 | Use fill: | No | Limit depth applied: | No |
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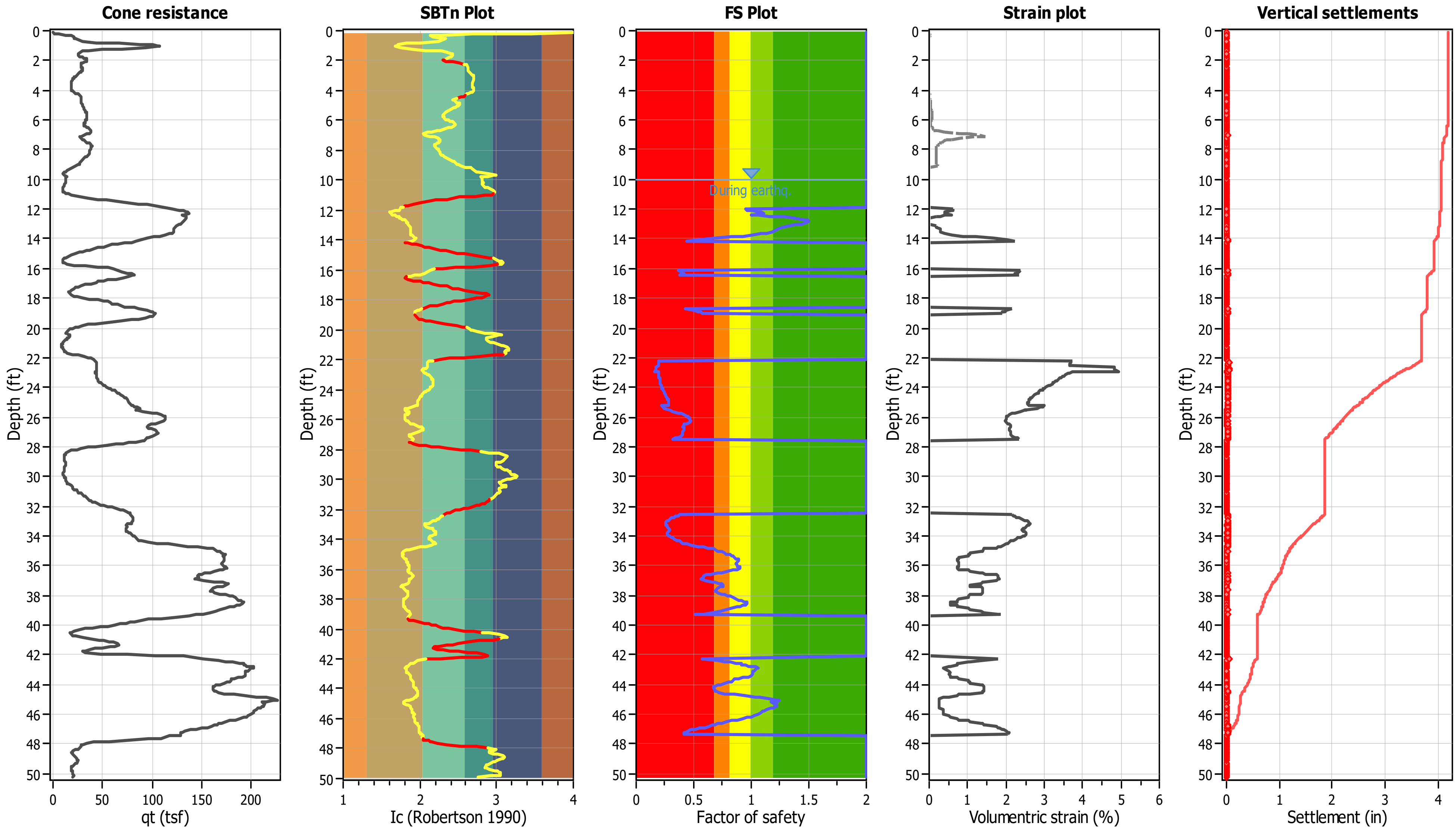
## Estimation of post-earthquake settlements




**Abbreviations**

| | |
|---|---|
| $q_t$: | Total cone resistance (cone resistance $q_c$ corrected for pore water effects) |
| $I_c$: | Soil Behaviour Type Index |
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