# Sherman Recovery Center

## Geotechnical Engineering Report

## Pleasant Hill, Contra Costa County, California

**Prepared for:**

Contra Costa County Public Works Department
40 Muir Road, 2nd Floor
Martinez, CA, 94553

1220 Concord Avenue, Suite 450
Concord, CA 94520
P 925-609-7224
**Terracon.com**

November 5, 2025

Contra Costa County Public Works Department
40 Muir Road, 2nd Floor
Martinez, CA, 94553

Attn: Gaile Suarez
P: (925) 839-3039
E: Gaile.Suarez@pw.cccounty.us

Re: Geotechnical Engineering Report
Sherman Recovery Center
2025 Sherman Drive
Pleasant Hill, Contra Costa County, California
Terracon Project No R1255060

Dear Ms. Suarez:

We have completed the scope of Geotechnical Engineering services for the referenced project in general accordance with Terracon Proposal No. PR1255060 dated July 16, 2025. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

**Terracon**

REGISTERED PROFESSSIONAL ENGINEER
ANTHONY ARGYRIOU
No. C56414
No. GE2687
Exp. 06/30/2027
CIVIL & GEOTECHNICAL
STATE OF CALIFORNIA

Christopher Kadi, P.G., C.E.G
Senior Geologist

Anthony Argyriou, P.E., G.E.
Senior Engineer

# Table of Contents

## Figures



## Attachments



**Note:** This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

# Report Summary

| Topic [1] | Overview Statement [2] |
|---|---|
| Project Description | The project includes a proposed single-story, wood-framed Social Rehabilitation Program Facility. Proposed site improvements include 14 parking spaces, a fire truck access road, and an outdoor recreational space.<br>Future plans include construction of 5 residential units in 3 residential buildings of approximately 800 square feet each, as noted in the site plan. |
| Geotechnical Characterization | Subgrade soils encountered in our borings and CPT generally consisted of interbedded Lean Clay and Silty Sand.<br>Groundwater was observed at an approximate depth of 17.6 feet bgs in our CPT during our exploration |
| Earthwork | Remove existing fill where observed (or) Geotechnical engineer to further evaluate areas where existing fill is present after site demolition and stripping.<br>Cuts and fills on the order of 2 feet or less are anticipated to develop final grades.<br>Existing lean clays can be used for engineered fill<br>Clays are sensitive to moisture variation<br>Minor excavation other than foundation construction and utility installation |
| Shallow Foundations | Shallow foundations are recommended for building support<br>Allowable bearing pressure = 2,000 psf<br>Expected settlements: < 1-inch total, < ½-inch differential |
| Pavements | Pavement sections are provided for both rigid and flexible pavements, with subgrade prepared as noted in Earthwork. |
| General Comments | This section contains important information about the limitations of this geotechnical engineering report. |

1. If the reader is reviewing this report as a pdf, the topics in the table can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

| Item | Description |
| --- | --- |
| Information Provided | An email request for proposal was sent by Gaile Suarez on July 8th, 2025. The email included an ALTA Survey, As-Built Drawings, Conceptual Design file, and a Phase 1 Environmental Site Assessment report for the project site.<br>An updated Conceptual Design and facility layout was provided on September 30, 2025. |
| Project Description | The project includes a proposed 16-bed single-story Social Rehabilitation Program Facility. Proposed site improvements include 14 parking spaces, a fire truck access road, and an outdoor recreational space.<br>Future plans include construction of a total of 5 residential units in 3 residential buildings of approximately 800 square feet each along the northern edge of the property; the RFP asked that the geotechnical report cover recommendations for the planned future construction. |
| Proposed Structures | Structures associated with the project include an 8,663 square foot wood-frame building supported on a concrete slab-on-grade foundation, and 3 wood-frame on slab floor buildings approximately 800 square feet each. |
| Building Construction | We anticipate the buildings will be wood-frame structures with concrete slab-on-grade floors. |
| Finished Floor Elevation | The anticipated finished floor elevations were not provided; boring/CPT depths have assumed the finished floor is not more than 2 feet below/above existing grade. |
| Maximum Loads | Anticipated structural loads were not provided. In the absence of information provided by the design team, we will use the following loads in estimating settlement based on our experience with similar projects.<br>■ Columns: 25 to 50 kips<br>■ Walls: up to 2 kips per linear foot (klf)<br>■ Slabs: 200 pounds per square foot (psf) |
| Grading/Slopes | A preliminary grading plan was not available for review at the time this proposal was prepared. Proposed finished grade elevations for the building pad and pavements are expected to be within about 2 feet of existing grades. |
| Below-Grade Structures | None anticipated |
| Free-Standing Retaining Walls | None anticipated |

# Introduction

This report presents the results of our subsurface exploration and Geotechnical Engineering services performed for the proposed rehabilitation center to be located at 2025 Sherman Drive in Pleasant Hill, Contra Costa County, California. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Seismic site classification per the 2025 California Building Code (CBC)
- Site preparation and earthwork
- Demolition considerations
- Dewatering considerations
- Foundation design and construction
- Floor slab design and construction
- Lateral earth pressure
- Pavement design and construction
- Stormwater detention considerations
- Liquefaction potential

The geotechnical engineering Scope of Services for this project included the advancement of test borings and CPTs, laboratory testing, engineering analysis, and preparation of this report.

Drawings showing the site and boring and CPT locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section.

# Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

- The original layout was changed; we received a revised layout before our field exploration.

| Item | Description |
|---|---|
| Information Provided | An email request for proposal was sent by Gaile Suarez on July 8th, 2025. The email included an ALTA Survey, As-Built Drawings, Conceptual Design file, and a Phase 1 Environmental Site Assessment report for the project site.<br>An updated Conceptual Design and facility layout was provided on September 30, 2025. |
| Project Description | The project includes a proposed 16-bed single-story Social Rehabilitation Program Facility. Proposed site improvements include 14 parking spaces, a fire truck access road, and an outdoor recreational space.<br>Future plans include construction of a total of 5 residential units in 3 residential buildings of approximately 800 square feet each along the northern edge of the property; the RFP asked that the geotechnical report cover recommendations for the planned future construction. |
| Proposed Structures | Structures associated with the project include an 8,663 square foot wood-frame building supported on a concrete slab-on-grade foundation, and 3 wood-frame on slab floor buildings approximately 800 square feet each. |
| Building Construction | We anticipate the buildings will be wood-frame structures with concrete slab-on-grade floors. |
| Finished Floor Elevation | The anticipated finished floor elevations were not provided; boring/CPT depths have assumed the finished floor is not more than 2 feet below/above existing grade. |
| Maximum Loads | Anticipated structural loads were not provided. In the absence of information provided by the design team, we will use the following loads in estimating settlement based on our experience with similar projects.<br>■ Columns: 25 to 50 kips<br>■ Walls: up to 2 kips per linear foot (klf)<br>■ Slabs: 200 pounds per square foot (psf) |
| Grading/Slopes | A preliminary grading plan was not available for review at the time this proposal was prepared. Proposed finished grade elevations for the building pad and pavements are expected to be within about 2 feet of existing grades. |
| Below-Grade Structures | None anticipated |
| Free-Standing Retaining Walls | None anticipated |

| Item | Description |
|---|---|
| **Pavements** | A paved driveway and parking area will be constructed on approximately 0.2 acres of the parcel.<br>A preferred pavement surfacing was not identified as part of the provided information. Asphalt surfacing is common in the area for projects of this nature and is the assumed preference.<br>Unless information is provided prior to the report, we assume the following traffic indices (TIs) will be used:<br>■ Auto Parking Areas: TI = 5.0:<br>■ Auto Road: TI = 5.5<br>■ Truck Parking Areas: TI = 6.0<br>■ Fire Truck Access Road: TI = 8.0<br>The pavement design period is 20 years. |
| **Building Code** | 2025 California Building Code (CBC) |

Terracon should be notified if any of this information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

# Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

| Item | Description |
|---|---|
| **Parcel Information** | The project is located at 2025 Sherman Drive in Pleasant Hill, Contra Costa County, California.<br>APN: 127-170-027, recorded as 0.96 acres.<br>Latitude/Longitude (approximate): 37.9523° N, 122.0542° W<br>See **Site Location** |
| **Existing Improvements** | An existing single-story county facility with associated parking lot and recreational sports area located on the north side of the site. |
| **Current Ground Cover** | Grasses cover the northern and western sides of the site. Mature trees are located near the southern and western site perimeter. There is an asphalt-paved parking area in the southeast area of the site, and an asphalt-paved driveway along the southern perimeter. |

| Item | Description |
|---|---|
| **Existing Topography** | The project site has an approximate elevation of +55 feet (NAVD88)[1]. Actual elevations can be referenced if a site-specific topo map is provided. Boring and CPT depths have been estimated in part with this information and improved topographic information should be provided if available. |

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

# Geotechnical Characterization

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting, and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in **Exploration and Laboratory Results** and the GeoModel can be found in the **Figures** attachment of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

| Model Layer | Layer Name | General Description |
|---|---|---|
| 1 | **Lean Clay** | Medium Plasticity Lean Clay with Fine Sand |
| 2 | **Silty Sand** | Silty/Clayey Fine-Grained Sand |
| 3 | **Lean Clay** | Lean Clay with trace Sand |
| 4 | **Fat Clay** | High Plasticity Fat Clay in Boring B1 |

[1] United States Geological Survey (USGS) National Map 3D Elevation Program (3DEP) , "*3DEP Demonstration Elevation Viewer*" May 16, 2023 **https://apps.nationalmap.gov/3depdem/** .

## Groundwater Conditions

The borings were advanced in the dry using an auger drilling technique that allow short term groundwater observations to be made while drilling. The boreholes was/were observed while drilling and after completion for the presence and level of groundwater. A pore pressure dissipation test was also performed in the CPT to help determine groundwater levels.

Groundwater and/or seepage were not encountered within the maximum depths of the borings/CPT at the time of our field exploration, or for the short duration the borings/CPT could remain open. The pore pressure dissipation testes indicated estimated groundwater depths of 17.6 and 19.3 feet bgs.

| Boring/CPT Number | Approximate Depth to Groundwater while Testing[1] (feet) | Approximate Depth to Groundwater immediately after Drilling[1] (feet) |
|---|---|---|
| CPT1 (52′) | 17.6 | N/A |
| CPT1 (67′) | 19.3 | N/A |

1. Below ground surface.

Due to the low permeability of soils encountered in the borings, a relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole. Long term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type. Long-term groundwater monitoring was outside the scope of services for this project. Terracon is experienced in installing groundwater monitoring wells/piezometers to provide more groundwater data prior to construction if required.

Groundwater conditions may be different at the time of construction. Mapping by the Seismic Hazard Zone Report 136 indicates the historical high groundwater level at the site is about 10 feet below ground surface.

Groundwater conditions may change because of seasonal variations in rainfall, runoff, and other conditions not apparent at the time the borings and CPT were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring and CPT logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project. We recommend a groundwater level of 10 feet bgs be used in design.

# Geologic Hazards

Geologic maps indicate subsurface conditions at the site consist of Holocene age alluvial gravel, sand and clay[1]. Below the pavement section associated with the parking area, the subgrade soils encountered in our borings and CPT were generally consistent with mapped geology.

## Faulting and Estimated Ground Motions

The site is located in the San Francisco Bay area of California, which is a relatively high seismicity region. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, the intensity, and the magnitude of the seismic event. The following table indicates the distance of the fault zones and the associated maximum credible earthquake that can be produced by nearby seismic events, as calculated using the USGS Earthquake Hazard Toolbox. Segments of the Mount Diablo Fault, which is located approximately 5.5 kilometers from the site, are considered to have the most significant effect at the site from a design standpoint.

| Fault Name | Approximate Contribution (%) | Approximate Distance to Site (kilometers) | Maximum Considered Earthquake (MCE) Magnitude |
|---|---|---|---|
| Mount Diablo (north) (1) | 13.24 | 5.50 | 7.25 |
| Hayward (north) (1) | 10.43 | 19.04 | 7.31 |

Based on the ASCE 7-22 Standard, the peak ground acceleration ($PGA_M$) at the subject site is approximately 0.81g. Based on the USGS 2018/2023 interactive disaggregations, the PGA at the subject site for a 2% probability of exceedance in 50 years (return period of 2475 years) is expected to be about 0.893g. The site is not located within an Alquist-Priolo Earthquake Fault Zone based on our review of the State Fault Hazard Maps.[2]

[1] Dibblee Geological Foundation, *"Geologic map of the Walnut Creek quadrangle, Contra Costa County, California"*, T.W. Dibblee and J.A. Minch, 2005

[2] California Geological Survey (CGS), *"California Earthquakes Hazards Zone Application (EQ Zapp)",* 2024 **https://maps.conservation.ca.gov/cgs/informationwarehouse/eqzapp/**

## Liquefaction

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils or low plasticity fine grained soils exist below groundwater. The California Geological Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is located within a mapped CGS liquefaction hazard zone. Therefore, a liquefaction evaluation was performed to estimate the potential for liquefaction induced settlement.

# Seismic Considerations

The 2025 California Building Code (CBC) Seismic Design Parameters have been generated using the ASCE Hazard Tool. This web-based software application calculates seismic design parameters in accordance with ASCE 7-22.

| Description | Value |
|---|---|
| 2025 California Building Code (CBC) Site Class[1] | F[1] |
| Risk Category | II |
| Site Latitude[2] | 37.9523º |
| Site Longitude[2] | -122.0543º |
| Mapped Spectral Acceleration Parameters[3] | $S_s$ = 2.59 and $S_1$ = 0.93 |
| Spectral Response Acceleration Parameters[3] | $S_{MS}$ = 2.31 and $S_{M1}$ = 1.88 |
| Design Spectral Acceleration Parameters[3] | $S_{DS}$ = 1.54 and $S_{D1}$ = 1.26 |

1. This site qualifies as a site class F due to the presence of liquefiable soils. A site class D was used to develop the listed seismic design parameters due to the shear wave velocity measurements obtained from the CPTs. Based on the exception for liquefiable soils provided in ASCE 7-22 Section 20.2.1, structures may use the listed design parameters provided they have a period of 0.5 seconds or less. *This exception applies only to seismic parameters; other requirements of Site Class F continue to apply.* Should the anticipated structure have a period greater than 0.5 seconds, a site-specific ground motion analysis is required to develop seismic design parameters. Terracon is qualified to perform such an analysis.

| Description | Value |
|---|---|

2. Provided coordinates represent a point located at the general center of the site.
3. These values were obtained using online seismic design maps and tools provided by ASCE (**https://ascehazardtool.org/**).

Typically, a site-specific ground motion study may reduce construction costs. We recommend consulting with a structural engineer to evaluate the need for such a study and its potential impact on construction costs. Terracon should be contacted if a site-specific ground motion study is desired.

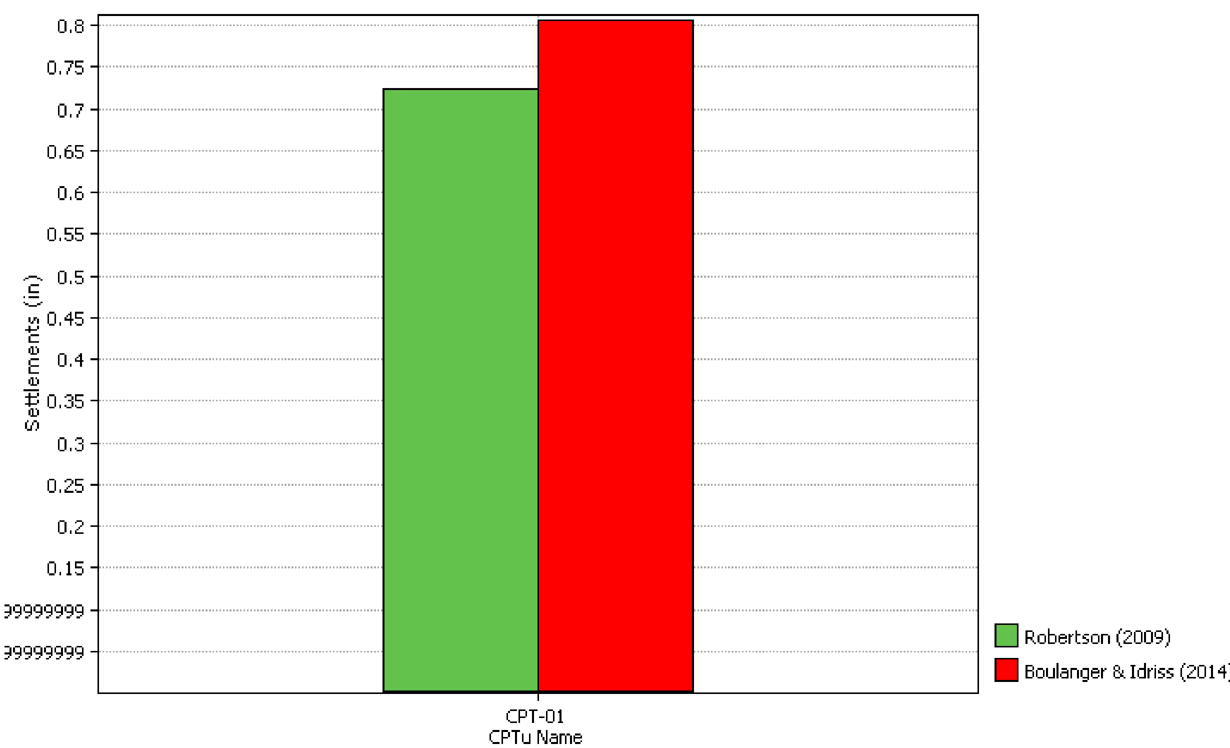
# Liquefaction Evaluation

We performed a liquefaction hazard evaluation in general compliance with the California Geological Survey (CGS) Special Publication 117A (2008); Southern California Earthquake Center "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," 1999 report; and the Seismic Hazard Zone Report for the Walnut Creek 7.5-Minute Quadrangle, Contra Costa County, California (SHZR 136).

As recommended in these reports, we performed a screening analysis to determine if there is potential for liquefaction to occur at the site. We evaluated the soils encountered in our borings advanced to a maximum depth of 26.5 feet below the existing ground surface (bgs) and cone penetration tests (CPTs) advanced to a maximum depth of 100 feet bgs. We evaluated these soils based on soil classification, corrected SPT blow counts, water content, Atterberg limits, groundwater elevation, shear strength, peak ground acceleration, and CPT data. In our screening investigation we looked at the Atterberg limits for cohesive soils in our soil borings. The Atterberg limits for these cohesive soils exhibited a liquid limit ranging from non-plastic to 31 and a plasticity index ranging from non-plastic to 17. We also calculated the ratio of the in-situ moisture content to the liquid limit. This data was then compared to the criteria by Idriss and Boulanger (2006) and Bray and Sancio (2006) for potential liquefaction or cyclic softening of fine-grained soils.

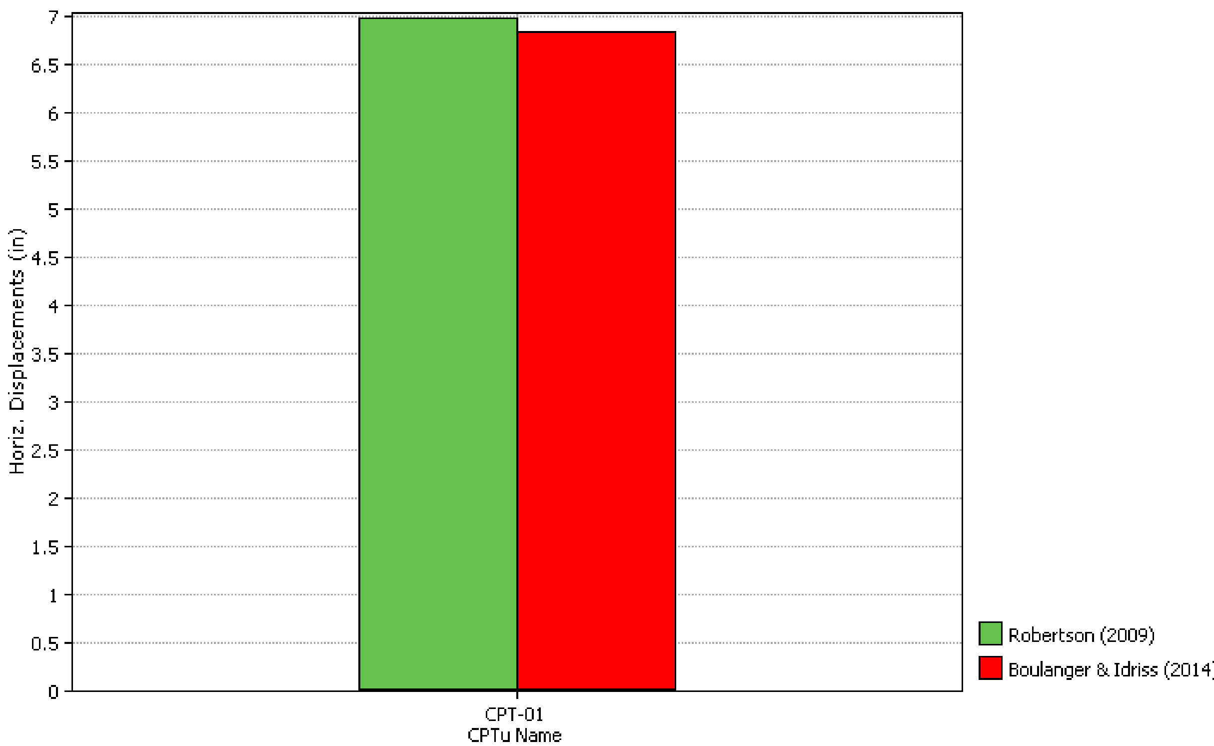
We reviewed the shear wave velocities measured during advancement of our CPTs. The shear wave velocities measured in the upper 60 feet of the CPTs varied from 410 feet per second (ft/s) to 1,057 ft/s. Based research by Andrus et al. (2009), it has been accepted that soils with shear wave velocities greater than 720 ft/s are generally not susceptible to liquefaction. Based on this criterion, the soils encountered in CPT-01 would not be susceptible to liquefaction below 47 feet bgs.A Peak Ground Acceleration (PGA) of 0.810g and an earthquake magnitude of 6.67 for the project site was used in our evaluation. The shallowest groundwater depth measured in our investigation was 17.6 feet; Seismic Hazard Zone Report 136 for the Walnut Creek Quadrangle indicates the historical high groundwater depth for this site is about 10 feet bgs. As a result, a groundwater depth of 10 feet was utilized in our evaluation.

The liquefaction study utilized the software "CLiq" by GeoLogismiki Geotechnical Software. Our analysis was based on the soil data obtained from the CPT soundings supplemented by laboratory data performed on samples obtained from our borings. Analyses were performed on data obtained from CPT-01. CPT calculations were assessed using the Robertson (2009) and Boulanger & Idriss (2014) methods. A factor of safety of 1.3 was used against liquefaction. A Soil Behavior Type Index, $I_c$, cutoff value of 2.60 was used in our analysis. The $I_c$ value is an index value that helps identify soil behaviors based on data obtained during cone penetration testing. The liquefaction potential analysis was calculated from a depth of 0 to 60 feet bgs. A summary of the results of our analysis using the Robertson (2009) method has been attached to this report. The following chart summarizes the vertical settlements calculated using the noted methods.




In order to help protect the proposed building against the effects of liquefaction, the proposed building could be supported on **Shallow Foundations** over soils mitigated by **Ground Improvement**.

With regards to the potential for lateral spreading, we note that the site and surrounding area is relatively level. However, free faces are present as close as 185 feet to the east along Walnut Creek. Based on the proximity of the site to the creek and the site elevations between the area of development and the bottom of the canal, the probability of liquefaction induced lateral spread to impact this site is considered high. Based on our analysis, we estimate approximately 7 inches of lateral spread could occur at the site.




In order to help protect the proposed improvements against the potential for excessive settlement, bearing capacity, and lateral spread due to liquefaction, we recommend the proposed improvements be supported by **Shallow Foundations** or **Mat Foundations** over subgrade soils mitigated by **Ground Improvement**.

# Percolation/Infiltration

We performed 1 percolation test within the proposed site development for use by the project civil engineer in the design of the storm water retention system. The percolation test was performed using boring P1 drilled to a depth of about 5 feet bgs. The approximate location of the test hole is shown on the **Exploration Plan**.

After drilling the test hole, we placed approximately 2 inches of gravel in the bottom of the hole, then placed a slotted PVC pipe in the hole, and filled the annular space around the pipe with gravel. The test hole was filled with water and left to saturate for a minimum 24 hours. We then filled the shallow hole with water to a depth of approximately 4.9 feet and measured the drop-in water surface over a period of approximately 4 hours, refilling the hole as necessary to maintain the desired head.

The measured percolation rates and calculated infiltration rates are summarized in the following table.

| Perc. Test Location | Depth (ft) | Avg. Head (ft) | Perc. Rate (min/inch) | Perc. Rate (inch/hr) | Infiltration Rate (inch/hr) |
|---|---|---|---|---|---|
| P1 | 5.0 | 4.93 | 335 | 0.18 | 0.003 |

Since we used a test boring to perform percolation testing, we have used the Porchet formula (aka Inverse Borehole Formula) to calculate the infiltration rate which takes into account sidewall area of the bore hole. Storm water runoff may likely contain materials such as silt, leaves, oil residues, and other matter that may reduce the percolation characteristics of the soil. We therefore recommend that a filtration system be implemented into the design and installed. An appropriate safety factor should be applied to the measured infiltration rates by the designer for use in design and be based on the amount of filtration designed into the system, at a minimum a Safety Factor of 2 shall be utilized. The values presented in the table are clear water rates and do not have a safety factor applied. In addition, we recommend a regular maintenance program be implemented to monitor the storm drainage/filtration system prior to the beginning of each wet weather season.

We have provided the following considerations for the design and construction of the retention/detention facilities. Planned retention/detention facilities should be located no closer than 10 feet to structural site improvements.

The long-term infiltration rates will depend on many factors, and can vary or be reduced if the following conditions are present:

- Fill placement,
- Variability of site soils,
- Fine layering of soils, or
- Maintenance and pre-treatment (filtration) of the influent are not performed regularly

<u>Fill Placement:</u> We anticipate earthwork required to develop the site may consist of cuts and fills of 2 feet or less. It is unknown whether final grades will consist of native material or imported fill. As a result, the percolation tests performed may not be representative of the final soil conditions depending on the blend of soils utilized as structural fill and native soils exposed where cuts and fills are made. Additional percolation testing may be warranted following rough grading to confirm the values utilized in design are appropriate.

Subsurface Soil Variation: Variations in subsurface soil conditions and the presence of fine layering can affect the infiltration rate of the receptor soils. Due to variation in thickness of the upper surface fine grained soils, infiltration rates may vary across the site.

Construction Considerations: The infiltration rate of the receptor soils will be reduced in the event that fine sediment, organic materials, and/or oil residue are allowed to accumulate in the retention facilities. The use of a filtration system is highly recommended as well as a maintenance program.

Operation of heavy equipment during construction may densify the receptor soils below the infiltration facility. The soils exposed in the bottom of the infiltration facility should not be compacted and should remain in their native condition. This may require scarification of the soils prior to construction.

Maintenance of Facilities: Satisfactory long-term performance of an infiltration facility will require some degree of maintenance. Accumulations of sediment, organic materials, or other material that serve to reduce their permeability of the receptor soils should be removed from the filtration system on a regular basis so as not to enter the retention system. The filtration system shall have a rigorous maintenance program, debris from the filtration maintenance should be disposed of at an approved facility in accordance with applicable regulations.

# Corrosivity

The following table lists the results of laboratory soluble sulfate, soluble chloride, electrical resistivity, and pH testing. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

**Corrosivity Test Results Summary**

| Boring | Sample Depth (feet) | Soil Description | Soluble Sulfate (%) | Soluble Chloride (%) | Electrical Resistivity (Ω-cm) | pH |
|---|---|---|---|---|---|---|
| P1 | 1-5 | Lean Clay with Sand (CL) | 0.0052 | 0 | 2400 | 8.2 |

Results of soluble sulfate testing can be classified in accordance with ACI 318 – Building Code Requirements for Structural Concrete. Numerous sources are available to characterize corrosion potential to buried metals using the parameters presented in the previous table. ANSI/AWWA is commonly used for ductile iron, while threshold values for evaluating the effect on steel can be specific to the buried feature (e.g., piling, culverts, welded wire reinforcement, etc.) or agency for which the work is performed. Imported fill materials may have significantly different properties than the site materials noted in the table and should be evaluated if expected to be in contact with metals used for construction. Consultation with a NACE certified corrosion professional is recommended for buried metals on the site.

Mapping by the NRCS includes qualitative severity of corrosion to concrete and steel. This source rates the near-surface materials as "Low" for corrosion to concrete and "Moderate" to "High" for corrosion of steel.

# Geotechnical Overview

The subject site has geotechnical considerations that will affect the construction and performance of the proposed improvements that are discussed in this report. The primary geotechnical consideration(s) that has/have been identified at the subject site that will affect development are the following:

- Expansive soils
- Liquefaction settlement
- Potential Pre-Existing Fill

## Expansive Soils

Expansive soils are present on this site. This report provides recommendations to help mitigate the effects of soil shrinkage and expansion. However, even if these procedures are followed, some movement and (at least minor) cracking in the structure should be anticipated. The severity of cracking and other damage such as uneven floor slabs will probably increase if modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. Some of these options are discussed in this report such as complete replacement or chemical treatment of expansive soils or using a structural slab.

The near surface, stiff to hard medium plasticity lean clay could become unstable with typical earthwork and construction traffic, especially after precipitation events. The effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

The soils which form the bearing stratum for shallow foundations are plastic and exhibit potential for shrink-swell movements with changes in moisture. Additional areas of localized highly plastic soils are likely present where borings were not performed. Maintaining above optimum moisture conditions in the bearing soils and a minimum dead load pressure on footings should reduce the anticipated swell movements to tolerable levels. The **Shallow Foundations** section addresses support of the building directly bearing on native stiff to hard lean/fat clay or structural fill. We do not expect significant dead load on the floors and recommend either overexcavation of near-surface high plasticity clays to reduce the heave potential or use of suspended slabs to accommodate potential ground heave. The **Floor Slabs** section addresses slab-on-grade support of the building using overexcavation techniques.

## Liquefaction Settlement and Lateral Spreading

The primary seismic hazards for the site are the potential for strong to very strong earthquake shaking within the lifetime of the structure, and the potential for liquefaction at the site. Seismically-induced settlements of up to ½ to 1 inch should be expected in the event of liquefaction during the design earthquake. However, potentially liquefiable soils are not continuous or of uniform thickness across the site and ground settlements are expected to be irregular around and below the improvements. In addition, we estimate approximately 7½ inches of lateral spread could occur as a result of a seismic event. The proposed building may be supported by **Shallow Foundations** or a **Mat Foundations** provided the anticipated displacements due to liquefaction can be accommodated. Alternately, the effects of liquefaction settlement can be mitigated by improving the subgrade soil with **Ground Improvement** methods.

## Potential Pre-Existing Fill

Fill material was not encountered in our borings; however, we anticipate that up to 2 feet of pre-existing fill may be encountered beneath the footprint of the existing structure. We assume any fill material encountered during site demolition will be removed to native soil depth. Pre-existing fill material may be suitable for general or structural backfill, as specified in the Fill Material Types section.

# Earthwork

We anticipate grading may consist of cuts and fills on the order of 2 feet or less and that site grades will remain at the same elevation as existing. <u>Specific site grading information was unavailable at the time this report was prepared. If elevation and site grading differ from our stated assumptions, Terracon should be contacted to determine if additional earthwork recommendations are warranted, particularly with regard to potential ground settlement.</u>

Earthwork is anticipated to include demolition, clearing and grubbing, excavations, and engineered fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations, floor slabs, and pavements.

## Demolition

The proposed building will be constructed partially within the footprint of the existing structure which will need to be demolished, along with exterior sidewalks, pavements, and utilities. We recommend existing foundations, slabs, and utilities be removed from within the proposed building footprint and at least 5 feet beyond the outer edge of foundations. This should include removal of any loose backfill found adjacent to existing foundations. If pipes are abandoned in-place, they should be filled completely with lean cement grout, or other suitable material, to avoid collapse in the future. All materials derived from the demolition of existing structures and pavements should be removed from the site and not be allowed for use as on-site fill, unless processed in accordance with the fill requirements included in this report.

For areas outside the proposed building footprints and foundation bearing zones, existing foundations, floor slabs, and utilities should be removed where they conflict with proposed utilities, retaining walls, and pavements. In such cases, existing foundations, floor slabs, and utilities should be removed to a depth of at least 2 feet below the affected utility or design pavement subgrade elevation.

## Site Preparation

Prior to placing fill, existing vegetation, topsoil, and root mats should be removed. Complete stripping of the topsoil should be performed in the proposed building and parking/driveway areas. Stripping should extend laterally a minimum of 5 feet beyond the limits of proposed improvements.

Mature trees are located within or near the footprint of some of the proposed buildings, which may require removal at the onset of construction. Tree root systems can remove substantial moisture from surrounding soils. Where trees are removed, the full root ball and all associated dry and desiccated soils should be removed. The soil materials which contain less than 3 percent organics can be reused as engineered fill provided the material is moisture conditioned and properly compacted.

Although no evidence of fill or underground facilities (such as septic tanks, cesspools, basements, and utilities) was observed during the exploration and site reconnaissance, such features could be encountered during construction. If unexpected fills or underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction.

## Existing Fill

As noted in **Geotechnical Characterization**, no pre-existing fill material was identified in our borings, but we anticipate up to 2 feet of pre-existing fill may be present under the existing structure. We have no records to indicate the degree of control under which any fill was placed, and consequently, any fill encountered during site demolition would be considered unreliable for support of foundation loads without additional characterization.

If the owner elects to construct pavements and exterior slabs on existing fill, the following protocol should be followed. The fill below pavements and exterior slab areas should be over-excavated to a depth of 2 feet and the resulting subgrade should be scarified to a minimum depth of 12 inches, moisture conditioned, and compacted per the recommendations in ***Fill Placement and Compaction Requirements***. Following compaction of the subgrade, the over-excavated areas may be backfilled with compacted structural fill. The existing fill may be reused, provided the material is cleaned of any debris and meets the criteria for general or structural fill in ***Fill Material Types***. A representative from Terracon should be present to observe the over-excavation of fill. Once the planned subgrade elevation has been reached, the entire pavement area should be proofrolled. Areas of soft or otherwise unsuitable material should be undercut and replaced with either new structural fill or suitable, existing on-site materials.

## Subgrade Preparation

After clearing, any required cuts and overexcavation should be made.

We recommend that the soils within the footprint of the proposed structures be removed to a minimum depth of 2 feet below the bottom of footings, or 2 feet below existing grades, whichever is deeper. Structural fill placed beneath the entire footprint of the foundations should extend horizontally a minimum distance of 5 feet beyond the outside edge of footings. Portions of the near-surface materials anticipated to be developed as excavation spoils are not considered suitable for use as structural fill.

Subgrade soils beneath proposed floor slabs, exterior hardscape, and pavements should be removed to a depth of 2 foot beneath proposed slab or pavement section, or existing grade, whichever is greater.

Any pre-existing uncontrolled fill below pavements and exterior slabs should be over-excavated to a minimum depth of 2 feet. The presence of over-sized debris or a high volume of organic material may warrant additional over-excavation at the time of grading operations. If needed, a geotextile fabric may be utilized as a separator between the uncontrolled fill and structural fill. This over-excavation requirement is not required in areas improved by **Ground Improvement** methods.

Where fill is placed on existing slopes steeper than 5H:1V (Horizontal:Vertical), benches should be cut into the existing slopes prior to fill placement. The benches should have a minimum vertical face height of 1 foot and a maximum vertical face height of 3 feet and should be cut wide enough to accommodate the compaction equipment. This benching will help provide a positive bond between the fill and natural soils and reduce the possibility of failure along the fill/natural soil interface.

Excavated material may be stockpiled for use as fill provided it is cleaned of organic material, debris, and any other deleterious material and meets the criteria for general or structural fill specified in the ***Fill Material Types*** section of this report.

Once cuts and over-excavation operations are complete, the resulting subgrade should be proofrolled with an adequately loaded vehicle such as a fully-loaded tandem-axle dump truck. The proofrolling should be performed under the observation of the Geotechnical Engineer or their representative. Areas excessively deflecting under the proofroll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed or modified by stabilizing as noted in the ***Soil Stabilization*** section of this report. Excessively wet or dry material should either be removed, or moisture conditioned and recompacted.

The depth of scarification of subgrade soils and moisture conditioning of the subgrade is highly dependent upon the time of year of construction and the site conditions that exist immediately prior to construction. If construction occurs during the winter or spring, when the subgrade soils are typically already in a moist condition, scarification and compaction may only be 8 inches. If construction occurs during the summer or fall when the subgrade soils have been allowed to dry out deeper, the depth of scarification and moisture conditioning may be as much as 18 inches or more. A representative from Terracon should be present during earthwork to observe the exposed subgrade and confirm the depth of scarification and moisture conditioning required.

Some of the exposed subgrade may be at an elevated moisture content where it has been covered by structures or pavement and may require some drying to achieve the required compaction. The depth of required scarification and moisture conditioning of the subgrade may therefore likely be only 6 to 8 inches. A representative from Terracon should be present to observe the exposed subgrade and specify the depth of scarification and moisture conditioning required subsequent to grading cuts and prior to placing fill.

Following scarification, moisture conditioning, and compaction of the subgrade soils, compacted structural fill soils should then be placed to the proposed design grade and the moisture content and compaction of subgrade soils should be maintained until foundation, floor slab, or pavement construction.

Very soft soil conditions could be encountered in the bottom of excavations. These soils may be unworkable. The contractor may utilize dry crushed rock or clean granular fill material placed over a geotextile such as Mirafi RS580i or equivalent to stabilize wet subgrade materials in the bottom of the excavations prior to backfill. If further soil stabilization is needed or another method is preferred or desired, Terracon should be consulted to evaluate the situation as needed.

Based upon the subsurface conditions determined from the geotechnical exploration, subgrade soils exposed during construction are anticipated to be relatively workable; however, the workability of the subgrade may be affected by precipitation, repetitive construction traffic or other factors. If unworkable conditions develop, workability may be improved by scarifying and drying.

## Excavation

We anticipate that excavations for the proposed construction can be accomplished with conventional earthmoving equipment. The bottom of excavations should be thoroughly cleaned of loose soils and disturbed materials prior to backfill placement and/or construction.

## Soil Stabilization

Soil stabilization may be needed if pre-existing fill or soft clay soils are encountered during site demolition. Methods of subgrade stabilization, as described in this section, could include scarification, moisture conditioning and recompaction, removal of unstable materials and replacement with granular fill (with or without geosynthetics), and chemical stabilization. The appropriate method of improvement, if required, would be dependent on factors such as schedule, weather, the size of area to be stabilized, and the nature of the instability. More detailed recommendations can be provided during construction as the need for subgrade stabilization occurs. Performing site grading operations during warm seasons and dry periods would help reduce the amount of subgrade stabilization required.

If the exposed subgrade is unstable during proofrolling operations, it could be stabilized using one of the following methods:

- **Scarification and Recompaction** - It may be feasible to scarify, dry, and recompact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Stable subgrades likely would not be achievable if the thickness of the unstable soil is greater than about 1 foot, if the unstable soil is at or near groundwater levels, or if construction is performed during a period of wet or cool weather when drying is difficult.

- **Aggregate Base** - The use of Caltrans Class II aggregate base is a common procedure to improve subgrade stability. Typical undercut depths would be expected to range from about 6 inches to 12 inches below finished subgrade elevation. The use of high modulus geosynthetics (i.e., engineering fabric or geogrid) could also be considered after underground work such as utility construction is completed. Prior to placing the fabric or geogrid, we recommend that all below grade construction, such as utility line installation, be completed to avoid damaging the fabric or geogrid. Equipment should not be operated above the fabric or geogrid until one full lift of aggregate base is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should meet the manufacturer's specifications.
- **Chemical Stabilization** - Improvement of subgrades with Portland cement or quicklime could be considered for improving unstable soils. Chemical stabilization should be performed by a pre-qualified contractor having experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions. The hazards of chemicals blowing across the site or onto adjacent property should also be considered. Additional testing would be needed to develop specific recommendations to improve subgrade stability by blending chemicals with the site soils. Additional testing could include, but not be limited to, determining the most suitable stabilizing agent, the optimum amounts required, and the presence of sulfates in the soil. If this method is chosen to stabilize subgrade soils the actual amount of high calcium quicklime/Portland cement to be used should be determined by Terracon and by laboratory testing **at least three weeks prior** to the start of grading operations.

Further evaluation of the need and recommendations for subgrade stabilization can be provided during construction as the geotechnical conditions are exposed.

## Fill Material Types

Fill required to achieve design grade should be classified as structural fill and general fill. Structural fill is material used below, or within 5 feet of structures, pavements or constructed slopes. General fill is material used to achieve grade outside of these areas.

**Reuse of On-Site Soil:** Excavated on-site soil may be selectively reused as fill below pavement and landscaping areas. Portions of the on-site soil have an elevated fines content and will be sensitive to moisture conditions (particularly during seasonally wet periods) and may not be suitable for reuse when above optimum moisture content.

Material property requirements for on-site soil for use as general fill and structural fill are noted in the following table:

| Property | General Fill | Structural Fill |
|---|---|---|
| Composition | Free of deleterious material | Free of deleterious material |
| Maximum particle size | 6 inches<br>(or 2/3 of the lift thickness) | 3 inches |
| Fines content | Not limited | Less than 70% Passing No. 200 sieve |
| Plasticity | Not limited | Maximum liquid limit of 30<br>Maximum plasticity index of 10 |
| GeoModel Layer Expected to be Suitable[1] | 1, 2, 3 | 2, 3 |

1. Based on subsurface exploration. Actual material suitability should be determined in the field at time of construction.

**Imported Fill Materials:** Imported fill materials should meet the following material property requirements. Regardless of its source, compacted fill should consist of approved materials that are free of organic matter and debris. For all import material, the contractor shall submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current ACI criteria and is "mildly corrosive" to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the project.

| Soil Type [1] | USCS Classification | Acceptable Parameters (for Structural Fill) |
|---|---|---|
| Low Plasticity Cohesive | CL | Liquid Limit less than 30<br>Plasticity index less than 10<br>Less than 70% passing No. 200 sieve |
| Granular[2] | GW, GM, GC, SW, SM | Less than 40% passing No. 200 sieve |

| Soil Type [1] | USCS Classification | Acceptable Parameters (for Structural Fill) |
|---|---|---|

1. Structural and general fill should consist of approved materials free of organic matter and debris and should contain no material larger than 3 inches and 6 inches in greatest dimension, respectively. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation at least two weeks prior to use on this site. Additional geotechnical consultation should be provided prior to use of uniformly graded gravel on the site.
2. Caltrans Class II aggregate base may be used for this material. Recycled aggregate base should not be used without prior approval by the Geotechnical Engineer.

## Fill Placement and Compaction Requirements

Compacted native soil and structural and general fill should meet the following compaction requirements.

| Item | Structural Fill | General Fill |
|---|---|---|
| **Maximum Lift Thickness** | 8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used<br>4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used | Same as structural fill |
| **Minimum Compaction Requirements** [1,2] | 95% of max. for structural fill below foundations and slabs, within 1 foot of finished pavement subgrade, for aggregate base and chemically treated soil, and for fills thicker than 5 feet<br>90% of max. for all other locations | 90% of max. |
| **Water Content Range** [1] | Low plasticity cohesive: +1% to +3% above optimum<br>Granular: -2% to +2% of optimum | As required to achieve min. compaction requirements |

| Item | Structural Fill | General Fill |
| --- | --- | --- |

1. Maximum density and optimum water content as determined by the Modified Proctor test (ASTM D 1557).
2. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D 4253 and D 4254). Materials not amenable to density testing should be placed and compacted to a stable condition observed full time by the Geotechnical Engineer or representative.

## Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with structural fill or bedding material in accordance with public works specifications for the utility be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1:1 projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

It is recommended utilities and piping be designed with flexible connections and/or other means to accommodate soil movement to preclude damage due to excessive settlement from liquefaction. Utility and drain lines designed for gravity flow should consider steeper gradients to account for these settlements, especially where such lines enter a building supported over soil mitigated by **Ground Improvement**.

On-site materials are considered suitable for backfill of utility and pipe trenches from 1 foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and Atterberg limit requirements for structural fill discussed in this report.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Flooding or jetting for placement and compaction of backfill is not recommended.

All trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If utility trenches are backfilled with relatively clean granular material, they should be capped with at least 18 inches of cementitious flowable fill or cohesive fill in non-pavement areas to reduce the infiltration and conveyance of surface water through the trench backfill. Attempts should also be made to limit the amount of fines migration into the clean granular material. Fines migration into clean granular fill may result in unanticipated localized settlements over a period of time. To help limit the amount of fines migration, Terracon recommends the use of a geotextile fabric that is designed to prevent fines migration in areas of contact between clean granular material and fine-grained soils. Terracon also recommends that clean granular fill be tracked or tamped in place where possible to limit the amount of future densification which may cause localized settlements over time.

For low permeability subgrades, utility trenches are a common source of water infiltration and migration. Utility trenches penetrating beneath the building should be effectively sealed to restrict water intrusion and flow through the trenches, which could migrate below the building. The trench should provide an effective trench plug that extends at least 5 feet from the face of the building exterior. The plug material should consist of cementitious flowable fill or low permeability clay. The trench plug material should be placed to surround the utility line. If used, the clay trench plug material should be placed and compacted to comply with the water content and compaction recommendations for structural fill stated previously in this report.

If chemical treatment of subgrade soils occurs before utility construction, Controlled Low Strength Material (CLSM) or sand/cement slurry should be used as backfill material to cap utility trenches in all areas where trenches have cut through the treated subgrade. The thickness of the CLSM or slurry should be at least the thickness or depth of chemically treated subgrade. Below that depth, imported structural fill or moisture conditioned native clay may be used for backfill. Such areas trenched through chemically treated soil should not be backfilled with aggregate base, native soil, or disturbed chemically treated soil.

Post construction trenching through geogrid reinforced pavement areas shall be accomplished with conventional trenching equipment. Repairs to the trenched section shall be accomplished using a full structural replacement of the displaced materials or with a repaired section that is identical to the original section. If the trench section is repaired to match the original, the trench backfill must be compacted to the same or higher density and the geogrid must be over-lapped a minimum 3-inches at the proper geogrid elevation.

## Grading and Drainage

All grades must provide effective drainage away from the building during and after construction and should be maintained throughout the life of the structure. Water retained next to the building can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge onto splash blocks a distance of at least 10 feet from the building, onto pavements, or are tied to tight lines that discharge into a storm drain system.

Exposed ground should be sloped and maintained at a minimum 5 percent away from the building for at least 10 feet beyond the perimeter of the building. If a minimum 5 percent slope cannot be achieved due to site grades, a minimum 2½ percent slope could be used provided pavement or hardscape surrounds and extends to the building, or a subdrain could be installed around the perimeter of the foundations that carries water away from the building. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structure should also be periodically inspected and adjusted, as necessary, as part of the structure’s maintenance program. Where paving or flatwork abuts the structure, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Any planters and/or bio-swales located within 10 feet of the building should be self-contained or lined with an impermeable membrane to prevent water from accessing subgrade soils below the building. Sprinkler mains and spray heads should be located a minimum of 5 feet away from the foundation lines.

No vegetation over 6 feet in height shall be planted within 20 feet of the building perimeter unless a root barrier is provided between the structure and tree to limit roots within 10 feet of the building. Roots can draw additional moisture from the soils and cause excessive volume changes in the soil resulting in building movement.

Implementation of adequate drainage for this project can affect the surrounding developments. Consequently, in addition to designing and constructing drainage for this project, the effects of site drainage should be taken into consideration for the planned structures on this property, the undeveloped portions of this property, and surrounding sites. Extra care should be taken to ensure irrigation and drainage from adjacent areas do not drain onto the project site or saturate the construction area.

## Earthwork Construction Considerations

Shallow excavations for the proposed structure are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of grade-supported improvements such as floor slabs, exterior hardscape, and pavements. Construction traffic over the completed subgrades should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade should become desiccated, saturated, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to construction.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through April) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork operations may require additional mitigation measures beyond that which would be expected during the drier summer and fall months. This could include ground stabilization utilizing chemical treatment of the subgrade, diversion of surface runoff around exposed soils, and draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local and/or state regulations. Stockpiles of soil, construction materials, and construction equipment should not be placed near trenches or excavations. ***The Contractor is responsible for maintaining the stability of adjacent structures during construction.***

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Excavations or other activities resulting in ground disturbance have the potential to affect adjoining properties and structures. Our scope of services does not include review of available final grading information or consider potential temporary grading performed by the contractor for potential effects such as ground movement beyond the project limits. A preconstruction/precondition survey should be conducted to document nearby property/infrastructure prior to any site development activity. Excavation or ground disturbance activities adjacent or near property lines should be monitored or instrumented for potential ground movements that could negatively affect adjoining property and/or structures.

## Construction Observation and Testing

The earthwork efforts should be observed by the Geotechnical Engineer (or others under their direction). Observation should include documentation of adequate removal of surficial materials (vegetation, topsoil, debris, and pavements), evaluation and remediation of existing fill materials, as well as proofrolling and mitigation of unsuitable areas delineated by the proofroll.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, as recommended by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 1,500 square feet of compacted fill in the building areas and 2,500 square feet in pavement areas. Where not specified by local ordinance, one density and water content test should be performed for every 50 linear feet of compacted utility trench backfill and a minimum of one test performed for every 12 vertical inches of compacted backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated by the Geotechnical Engineer. If unanticipated conditions are observed, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

# Shallow Foundations

The proposed building may be supported by spread footings. If the site has been prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable for shallow foundations.

## Design Parameters – Compressive Loads

| Item | Description |
|---|---|
| **Maximum Net Allowable Bearing Pressure [1, 2]** | 2,000 psf |
| **Required Bearing Stratum [3]** | Undisturbed native soils or structural fill extending to undisturbed native soils. |
| **Minimum Foundation Dimensions** | Per CBC 1809.4 |
| **Maximum Foundation Dimensions** | Column footings: 8 feet<br>Continuous footings: 4 feet |
| **Passive Resistance[4, 8] (equivalent fluid pressures)** | 250 pcf (cohesive backfill)<br>360 pcf (granular backfill) |
| **Sliding Resistance [5, 8]** | 500 psf allowable cohesion (native/structural fill clay)<br>0.35 allowable coefficient of friction - granular material |

| Item | Description |
|---|---|
| **Minimum Embedment below Finished Subgrade** [6] | Exterior footings: 24 inches<br>Interior footings: 24 inches |
| **Estimated Total Settlement from Structural Loads** [2] | Less than about 1 inch |
| **Estimated Differential Settlement** [2, 7] | About ½ of total settlement |

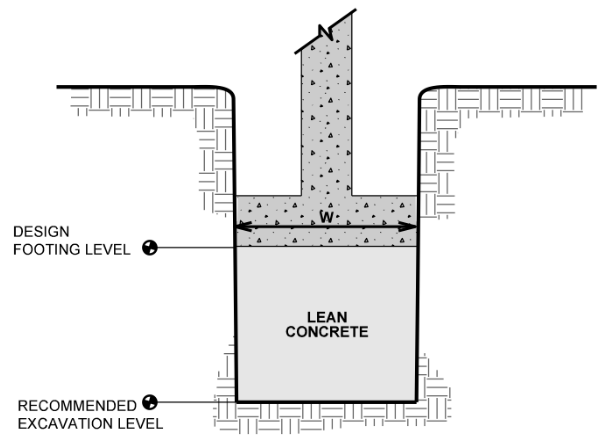
1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. This bearing pressure can be increased by 1/3 for transient loads unless those loads have been factored to account for transient conditions. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.
2. Values provided are for maximum loads noted in **Project Description**. Additional geotechnical consultation will be necessary if higher loads are anticipated. **Estimated settlement does not include settlement due to liquefaction.**
3. Unsuitable or soft soils should be overexcavated and replaced per the recommendations presented in **Earthwork**.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face. Assumes no hydrostatic pressure.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Frictional resistance for granular materials is dependent on the bearing pressure which may vary due to load combinations. For fine-grained materials, lateral resistance using cohesion should not exceed ½ the dead load.
6. Embedment necessary to minimize the effects of seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior subgrade within 5 horizontal feet of the structure.
7. Differential settlements are noted for equivalent-loaded foundations and bearing elevation as measured over a span of 50 feet.
8. Passive Resistance and Sliding Resistance may be combined to resist sliding provided the Passive Resistance is reduced by 50 percent.

## Foundation Construction Considerations

As noted in **Earthwork**, the footing excavations should be evaluated under the observation of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

Sensitive soils exposed at the surface of footing excavations may require surficial compaction with hand-held dynamic compaction equipment prior to placing structural fill, steel, and/or concrete. Should surficial compaction not be adequate, construction of a working surface consisting of either crushed stone or a lean concrete mud mat may be required prior to the placement of reinforcing steel and construction of foundations.

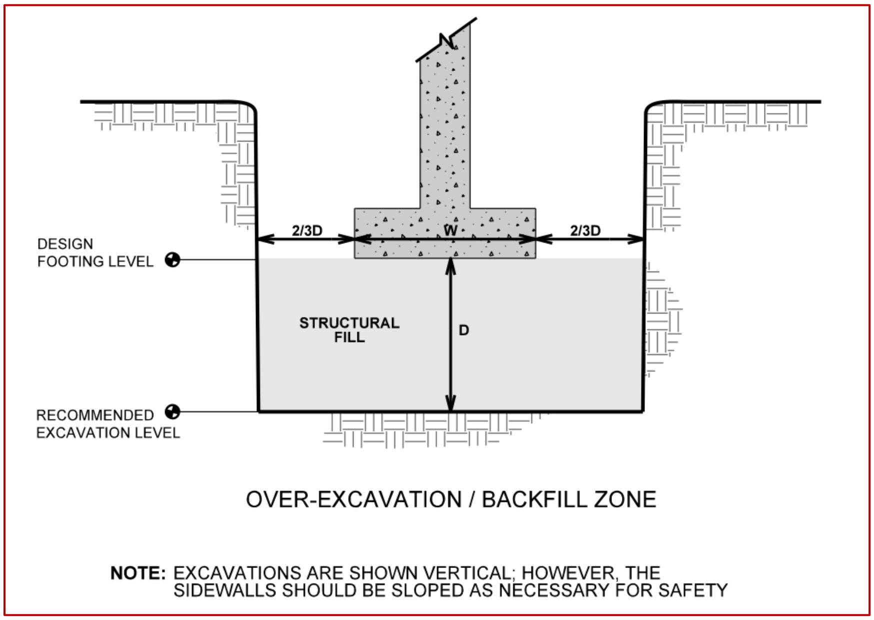
If unsuitable bearing soils are observed at the base of the planned footing excavation, the excavation should be extended deeper to suitable soils, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. The lean concrete replacement zone is illustrated on the following sketch.




LEAN CONCRETE BACKFILL

**NOTE:** EXCAVATIONS ARE SHOWN VERTICAL; HOWEVER, THE SIDEWALLS SHOULD BE SLOPED AS NECESSARY FOR SAFETY

Overexcavation for structural fill placement below footings should be conducted as shown in the following sketch. The overexcavation should be backfilled up to the footing base elevation, with granular structural fill placed, as recommended in the **Earthwork** section.




To ensure foundations have adequate support, special care should be taken when footings are located adjacent to trenches. The bottom of such footings should be at least 1 foot below an imaginary plane with an inclination of 1.5 horizontal to 1.0 vertical extending upward from the nearest edge of the adjacent trench.

# Ground Improvement

Ground improvement may be utilized to help mitigate the anticipated excessive settlement due to the potential liquefaction and lateral spreading of the underlying medium dense sand layers. Ground improvement methods such as aggregate piers and drilled displacement columns (DDC) are proprietary systems designed by licensed contractors who could provide further information regarding support options. Considering the various methods available for ground improvement, it is our opinion aggregate piers would be suitable options for ground improvement at this site. However, if the Contractor or Structural Engineer have worked with a different ground improvement method that has proven successful to mitigate the hazards present at this site with similar subgrade soil conditions, Terracon could consider such options if desired.

## Aggregate Piers

As a way to mitigate the anticipated settlement and effects of liquefaction below the proposed improvements, the subgrade soils could be improved with aggregate piers installed on a grid pattern. This option would allow for the use of **Shallow Foundations** or a **Mat Foundation** over the aggregate pier-reinforced subgrade. Aggregate pier systems are typically installed after clearing. Aggregate piers can be used to densify the potentially liquefiable soils.

Aggregate piers are typically constructed by advancing a drill or mandrel to design depths, then building a bottom bulb of clean, open-graded stone. The pier is built on top of the bottom bulb, using graded aggregate placed in thin lifts (12 to 24 inches compacted thickness). We anticipate the aggregate piers would need to extend to a depth of at least 15 feet bgs at the site depending on the soil layers targeted for improvement. The result is a reinforced zone of soils directly under the foundations, which allows for the design and construction of foundations for relatively higher bearing pressures and with lower anticipated settlements. Aggregate piers can also be installed where differential movement is a concern between underground utility lines; site development such as hardscape, entrances, and pavements adjacent to structures supported by **Ground Improvement** and site drainage.

We anticipate foundations supported over aggregate piers installed following fill placement may be designed using an allowable bearing pressure of 2,500 to 3,000 pound per square foot (psf) for dead plus live loads. However, the final design allowable bearing pressure should be confirmed by the design-build contractor installing the aggregate piers and coordinated with the structural engineer. The aggregate pier ground improvement system for this project should meet the following design criteria:

Bearing Capacity Factor of Safety = 2.0
Global Stability (static) = 1.3
Global Stability (dynamic) = 1.1
Post-construction Settlement: <1 inch for combined static and liquefaction settlement
Post-construction Differential Settlement: < ½ inch / 40 feet for combined static and liquefaction settlement

Aggregate pier systems should be designed and constructed by a specialty ground improvement contractor. At least one load test should be performed in the footprint of each improvement to confirm the aggregate pier design capacity prior to full production of aggregate piers. Since this would be specialty work, we recommend consideration of using a design-build process if this alternative is selected. A design and installation package including a quality control plan for aggregate pier installation should be prepared by the design-build contractor license in the State of California and submitted to Terracon for review and approval prior to construction. The package should also include information regarding load testing such as proposed test location, set-up, and testing parameters. Terracon should be present on-site during load test and production to observe installation and testing of the aggregate piers.

## Drilled Displacement Columns

DDCs would help increase the bearing capacity of the subgrade soils while reducing settlement and help mitigate potential liquefaction settlement below the improvements by transferring their loads to deeper, more competent soils and improving the soil around the DDCs. DDCs are constructed by using a displacement auger to create a soil shaft filled with Controlled Low Strength Material (CLSM) injected under pressure as the displacement auger is withdrawn. DDCs generate a minimal amount of soil cuttings compared to other foundation and ground improvement methods. The diameter of DDCs typically vary between 18 to 36 inches and based on foundation load requirements the strength of CLSM can typically vary between 100 to 500 psi at 28 days.

We anticipate foundations supported on DDCs may be designed using an allowable bearing pressure of 3,000 to 3,500 pounds per square foot (psf) for dead plus live loads. However, the final design allowable bearing pressure should be confirmed by the design-build contractor installing the DDCs and coordinated with the structural engineer. The DDC ground improvement system for this project should meet the following design criteria:

- Bearing Capacity Factor of Safety = 2.0
- Global Stability (static) = 1.3
- Global Stability (dynamic) = 1.1
- Post-construction Settlement: < 1-inch for combined static and liquefaction settlement
- Post-construction Differential Settlement:< ½ -inch / 40 feet for combined static and liquefaction settlement

DDC capacities and settlement based on anticipated loading should be evaluated by a design-build contractor. Terracon should be afforded the opportunity to review the bearing capacity and settlement evaluation.

We anticipate the DDCs would need to extend to a depth of at least 15 feet bgs depending on the soil layers targeted for improvement. Minimum DDC lengths shall be confirmed by Terracon during construction. A 12 to 24-inch cushion of Caltrans Class II aggregate base should be installed between the top of the DDCs and the bottom of foundations and slabs.

Design and installation of DDCs shall be performed by a qualified design-build contractor. At least one load test should be performed in the footprint of each improvement to confirm DDC capacities prior to installing production columns. Axial compression load tests shall be performed according to ASTM D1143. Only columns meeting the load testing criteria shall be used as final production columns. A design and installation package including a quality control plan for DDC installation should be prepared by the design-build contractor and submitted to Terracon for review and approval prior to construction. The package should also include information regarding load test such as proposed test location, set-up, and testing parameters. Terracon should be present on-site during load test and production to observe installation and testing of the DDCs.

# Floor Slabs

Design parameters for floor slabs assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the floor slab support course beneath the floor slab.

Up to 2 feet of existing fill materials may be encountered beneath the existing site structure. As previously described, any existing fill present beneath floor slabs should be completely removed or further evaluated by the Geotechnical Engineer.

The subgrade soils are comprised of medium plasticity clays exhibiting the potential to shrink/swell with variations in water content. Construction of the floor slab, combined with the removal of trees, and revising site drainage creates the potential for gradual increased water contents within the clays. Increases in water content will cause the clays to swell and damage the floor slab. To reduce the potential effects of the medium plasticity clays on the building floor slabs, at least the upper 18 inches of subgrade soils below the floor slab should consist of granular structural fill or be chemically treated with high calcium quicklime.

Chemical treatment involves treating the subgrade soils with a certain percentage of high calcium quicklime, usually 3.5 to 5.5 percent based on the dry unit weight of the soil, for a depth of 18 inches. For estimating purposes, we recommend using 4.5 percent lime, and a soil unit weight of 110 pounds per cubic foot. For an 18-inch treatment depth, this results in an estimated minimum spread rate of 7.4 pounds per square foot lime. The actual amount of lime to be used should be determined by Terracon and by laboratory testing **at least three weeks prior** to the start of grading operations. Chemical treatment is performed after rough grading is completed. This procedure reduces the swell potential of the surface soils and creates a stable working platform on which construction can proceed. All chemical treatment operations should be observed by a representative of the project Geotechnical Engineer.

Due to the potential for significant moisture fluctuations of subgrade material beneath floor slabs supported at-grade, the Geotechnical Engineer should evaluate the material within 18 inches of the bottom of the structural granular fill or chemically treated zone immediately prior to placement of additional fill or floor slabs. In chemically treated areas, this can be accomplished by having the grading contractor excavate several test pits within the proposed construction areas prior to the start of grading operations to determine the moisture condition of the subgrade soils. A representative of the Geotechnical Engineer should be present during the excavation of these test pits and samples of the subgrade soils should be obtained for moisture content testing. Soils below the specified water contents within this zone should be moisture conditioned or replaced with structural fill as stated in our **Earthwork** section.

## Floor Slab Design Parameters

| Item | Description |
|---|---|
| **Floor Slab Support**[1] | Use 4 inches of ¾ inch free draining crushed rock[3] over 18 inches of low-plasticity or granular structural fill<br><br>Subgrade compacted to the recommendations in **Earthwork** |
| **Estimated Modulus of Subgrade Reaction** [2] | 90 pounds per square inch per inch (psi/in) for point loads |

1. Floor slabs should be structurally independent of building footings or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
2. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the floor slab support as noted in this table. It is provided for point loads. For large area loads the modulus of subgrade reaction would be lower.
3. Free-draining granular material should have less than 5% fines (material passing the No. 200 sieve). Other design considerations such as cold temperatures and condensation development could warrant more extensive design provisions.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, when the project includes humidity-controlled areas, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut contraction joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations, refer to the ACI Design Manual. Joints or cracks should be sealed with a waterproof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing, or other means.

## Floor Slab Construction Considerations

Finished subgrade, within and for at least 10 feet beyond the floor slab, should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed, and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

The Geotechnical Engineer should observe the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

## Exterior Hardscape

In order to help protect the exterior hardscape against the swell pressure of the surficial moderately plastic clays, we recommend the subgrade soil below hardscapes either be over-excavated to a minimum depth of 12 inches and replaced with compacted granular structural fill per the recommendations provided in this report or be chemically treated to a depth of 18 inches.

Exterior hardscape, exterior architectural features, and utilities may experience some movement due to the volume change of the subgrade soils. To reduce the potential for damage caused by movement, we recommend:

- Slabs should be underlain by a minimum of 12 inches of compacted granular structural fill or chemically treated material as indicated. However, at the contractor’s discretion, gravel may be placed between the slab and granular structural fill or chemically treated material to assist with constructability.
- Minimizing moisture increases in the subgrade soils and backfill;
- Controlling moisture-density during placement of fill;
- Using designs which allow vertical movement between the exterior features and adjoining structural elements;
- Placing effective control joints on relatively close centers.
- Ensuring clay subgrade soils are in a moist condition prior to slab construction.

- Reinforcing exterior slabs and flatwork with a minimum No. 4 bars at 12 inches on center.

# Pavements

## Pavement Subgrade Support Characteristics

R-Value testing was incomplete at the time of this report; an addendum letter will be provided upon its completion. In the interim, we recommend that a subgrade R-Value of 5 be used for the asphaltic concrete pavement designs. We recommend that a modulus of subgrade reaction of 100 pci be used for the Portland cement concrete pavement designs. These values were empirically derived based upon our experience with the clay subgrade soils and our expectation of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**.

## General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in **Project Description** and in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs noted in this section must be applied to the site which has been prepared as recommended in the **Earthwork** section.

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are placed and compacted in a uniform manner. However, as construction proceeds, excavations are made into these areas, rainfall and surface water saturates some areas, heavy traffic from concrete trucks and other delivery vehicles disturbs the subgrade and many surface irregularities are filled in with loose soils to improve trafficability temporarily. As a result, the pavement subgrades, initially prepared early in the project, should be carefully evaluated as the time for pavement construction approaches.

We recommend the moisture content and density of the top 12 inches of the subgrade be evaluated and the pavement subgrades be proofrolled within two days prior to commencement of actual paving operations. Areas not in compliance with the required ranges of moisture or density should be moisture conditioned and recompacted. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted fills.

If a significant precipitation event occurs after the evaluation or if the surface becomes disturbed, the subgrade should be reviewed by qualified personnel immediately prior to paving. The subgrade should be in its finished form at the time of the final review.

## Pavement Design Parameters

Design of Asphaltic Concrete (AC) pavement sections were calculated using the Caltrans Highway Design Manual, latest edition, and a 20-year design life. Design of Portland Cement Concrete (PCC) pavement sections were designed using ACI 330R-21, "Guide for the Design and Construction of Concrete Parking Lots."

Bulk samples of the near surface native soils were collected to perform Hveem Stabilometer (R-Value) testing. An addendum letter will be provided upon completing of the testing. In the interim, , an R-Value of 5 was used for the subgrade for the asphaltic concrete (AC) pavement designs. A modulus of subgrade reaction of 100 pci was used for the Portland cement concrete (PCC) pavement designs. The value was empirically derived based upon our experience with the clay subgrade soils and our expectation of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**. A modulus of rupture of 550 psi was used in design for the concrete (based on correlations with a minimum 28-day compressive strength of 4,500 psi).

Recommendations for conventional pavement sections are presented next. The recommendations are based on the subgrade being in a firm and unyielding condition.

## Pavement Section Thicknesses

The following table provides our opinion of minimum thickness for AC sections:

**Asphaltic Concrete Design**

| Layer | Thickness (inches) | | | |
|---|---|---|---|---|
| | Auto Parking Areas (TI=5.0)[1] | Auto Road (TI=5.5)[1] | Truck Parking Areas (TI=6.0)[1] | Truck Parking Areas (TI=8.0)[1] |
| AC [2, 3] | 3.0 | 3.5 | 3.5 | 5.0 |
| Aggregate Base[2] | 10.0 | 11.0 | 13.0 | 18.0 |

1. See **Project Description** for more specifics regarding traffic assumptions.
2. All materials should meet the current Caltrans Highway Design Manual specifications.

**Asphaltic Concrete Design**

| Layer | Thickness (inches) | | | |
|---|---|---|---|---|
| | Auto Parking Areas (TI=5.0)[1] | Auto Road (TI=5.5)[1] | Truck Parking Areas (TI=6.0)[1] | Truck Parking Areas (TI=8.0)[1] |

- Base – Caltrans Class 2 aggregate base

3. A minimum 1.5-inch surface course should be used on ACC pavements.

**Portland Cement Concrete Design**

| Layer | Thickness (inches) | | |
|---|---|---|---|
| | Traffic Category A [1] | Traffic Category B [1] | Traffic Category E [1] |
| PCC [2] | 5.0 | 6.5 | 7.5 |
| Aggregate Base[2] | 4.0 | 6.0 | 6.0 |

1. See **Project Description** for more specifics regarding traffic classifications.
2. All materials should meet the current Caltrans Highway Design Manual specifications.

Areas for parking of heavy vehicles, concentrated turn areas, and start/stop maneuvers could require thicker pavement sections. Edge restraints (i.e. concrete curbs or aggregate shoulders) should be planned along curves and areas of maneuvering vehicles.

Although not required for structural support, a minimum 4-inch-thick base course layer is recommended to help reduce potential for slab curl, shrinkage cracking, and subgrade pumping through joints. Proper joint spacing will also be required to prevent excessive slab curling and shrinkage cracking. Joints should be sealed to prevent entry of foreign material and doweled where necessary for load transfer. PCC pavement details for joint spacing, joint reinforcement, and joint sealing should be prepared in accordance with ACI 330 and ACI 325.

Where practical, we recommend early-entry cutting of crack-control joints in PCC pavements. Cutting of the concrete in its "green" state typically reduces the potential for micro-cracking of the pavements prior to the crack control joints being formed, compared to cutting the joints after the concrete has fully set. Micro-cracking of pavements may lead to crack formation in locations other than the sawed joints, and/or reduction of fatigue life of the pavement.

Openings in pavements, such as decorative landscaped areas, are sources for water infiltration into surrounding pavement systems. Water can collect in the islands and migrate into the surrounding subgrade soils thereby degrading support of the pavement. Islands with raised concrete curbs, irrigated foliage, and low permeability near-surface soils are particular areas of concern. The civil design for the pavements with these conditions should include features to restrict or collect and discharge excess water from the islands. Examples of features are edge drains connected to the stormwater collection system, longitudinal subdrains, or other suitable outlets and impermeable barriers preventing lateral migration of water such as a cutoff wall installed to a depth below the pavement structure.

## Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

The pavement surfacing, and adjacent sidewalks should be sloped to provide rapid drainage of surface water. Water should not be allowed to pond on or adjacent to these grade-supported slabs, since this could saturate the subgrade and contribute to premature pavement or slab deterioration. In areas where pavement sections abut bioswales, curb should extend below the planned AB section to intercept water infiltration below the pavement section. Water migration in and out of the pavement sections may result in repeated shrinkage and swelling and increasing pavement section fatigue.

## Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic upkeep should be anticipated. Preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Pavement care consists of both localized (e.g., crack, and joint sealing and patching) and global maintenance (e.g., surface sealing). Additional engineering consultation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur, and repairs may be required.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Final grade adjacent to paved areas should slope down from the edges at a minimum 2%.
- Subgrade and pavement surfaces should have a minimum 2% slope to promote proper surface drainage.
- Install pavement drainage systems surrounding areas anticipated for frequent wetting.
- Install joint sealant and seal cracks immediately.
- Seal all landscaped areas in or adjacent to pavements to reduce moisture migration to subgrade soils.
- Place compacted, low permeability backfill against the exterior side of curb and gutter.

# General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

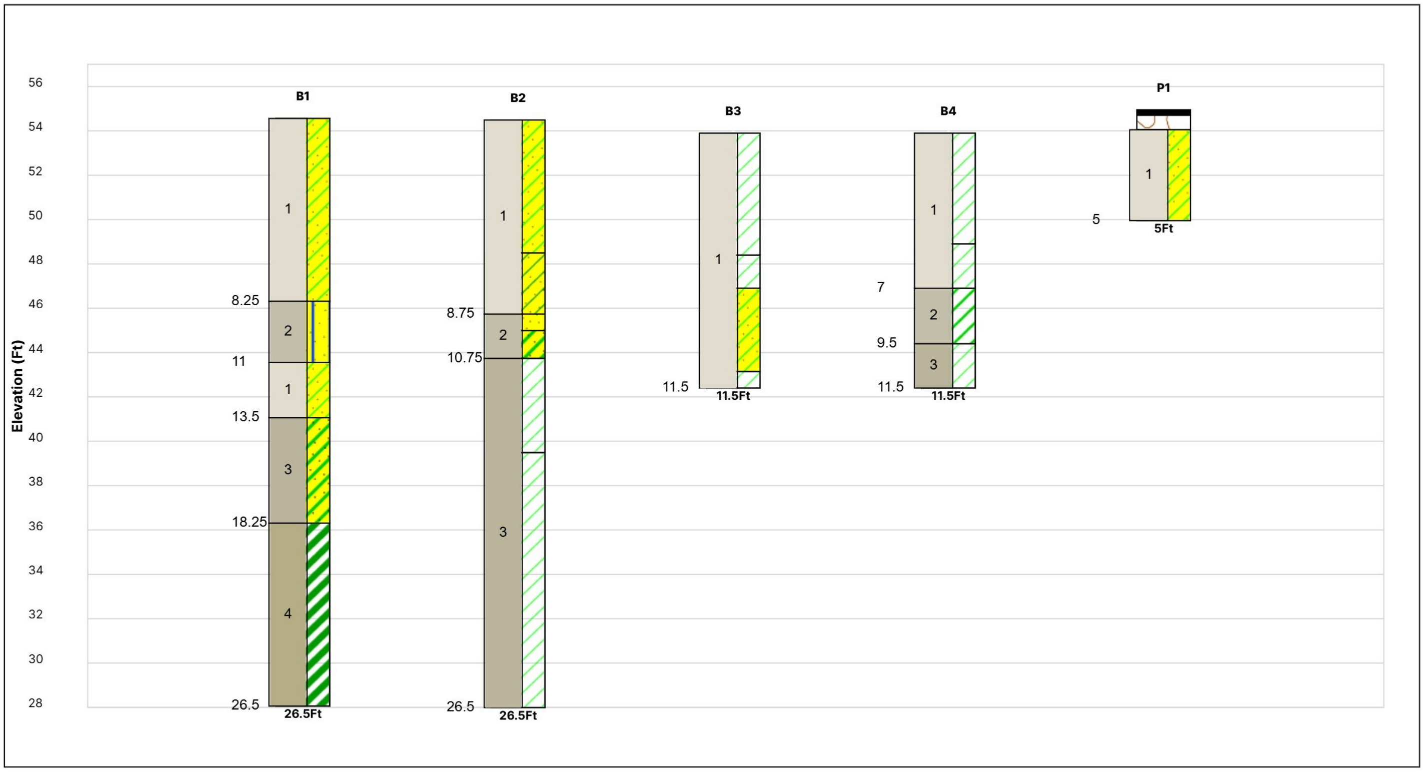
Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing. This report should not be used after 3 years without written authorization from Terracon.

# Figures

**Contents:**

GeoModel

# GeoModel




This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions

| # | Layer Name | General Description |
|---|---|---|
| 1 | **Lean Clay** | Medium Plasticity Lean Clay with Fine Sand |
| 2 | **Silty Sand** | Silty/Clayey Fine-Grained Sand |
| 3 | **Lean Clay** | Lean Clay with Trace Sand |
| 4 | **Fat Clay** | High Plasticity Fat Clay in Boring B1 |

**Legend**


Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

First Water Observation

Second Water Observation

Third Water Observation

**Notes:**

Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below groundsurface.

# Attachments

# Exploration and Testing Procedures

## Field Exploration

| Number of Borings/CPTs | Approximate Boring/CPT Depth (feet) | Location |
|---|---|---|
| 2 Borings | 26.5 | pavement / utilities / buildings |
| 2 Borings | 11.5 | pavement / parking |
| 1 Boring | 5 | pavement / utilities / buildings |
| 1 CPT | 100 | pavement / utilities / buildings |

**Boring Layout and Elevations:** Terracon personnel provided the boring and CPT layout using handheld GPS equipment (estimated horizontal accuracy of about ±10 feet) and referencing existing site features. Approximate ground surface elevations were estimated using Google Earth. If elevations and a more precise boring/CPT layout are desired, we recommend the exploration locations be surveyed.

**Subsurface Exploration Procedures:** We advanced the borings with a truck-mounted rotary drill rig using continuous flight augers (solid stem and/or hollow stem, as necessary, depending on soil conditions). Four samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge was pushed hydraulically into the soil to obtain a relatively undisturbed sample. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. A 3-inch O.D. split-barrel sampling spoon with 2.5-inch I.D. tube lined sampler was used for sampling in the upper 25 feet. Tube-lined, split-barrel sampling procedures are similar to standard split spoon sampling procedure; however, blow counts are typically recorded for 6-inch intervals for a total of 18 inches of penetration. We observed and recorded groundwater levels during drilling and sampling. For safety purposes, all borings were backfilled with cement-grout after their completion. Pavements were patched with cold-mix asphalt and/or pre-mixed concrete, as appropriate.

We also observed the boreholes while drilling and at the completion of drilling for the presence of groundwater. Groundwater was not observed at these times in the boreholes.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials observed during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

For the cone penetrometer testing, the CPT rig hydraulically pushed an instrumented cone through the soil while nearly continuous readings were recorded to a portable computer. The cone was equipped with electronic load cells to measure tip resistance and sleeve resistance and a pressure transducer to measure the generated ambient pore pressure. The face of the cone has an apex angle of 60° and an area of 15 $cm^2$. Digital Data representing the tip resistance, friction resistance, pore water pressure, and probe inclination angle were recorded about every 2 centimeters while advancing through the ground at a rate between 1½ and 2½ centimeters per second. These measurements were correlated to various soil properties used for geotechnical design. In addition, seismic shear-wave velocities were measured every 1.5 to 3 meters (5 to 10 feet) of probe advancement in CPT-01. No soil samples were gathered through this subsurface investigation technique. CPT testing was conducted in general accordance with ASTM D5778 "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils."

## Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- Moisture Content
- Dry Unit Weight
- Unconfined Compression
- Atterberg Limits
- Hveem Stabilometer

The laboratory testing program included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the Unified Soil Classification System.

# Photography Log

Northeast portion of property and existing structure


Southern portion of property – CPT and P1 location

# Site Location and Exploration Plans

**Contents:**



Note: All attachments are one page unless otherwise noted.

## Site Location

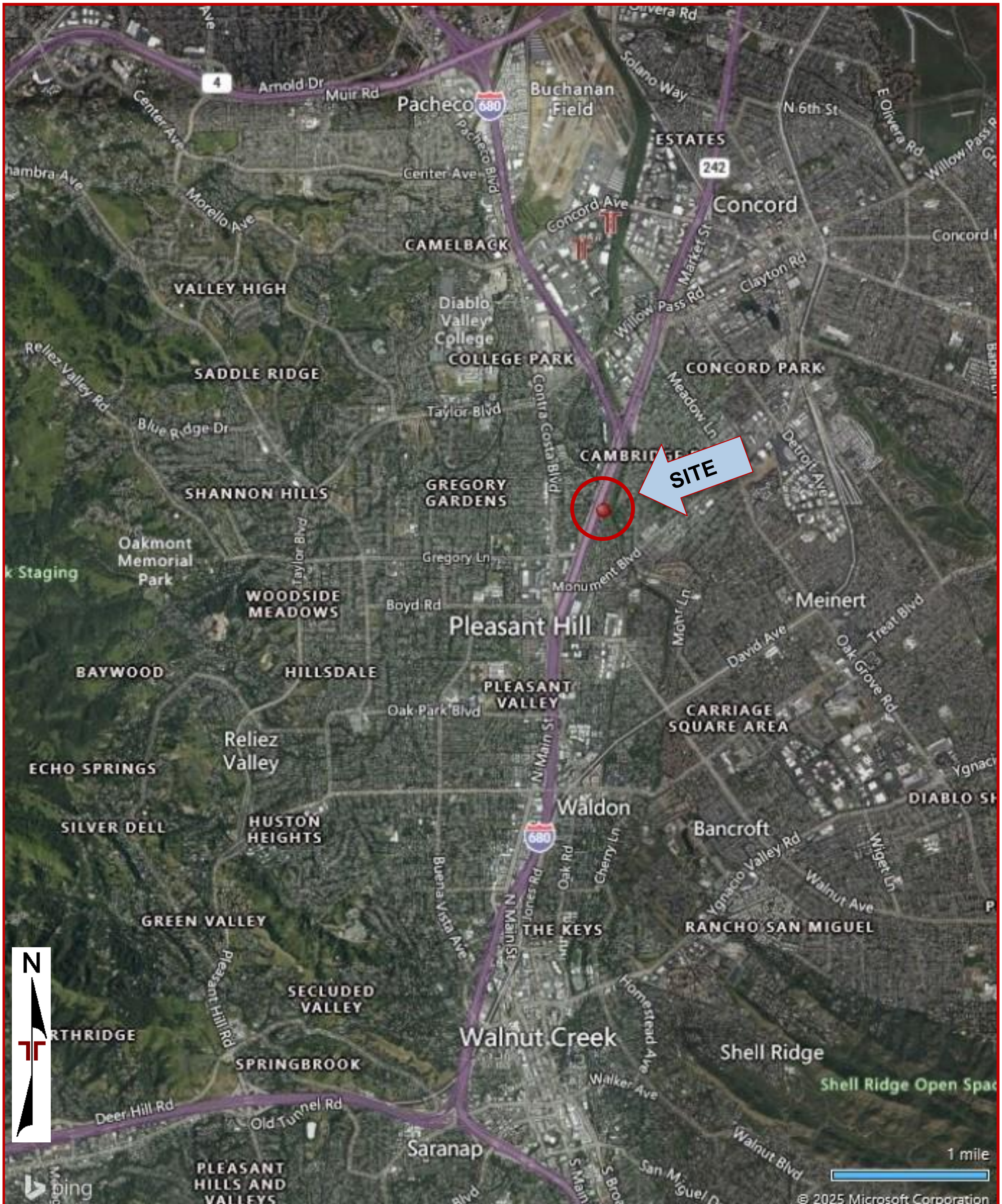



DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES MAP PROVIDED BY MICROSOFT BING MAPS

## Exploration Plan

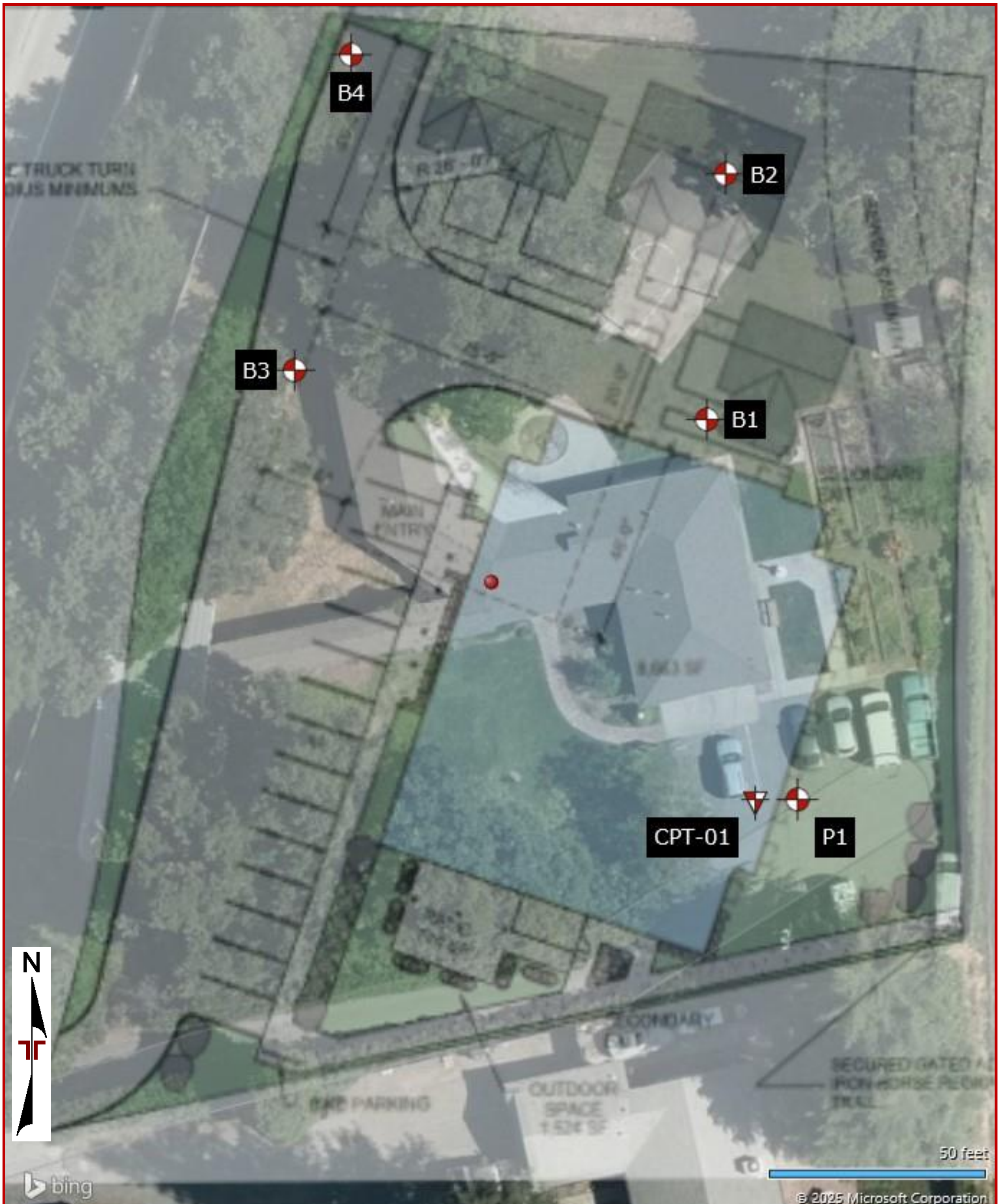



DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES
MAP PROVIDED BY MICROSOFT BING MAPS

# Exploration and Laboratory Results

**Contents:**



Note: All attachments are one page unless otherwise noted.

Sherman Recovery Center
2025 Sherman Drive, Pleasant Hill, CA, 94523,USA
Terracon Project No. R1255060

Latitude: 37.9525° Longitude: -122.0541°

# BORING LOG NO. B1

Terracon
1220 Concord Ave., Ste. 450
Concord, CA 94520-4906

Surface Elevation:
54.56(Ft) +/-

| Model Layer | Graphic Log | Lithology Depth (Ft.) | Material Description | Depth (Ft.) | Elevation (Ft.) | Sample Type | Recovery (In.) | Field Test Results | Unconfined Compressive Strength (tsf) | Water Content (%) | Dry Unit Weight (pcf.) | Percent Fines | Atterberg Limits LL | Atterberg Limits PL | Atterberg Limits PI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | **LEAN CLAY WITH SAND (CL)**, with fine grained sand, dark brown, trace roots | | | | | | | | | | | | |
| | | | | | | | 18 | 5, 6, 7 | | 10.6 | | | | | |
| | | | | 5.0 | | | | | | | | | | | |
| | | | | | | | 17 | 9, 9, 10 | | 12.8 | 82.53 | | | | |
| | | 8.3 | | | 46.3 | | 18 | 9, 10, 13 | | 9.6 | 84.4 | | | | |
| 2 | | | **SILTY SAND (SM)**, fine grained, light yellowish brown, dry, medium dense | | | | | | | | | | | | |
| | | | | 10.0 | | | | | | | | | | | |
| | | 11.0 | | | 43.6 | | 13 | 12, 12, 16 | | 9.4 | | | | | |
| 1 | | | **LEAN CLAY WITH SAND (CL)**, with fine grained sand, dark brown, stiff | | | | | | | | | | | | |
| | | 13.5 | | | 41.1 | | | | | | | | | | |
| 3 | | | **CLAYEY SAND (SC)**, fine to medium grained, dark brown, moist, medium dense | | | | | | | | | | | | |
| | | | | 15.0 | | | 14 | 10, 14, 15 | 0.92 | 14.9 | 113.5 | 32.1 | NP | NP | NP |
| | | | | | | | | | | 14.9 | 101.3 | | | | |
| | | 18.3 | | | 36.3 | | | | | | | | | | |
| 4 | | | **FAT CLAY (CH)**, dark brown, moist, stiff, trace fine sand | | | | | | | | | | | | |
| | | | | 20.0 | | | 12 | 12, 14, 15 | | 26.4 | 91.33 | | | | |
| | | | | 25.0 | | | 13 | 11, 14, 15 | | 23.9 | 89.83 | | | | |
| | | | **Boring Terminated at 26.5 Ft** | | | | | | | | | | | | |

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.

**Notes**

**Abandonment Method**
Boring backfilled with bentonite grout upon completion.

**Drill Rig**
Subcontractor - Mobile B-24

**Hammer Type**
Automatic

**Logged By**
Ethan Franklin

**Boring Started**
10/07/2025

**Boring Completed**
10/07/2025

Facilities | Environmental | Geotechnical | Materials

Sherman Recovery Center
2025 Sherman Drive, Pleasant Hill, CA, 94523,USA
Terracon Project No. R1255060

Latitude: 37.9526°    Longitude: -122.0541°

# BORING LOG NO. B2

Terracon
1220 Concord Ave., Ste. 450
Concord, CA 94520-4906

Surface Elevation:
54.49(Ft) +/-

| Model Layer | Graphic Log | Lithology Depth (Ft.) | Material Description | Depth (Ft.) | Elevation (Ft.) | Sample Type | Recovery (In.) | Field Test Results | Unconfined Compressive Strength (tsf) | Water Content (%) | Dry Unit Weight (pcf.) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | **LEAN CLAY WITH SAND (CL)**, with fine grained sand, brown, medium stiff | | | | | | | | |
| | | | | | | X | 9 | 5, 6, 7 | | 13.3 | |
| | | | | 5.0 | | X | 13 | 14, 17, 12 | | 15.4 | 91.77 |
| | | 6.0 | **SANDY LEAN CLAY (CL)**, fine grained sand, light brown, dry, medium stiff | | 48.5 | | | | | | |
| | | | | | | X | 14 | 10, 11, 11 | | 10.9 | 87.34 |
| 2 | | 8.8 | **POORLY GRADED SAND (SP)**, fine to medium grained, light yellowish brown, loose | | 45.7 | | | | | | |
| | | 9.5 | **CLAYEY SAND (SC)**, fine grained, light yellowish brown, medium dense | 10.0 | 45.0 | X | 14 | 9, 10, 13 | | 9.9 | 102 |
| 3 | | 10.8 | **LEAN CLAY (CL)**, trace sand, dark brown, moist, very stiff | | 43.7 | | | | | | |
| | | 15.0 | medium stiff | 15.0 | 39.5 | X | 17 | 8, 10, 14 | 1.54 | 21.5 | 107.5 |
| | | | | | | | | | | 21.5 | 94.89 |
| | | | | 20.0 | | X | 18 | 10, 12, 16 | | 26.2 | 89.21 |
| | | | | 25.0 | | X | 16 | 10, 12, 14 | | 23.2 | 92.64 |
| | | | **Boring Terminated at 26.5 Ft** | | | | | | | | |

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.

**Notes**

**Abandonment Method**
Boring backfilled with bentonite grout upon completion.

**Drill Rig**
Subcontractor - Mobile B-24

**Hammer Type**
Automatic

**Logged By**
Ethan Franklin

**Boring Started**
10/07/2025

**Boring Completed**
10/07/2025

Sherman Recovery Center
2025 Sherman Drive, Pleasant Hill, CA, 94523,USA
Terracon Project No. R1255060

Latitude: 37.9525° Longitude: -122.0545°

# BORING LOG NO. B3

1220 Concord Ave., Ste. 450
Concord, CA 94520-4906

Surface Elevation:
53.9(Ft) +/-

| Model Layer | Graphic Log | Lithology Depth (Ft.) | Material Description | Depth (Ft.) | Elevation (Ft.) | Sample Type | Recovery (In.) | Field Test Results | Water Content (%) | Dry Unit Weight (pcf.) | Percent Fines | Atterberg Limits LL | Atterberg Limits PL | Atterberg Limits PI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | **LEAN CLAY (CL)**, brown, dry, soft to medium stiff, trace roots | | | | | | | | | | | |
| | | | | | | | 15 | 4, 4, 5 | 11.6 | 77.1 | | | | |
| | | 5.5 | dark brown | 5.0 | 48.4 | | 14 | 12, 15, 13 | 17.9 | 87.15 | | | | |
| | | 7.0 | **LEAN CLAY WITH SAND (CL)**, with fine to medium grained sand, brown, stiff | | 46.9 | | 16 | 10, 12, 15 | 10.4 | 98.95 | 58.3 | 31 | 14 | 17 |
| | | 10.8 | **LEAN CLAY (CL)**, dark brown, dry, stiff | 10.0 | 43.1 | | 13 | 10, 13, 20 | 14.4 | | | | | |

**Boring Terminated at 11.5 Ft**

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.

**Notes**

**Abandonment Method**
Boring backfilled with bentonite grout upon completion.

**Drill Rig**
Subcontractor - Mobile B-24

**Hammer Type**
Automatic

**Logged By**
Ethan Franklin

**Boring Started**
10/07/2025

**Boring Completed**
10/07/2025

Sherman Recovery Center
2025 Sherman Drive, Pleasant Hill, CA, 94523,USA
Terracon Project No. R1255060

Latitude: 37.9526° Longitude: -122.0543°

# BORING LOG NO. B4

1220 Concord Ave., Ste. 450
Concord, CA 94520-4906

Surface Elevation:
53.9(Ft) +/-

| Model Layer | Graphic Log | Lithology Depth (Ft.) | Material Description | Depth (Ft.) | Elevation (Ft.) | Sample Type | Recovery (In.) | Field Test Results | Unconfined Compressive Strength (tsf) | Water Content (%) | Dry Unit Weight (pcf.) | Percent Fines |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | **LEAN CLAY (CL)**, dark brown, dry, medium stiff to stiff, trace roots | | | | 16 | 6, 6, 7 | | 13.8 | 70.86 | |
| | | 5.0 | brown, stiff | 5.0 | 48.9 | | 15 | 7, 9, 10 | | 14.3 | 90.83 | |
| 2 | | 7.0 | **POORLY GRADED SAND WITH CLAY (SP-SC)**, medium grained, with non-plastic clay, light brown, dry, loose to medium dense | | 46.9 | | 17 | 10, 10, 10 | | 7.5 | 106.1 | 30.3 |
| 3 | | 9.5 | **LEAN CLAY (CL)**, trace sand, brown, dry, stiff | 10.0 | 44.4 | | 11 | 8, 12, 17 | 7.69 | 9.7 | 117.4 | |
| | | | **Boring Terminated at 11.5 Ft** | | | | | | | 9.7 | 104 | |

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.

**Notes**

**Abandonment Method**
Boring backfilled with bentonite grout upon completion.

**Drill Rig**
Subcontractor - Mobile B-24

**Hammer Type**
Automatic

**Logged By**
Ethan Franklin

**Boring Started**
10/07/2025

**Boring Completed**
10/07/2025

Facilities | Environmental | Geotechnical | Materials

Sherman Recovery Center
2025 Sherman Drive, Pleasant Hill, CA, 94523,USA
Terracon Project No. R1255060

Latitude: 37.9522° Longitude: -122.0540°

# BORING LOG NO. P1

Terracon

1220 Concord Ave., Ste. 450
Concord, CA 94520-4906

Surface Elevation:
54.95(Ft) +/-

| Model Layer | Graphic Log | Lithology Depth (Ft.) | Material Description | Depth (Ft.) | Elevation (Ft.) | Sample Type | Water Content (%) |
|---|---|---|---|---|---|---|---|
| | | 0.3 | **ASPHALT** | | 54.7 | | |
| | | 0.9 | **AGGREGATE BASE COURSE** | | 54.1 | | |
| 1 | | | **LEAN CLAY WITH SAND (CL)**, dark brown | | | | 4.1 |

**Boring Terminated at 5 Ft**

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.

**Notes**

**Abandonment Method**
Boring backfilled with cement grout and capped with concrete upon completion.

**Drill Rig**
Subcontractor - Mobile B-24

**Hammer Type**
Automatic

**Logged By**
Ethan Franklin

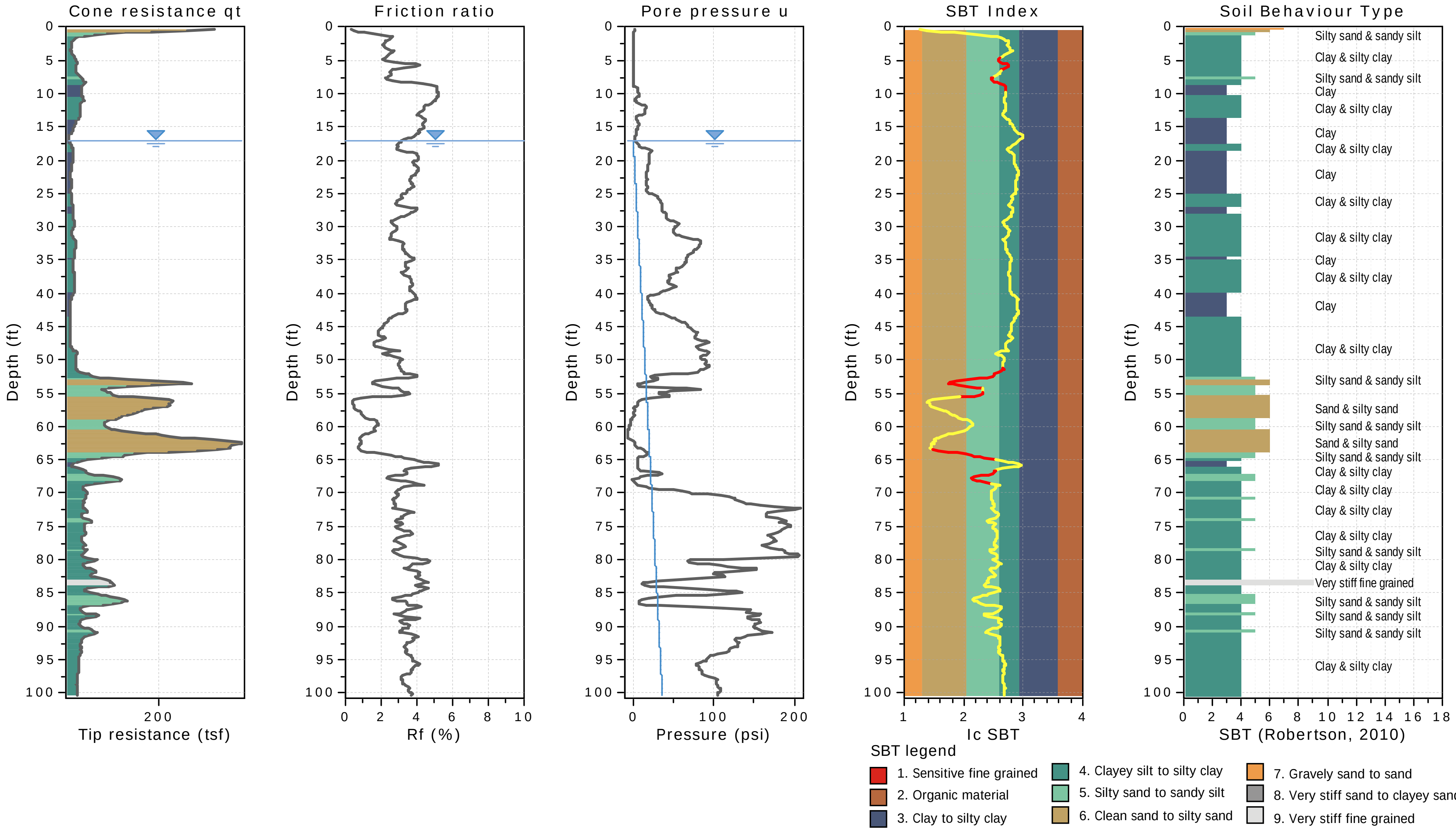
**Boring Started**
10/07/2025

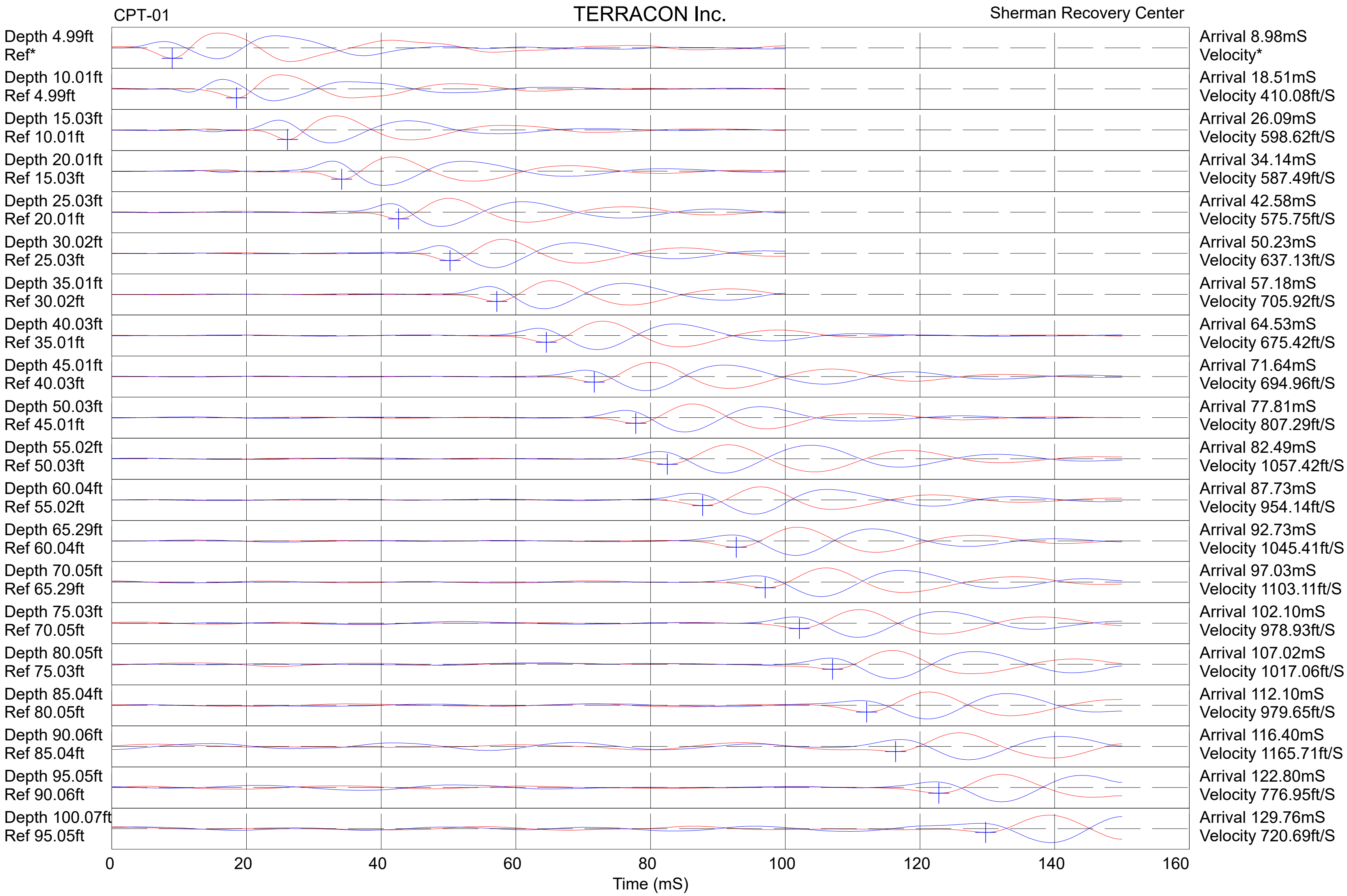
**Boring Completed**
10/07/2025

Terracon Consultants, Inc
1220 Concord Ave, Suite 450 I
Concord, CA 94520
Terracon.com

CPT: CPT-01
Total depth: 100.39 ft, Date: 10/15/2025
Surface Elevation: 0.00 ft, Est. GWL: 17.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Sherman Recovery Center
Location: 2025 Sherman Drive, Pleasant Hill, CA





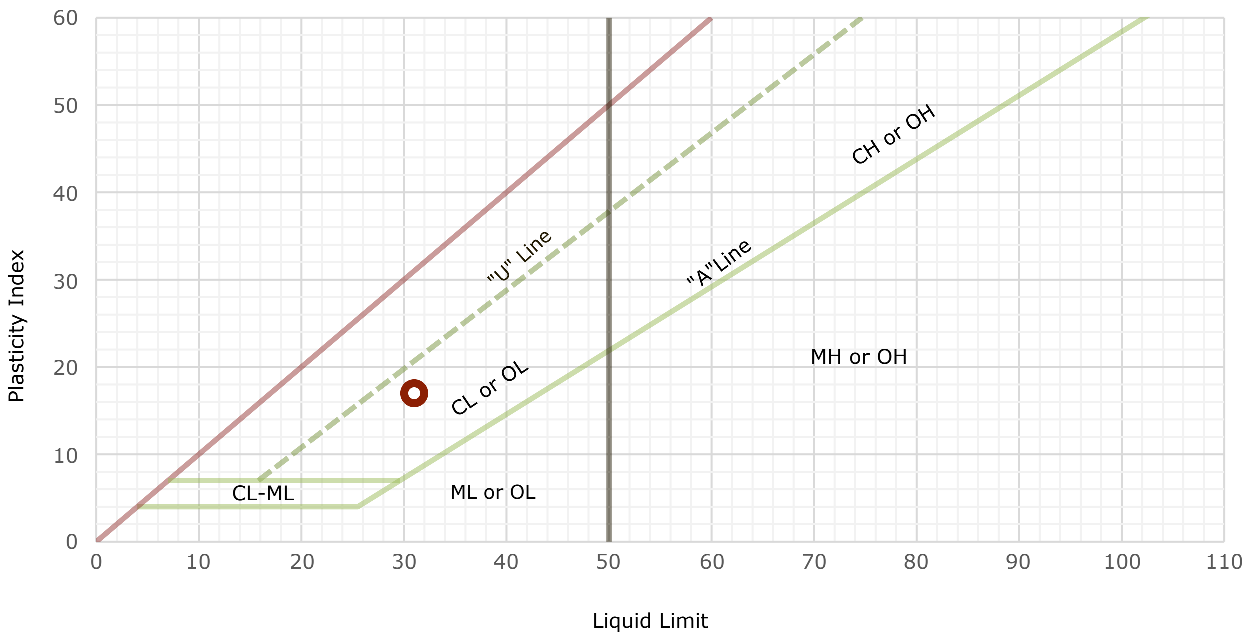

Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Terracon
1220 Concord Ave, Ste 450
Concord, CA 94520-4906

# Liquid Limit, Plastic Limit and Plasticity Index of Soils

**ASTM D4318**



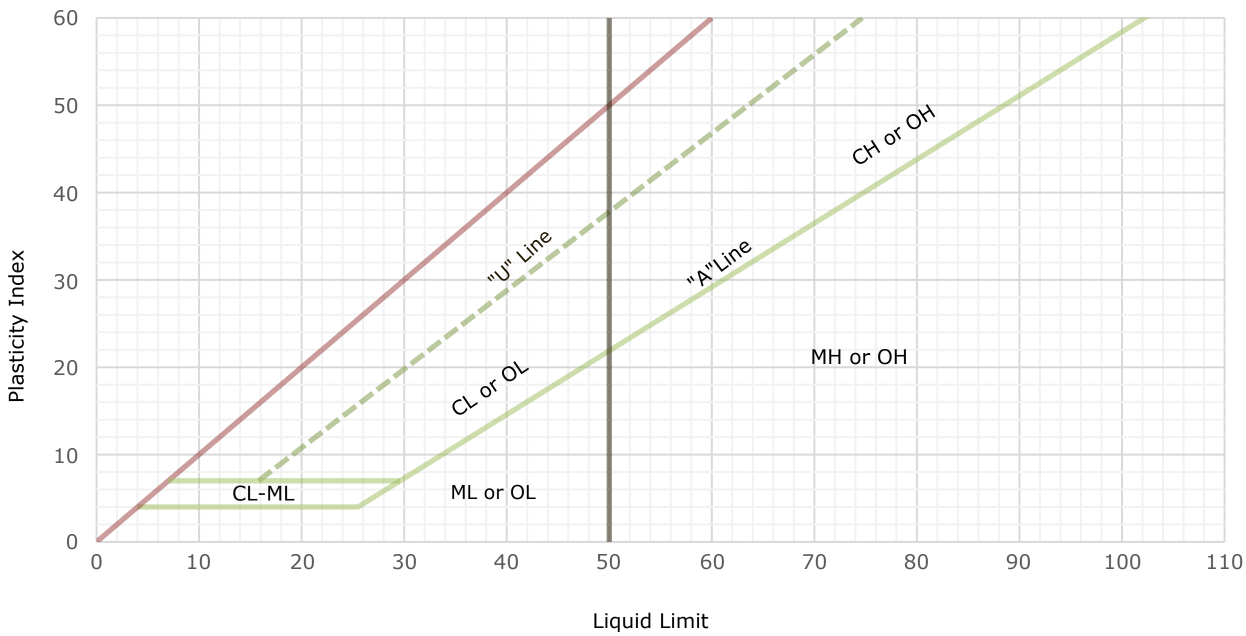

| Boring ID | Depth (Ft) | LL | PL | PI | Fines (%) | Description | USCS |
|---|---|---|---|---|---|---|---|
| B3 | 7.5-9 | 31 | 14 | 17 | 58.3 | Lean Clay with Sand | CL |

| Remarks | |
|---|---|

Terracon
1220 Concord Ave, Ste 450
Concord, CA 94520-4906

# Liquid Limit, Plastic Limit and Plasticity Index of Soils

**ASTM D4318**



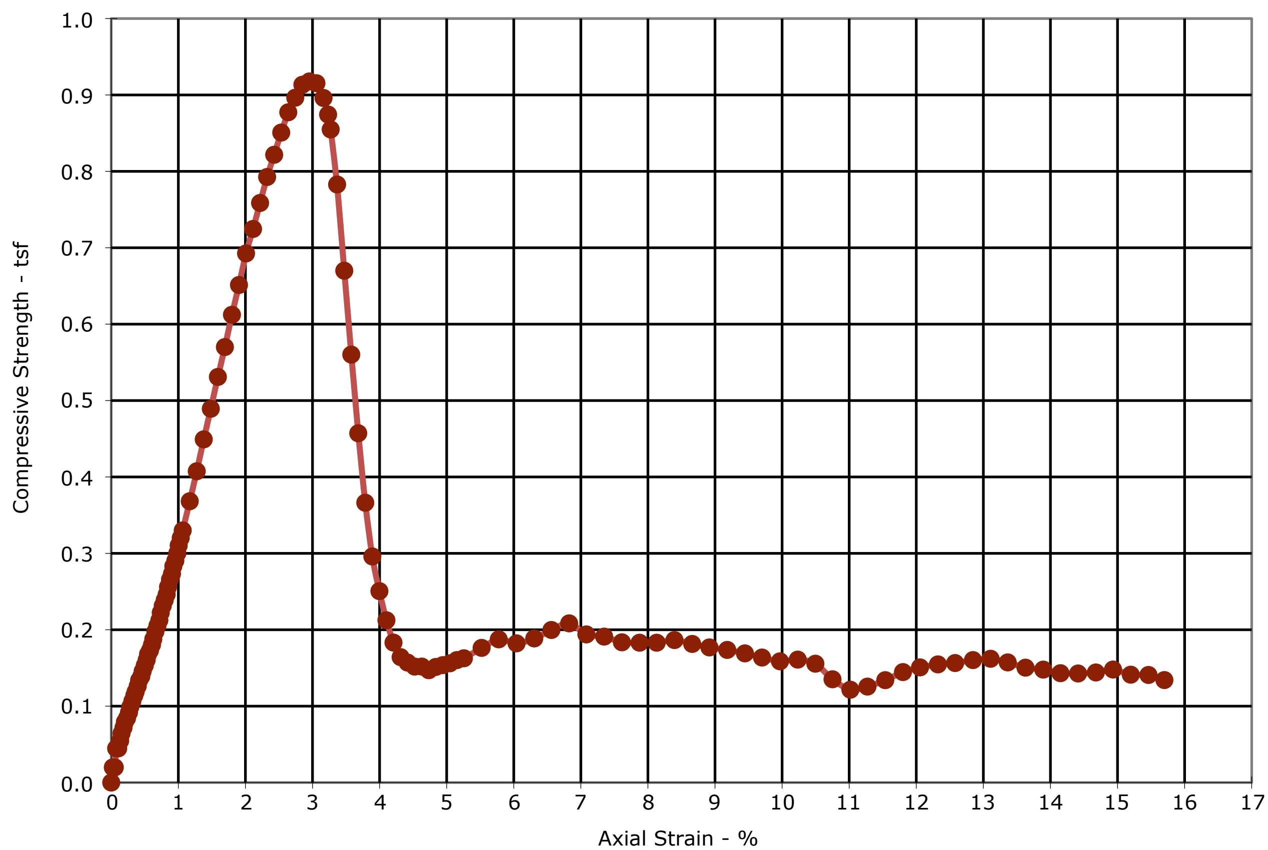

| Boring ID | Depth (Ft) | LL | PL | PI | Fines (%) | Description | USCS |
|---|---|---|---|---|---|---|---|
| B1 | 15-16.5 | NP | NP | NP | 32.1 | Clayey Sand | SC |

| **Remarks** | |
|---|---|

Terracon
1220 Concord Ave, Ste 450
Concord, CA 94520-4906

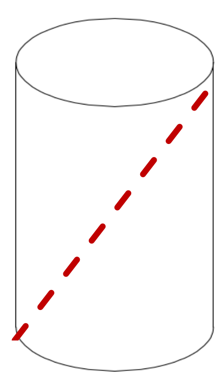
# Unconfined Compressive Strength of Cohesive Soil

**ASTM D2166**




| Boring ID | Depth (Ft) | Sample Type | LL | PL | PI | USCS | Fines (%) | Description |
|---|---|---|---|---|---|---|---|---|
| B1 | 15 - 16.5 | Intact | NP | NP | NP | SC | 32.1 | Clayey Sand |

**Specimen Failure Mode**



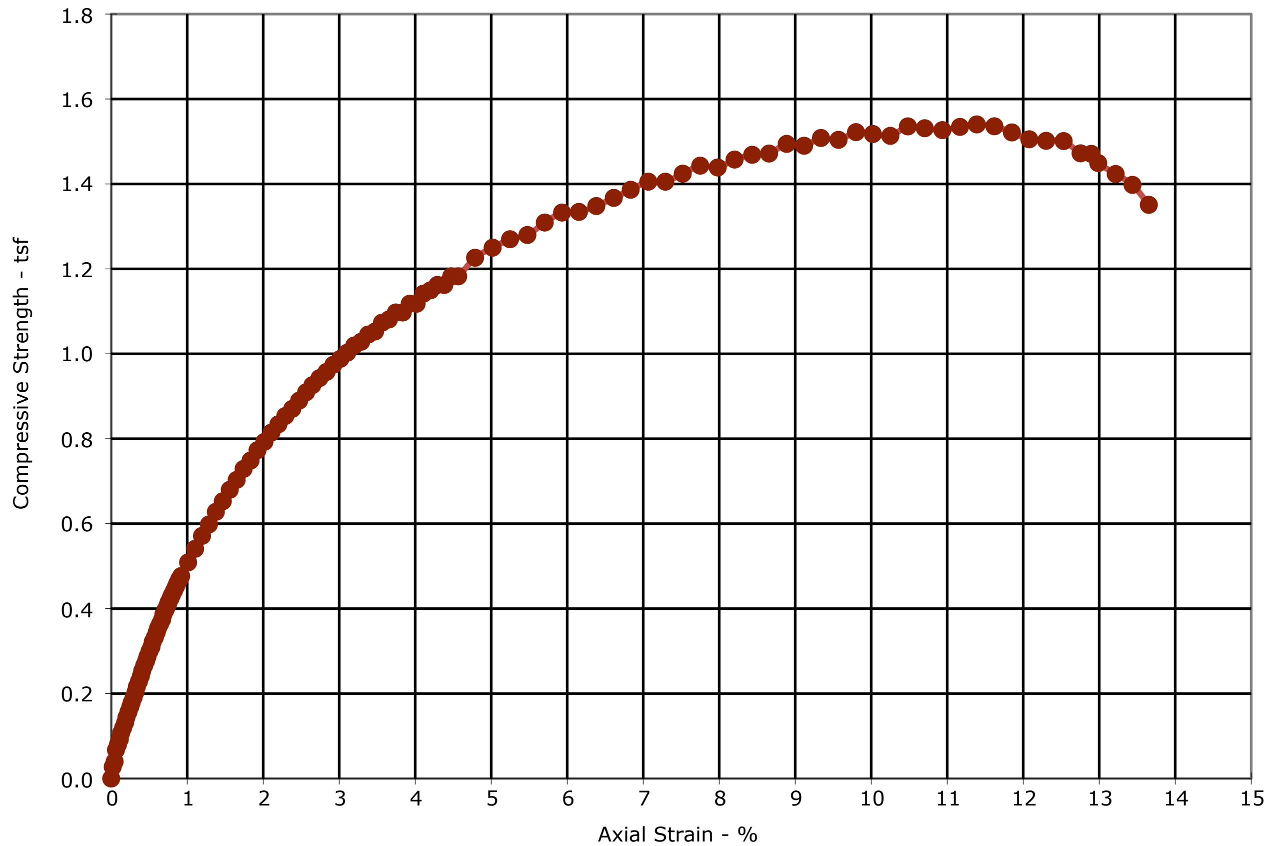
**Specimen Test Data**

| | |
|---|---|
| Water Content (%): | 14.9 |
| Dry Density (pcf): | 113.5 |
| Diameter (in.): | 2.39 |
| Height (in.): | 4.78 |
| Height / Diameter Ratio: | 2.00 |
| Calculated Saturation (%): | 83.0 |
| Calculated Void Ratio: | 0.49 |
| Assumed Specific Gravity: | 2.7 |
| Failure Strain (%): | 3.0 |
| Unconfined Compressive Strength (tsf): | 0.92 |
| Undrained Shear Strength (tsf): | 0.46 |
| Strain Rate (%/min): | 1.0 |
| Remarks: | |

Terracon
1220 Concord Ave, Ste 450
Concord, CA 94520-4906

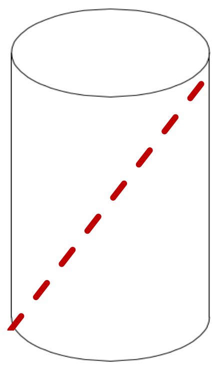
# Unconfined Compressive Strength of Cohesive Soil

**ASTM D2166**




| Boring ID | Depth (Ft) | Sample Type | LL | PL | PI | USCS | Fines (%) | Description |
|---|---|---|---|---|---|---|---|---|
| B2 | 15 - 16.5 | Intact | | | | | | Lean Clay |

**Specimen Failure Mode**



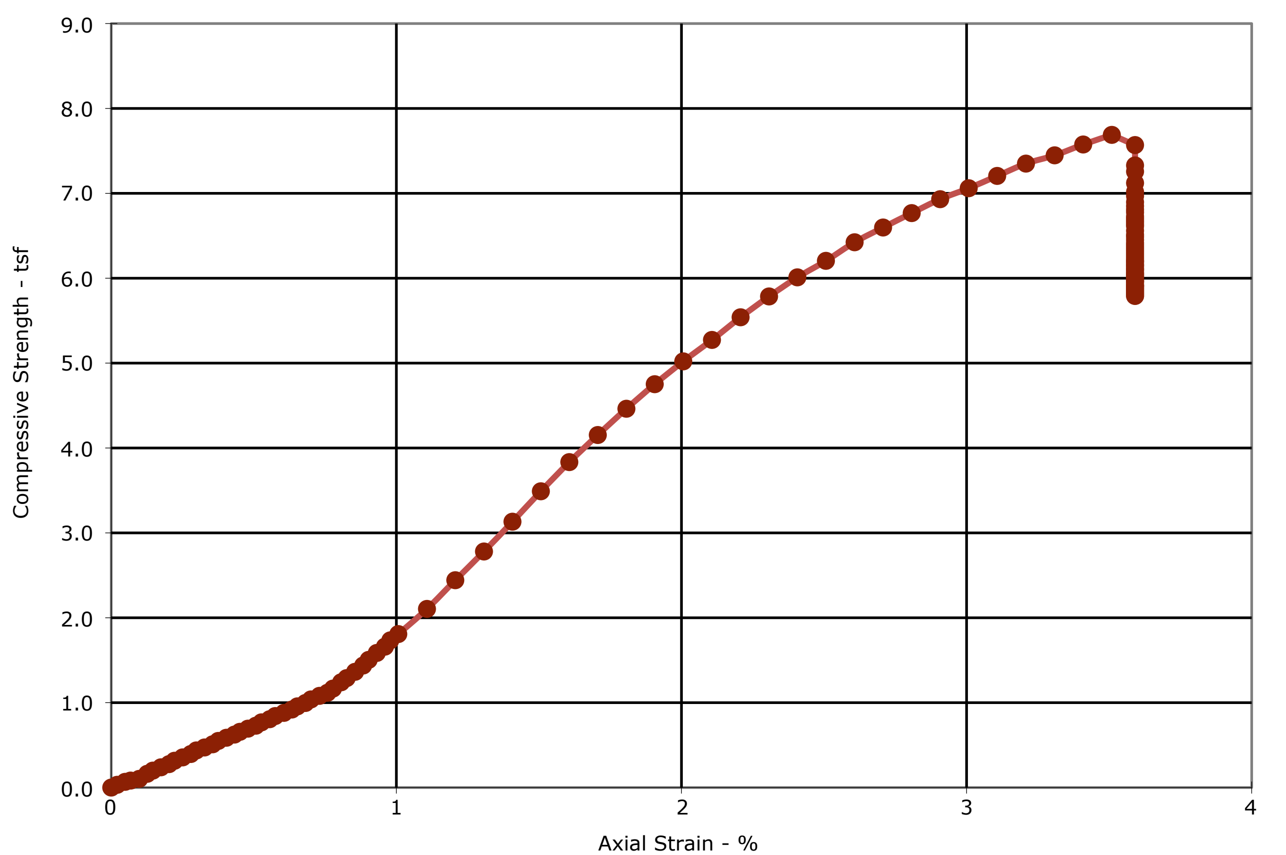
**Specimen Test Data**

| | |
|---|---|
| Water Content (%): | 21.5 |
| Dry Density (pcf): | 107.5 |
| Diameter (in.): | 2.39 |
| Height (in.): | 5.50 |
| Height / Diameter Ratio: | 2.30 |
| Calculated Saturation (%): | 101.9 |
| Calculated Void Ratio: | 0.57 |
| Assumed Specific Gravity: | 2.7 |
| Failure Strain (%): | 11.4 |
| Unconfined Compressive Strength (tsf): | 1.54 |
| Undrained Shear Strength (tsf): | 0.77 |
| Strain Rate (%/min): | 0.9 |
| Remarks: | |

Terracon
1220 Concord Ave, Ste 450
Concord, CA 94520-4906

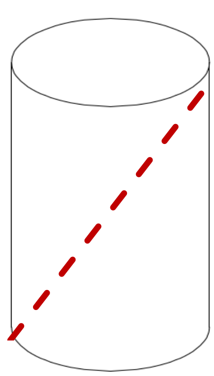
# Unconfined Compressive Strength of Cohesive Soil

**ASTM D2166**




| Boring ID | Depth (Ft) | Sample Type | LL | PL | PI | USCS | Fines (%) | Description |
|---|---|---|---|---|---|---|---|---|
| B4 | 10 - 11.5 | Intact | | | | | | Lean Clay |

**Specimen Failure Mode**



**Specimen Test Data**

| | |
|---|---|
| Water Content (%): | 9.7 |
| Dry Density (pcf): | 117.4 |
| Diameter (in.): | 2.39 |
| Height (in.): | 4.00 |
| Height / Diameter Ratio: | 1.68 |
| Calculated Saturation (%): | 60.0 |
| Calculated Void Ratio: | 0.44 |
| Assumed Specific Gravity: | 2.7 |
| Failure Strain (%): | 3.5 |
| Unconfined Compressive Strength (tsf): | 7.69 |
| Undrained Shear Strength (tsf): | 3.85 |
| Strain Rate (%/min): | 0.9 |
| Remarks: | |

Terracon
902 Industrial Way
Lodi, CA 95240-3106

# Soil Corrosivity Profile

## Summary Report

| | |
|---|---|
| Boring ID | P1 |
| Sample Depth, (ft) | 1 |
| Sample Description | Sandy Lean Clay |

| Test Method | Test Designation | Test Result |
|---|---|---|
| Electrical Resistivity (Ω·cm) | ASTM G57 | 2400 |
| Oxidation Reduction Potential (RmV) | ASTM G200 | 101 |
| Acidity (pH) | AASHTO T289 | 8.2 |
| Chloride Ion Content (mg/kg) | AWWA -4500-CL-B | 0 |
| Soluble Sulfate Content (mg/kg) | ASTM D516 | 52 |
| Total Dissolved Salts (mg/kg) | AWWA 2520 B | 127 |
| Sulfides (mg/kg) | AWWA 4500-S2- D | 0 |

| Instrument Identifications | | | |
|---|---|---|---|
| **Oxidation Reduction Potential** | | **Acidity (pH)** | |
| Probe ID | Oakton Testr 50 | Meter ID | Oakton pH Testr 30 |
| Meter ID | Oakton Testr 50 | Thermometric Type | Digital |
| Thermometric Type | Digital | | |

# Supporting Information

**Contents:**

General Notes
CPT General Notes
Unified Soil Classification System
Liquefaction Analysis Results

Note: All attachments are one page unless otherwise noted.

# General Notes

## Sampling

| | | |
|---|---|---|
| Auger Cuttings | Modified California Ring Sampler | Rock Core |
| Dynamic Cone Penetrometer | Modified Dames & Moore Ring Sampler | Dual Sampler SPT |
| Grab Sample | GeoProbe Macro Core or Large Bore | No Recovery |
| Ring Sampler | Shelby Tube | Standard Penetration Test |
| Split Spoon | Texas Cone Penetrometer | Vane Shear |

## Water Level

- Water Initially Encountered
- Water Level After a Specified Period of Time
- Water Level After a Specified Period of Time
- Cave In Encountered

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

## Field Tests

| | |
|---|---|
| N | Standard Penetration Test Resistance (Blows/Ft.) |
| (HP) | Hand Penetrometer |
| (T) | Torvane |
| (DCP) | Dynamic Cone Penetrometer |
| UC | Unconfined Compressive Strength |
| (PID) | Photo-Ionization Detector |
| (OVA) | Organic Vapor Analyzer |

## Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

## Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

## Strength Terms

| **Relative Density of Coarse-Grained Soils** (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance | | **Consistency of Fine-Grained Soils** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance | | |
|---|---|---|---|---|
| **Relative Density** | **Standard Penetration or N-Value (Blows/Ft.)** | **Consistency** | **Unconfined Compressive Strength Qu (tsf)** | **Standard Penetration or N-Value (Blows/Ft.)** |
| Very Loose | 0 - 3 | Very Soft | less than 0.25 | 0 - 1 |
| Loose | 4 - 9 | Soft | 0.25 to 0.50 | 2 - 4 |
| Medium Dense | 10 - 29 | Medium Stiff | 0.50 to 1.00 | 5 - 8 |
| Dense | 30 - 50 | Stiff | 1.00 to 2.00 | 9 - 15 |
| Very Dense | > 50 | Very Stiff | 2.00 to 4.00 | 16 - 30 |
| | | Hard | > 4.00 | > 30 |

## Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

# CPT GENERAL NOTES

Terracon

## DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

Uncorrected Tip Resistance, $q_c$
Measured force acting on the cone divided by the cone's projected area

Corrected Tip Resistance, $q_t$
Cone resistance corrected for porewater and net area ratio effects
$q_t = q_c + u_2(1 - a)$

Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

Pore Pressure, u
Pore pressure measured during penetration
$u_1$ - sensor on the face of the cone
$u_2$ - sensor on the shoulder (more common)

Sleeve Friction, $f_s$
Frictional force acting on the sleeve divided by its surface area

Normalized Friction Ratio, $F_r$
The ratio as a percentage of $f_s$ to $q_t$, accounting for overburden pressure

To be reported per ASTM D7400, if collected:

Shear Wave Velocity, $V_s$
Measured in a Seismic CPT and provides direct measure of soil stiffness

## DESCRIPTION OF GEOTECHNICAL CORRELATIONS

Normalized Tip Resistance, $Q_{tn}$
$Q_{tn} = ((q_t - \sigma_{V0})/P_a)(P_a/\sigma'_{V0})^n$
$n = 0.381(I_c) + 0.05(\sigma'_{V0}/P_a) - 0.15$

Over Consolidation Ratio, OCR
OCR (1) $= 0.25(Q_{tn})^{1.25}$
OCR (2) $= 0.33(Q_{tn})$

Undrained Shear Strength, $S_u$
$S_u = Q_{tn} \times \sigma'_{V0}/N_{kt}$
$N_{kt}$ is a soil-specific factor (shown on $S_u$ plot)

Sensitivity, $S_t$
$S_t = (q_t - \sigma_{V0}/N_{kt}) \times (1/f_s)$

Effective Friction Angle, $\phi'$
$\phi'(1) = \tan^{-1}(0.373[\log(q_c/\sigma'_{V0}) + 0.29])$
$\phi'(2) = 17.6 + 11[\log(Q_{tn})]$

Unit Weight, $\gamma$
$\gamma = (0.27[\log(F_r)]+0.36[\log(q_t/atm)]+1.236) \times \gamma_{water}$
$\sigma_{V0}$ is taken as the incremental sum of the unit weights

Small Strain Shear Modulus, $G_0$
$G_0\ (1) = \rho V_s^2$
$G_0\ (2) = 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$

Soil Behavior Type Index, $I_c$
$I_c = [(3.47 - \log(Q_{tn}))^2 + (\log(F_r) + 1.22)^2]^{0.5}$

SPT $N_{60}$
$N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817Ic)}$

Elastic Modulus, $E_s$ (assumes $q/q_{ultimate} \sim 0.3$, i.e. FS = 3)
$E_s\ (1) = 2.6\psi G_0$ where $\psi = 0.56 - 0.33\log Q_{tn,clean\ sand}$
$E_s\ (2) = G_0$
$E_s\ (3) = 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$
$E_s\ (4) = 2.5q_t$

Constrained Modulus, M
$M = \alpha_M(q_t - \sigma_{V0})$
For $I_c > 2.2$ (fine-grained soils)
$\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
$\alpha_M = 0.0188 \times 10^{(0.55Ic + 1.68)}$

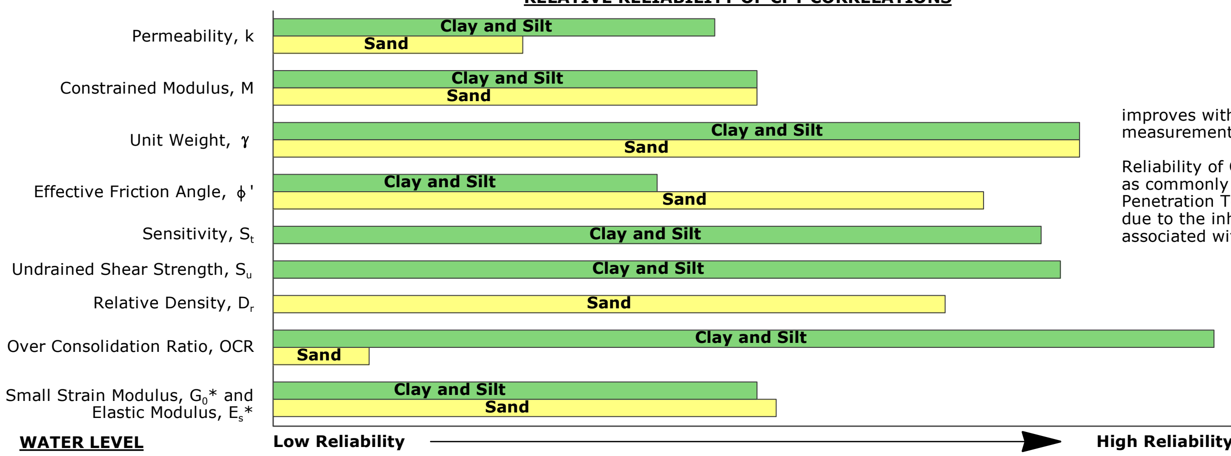
Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04Ic)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37Ic)}$

Relative Density, $D_r$
$D_r = (Q_{tn} / 350)^{0.5} \times 100$

## REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include $q_t$, $f_s$, and u. Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

## RELATIVE RELIABILITY OF CPT CORRELATIONS




improves with seismic $V_s$ measurements

Reliability of CPT-predicted $N_{60}$ values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

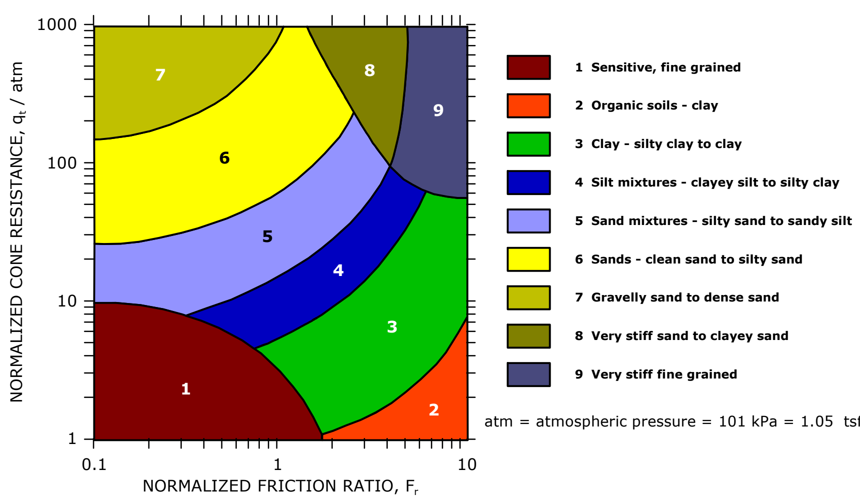
## WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated:"
*Measured - Depth to water directly measured in the field*
*Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions*
While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

## CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance ($q_t$), friction resistance ($f_s$), and porewater pressure ($u_2$). The normalized friction ratio ($F_r$) is used to classify the soil behavior type.

Typically, silts and clays have high $F_r$ values and generate large excess penetration porewater pressures; sands have lower $F_r$'s and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.




atm = atmospheric pressure = 101 kPa = 1.05 tsf

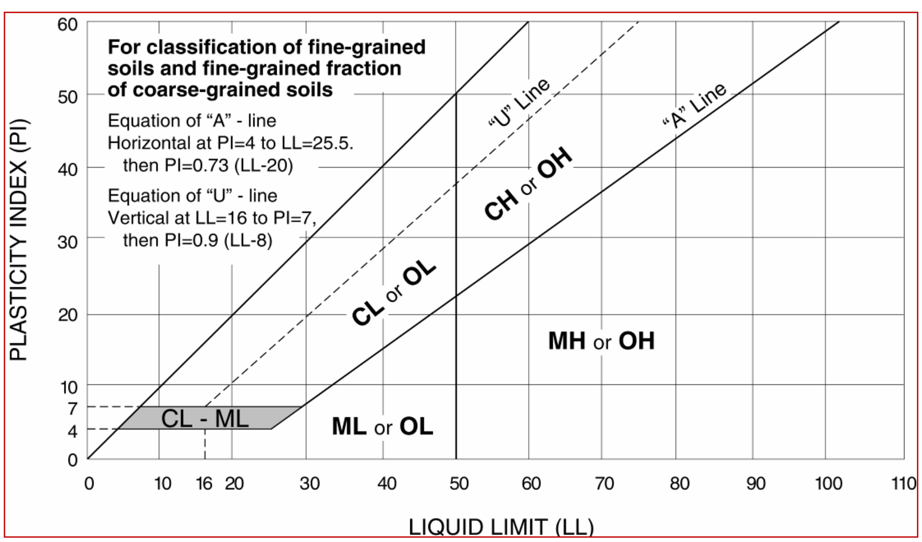
## REFERENCES


Kulhawy, F.H., Mayne, P.W., (1997). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.
Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institue of Technology, Atlanta, GA.
Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.
Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand,"*Journal of the Soil Mechanics and Foundations Division*,96(SM3), 1011-1043.

## Unified Soil Classification System

| Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A] | | | | Soil Classification | |
|---|---|---|---|---|---|
| | | | | Group Symbol | Group Name [B] |
| **Coarse-Grained Soils:** More than 50% retained on No. 200 sieve | **Gravels:** More than 50% of coarse fraction retained on No. 4 sieve | **Clean Gravels:** Less than 5% fines [C] | $Cu \geq 4$ and $1 \leq Cc \leq 3$ [E] | GW | Well-graded gravel [F] |
| | | | $Cu < 4$ and/or $[Cc < 1$ or $Cc > 3.0]$ [E] | GP | Poorly graded gravel [F] |
| | | **Gravels with Fines:** More than 12% fines [C] | Fines classify as ML or MH | GM | Silty gravel [F, G, H] |
| | | | Fines classify as CL or CH | GC | Clayey gravel [F, G, H] |
| | **Sands:** 50% or more of coarse fraction passes No. 4 sieve | **Clean Sands:** Less than 5% fines [D] | $Cu \geq 6$ and $1 \leq Cc \leq 3$ [E] | SW | Well-graded sand [I] |
| | | | $Cu < 6$ and/or $[Cc < 1$ or $Cc > 3.0]$ [E] | SP | Poorly graded sand [I] |
| | | **Sands with Fines:** More than 12% fines [D] | Fines classify as ML or MH | SM | Silty sand [G, H, I] |
| | | | Fines classify as CL or CH | SC | Clayey sand [G, H, I] |
| **Fine-Grained Soils:** 50% or more passes the No. 200 sieve | **Silts and Clays:** Liquid limit less than 50 | **Inorganic:** | PI > 7 and plots above "A" line [J] | CL | Lean clay [K, L, M] |
| | | | PI < 4 or plots below "A" line [J] | ML | Silt [K, L, M] |
| | | **Organic:** | $\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$ | OL | Organic clay [K, L, M, N] |
| | | | | | Organic silt [K, L, M, O] |
| | **Silts and Clays:** Liquid limit 50 or more | **Inorganic:** | PI plots on or above "A" line | CH | Fat clay [K, L, M] |
| | | | PI plots below "A" line | MH | Elastic silt [K, L, M] |
| | | **Organic:** | $\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$ | OH | Organic clay [K, L, M, P] |
| | | | | | Organic silt [K, L, M, Q] |
| **Highly organic soils:** | Primarily organic matter, dark in color, and organic odor | | | PT | Peat |

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



Terracon Consultants Inc
1220 Concord Ave
Concord, CA 94520
https://www.terracon.com/

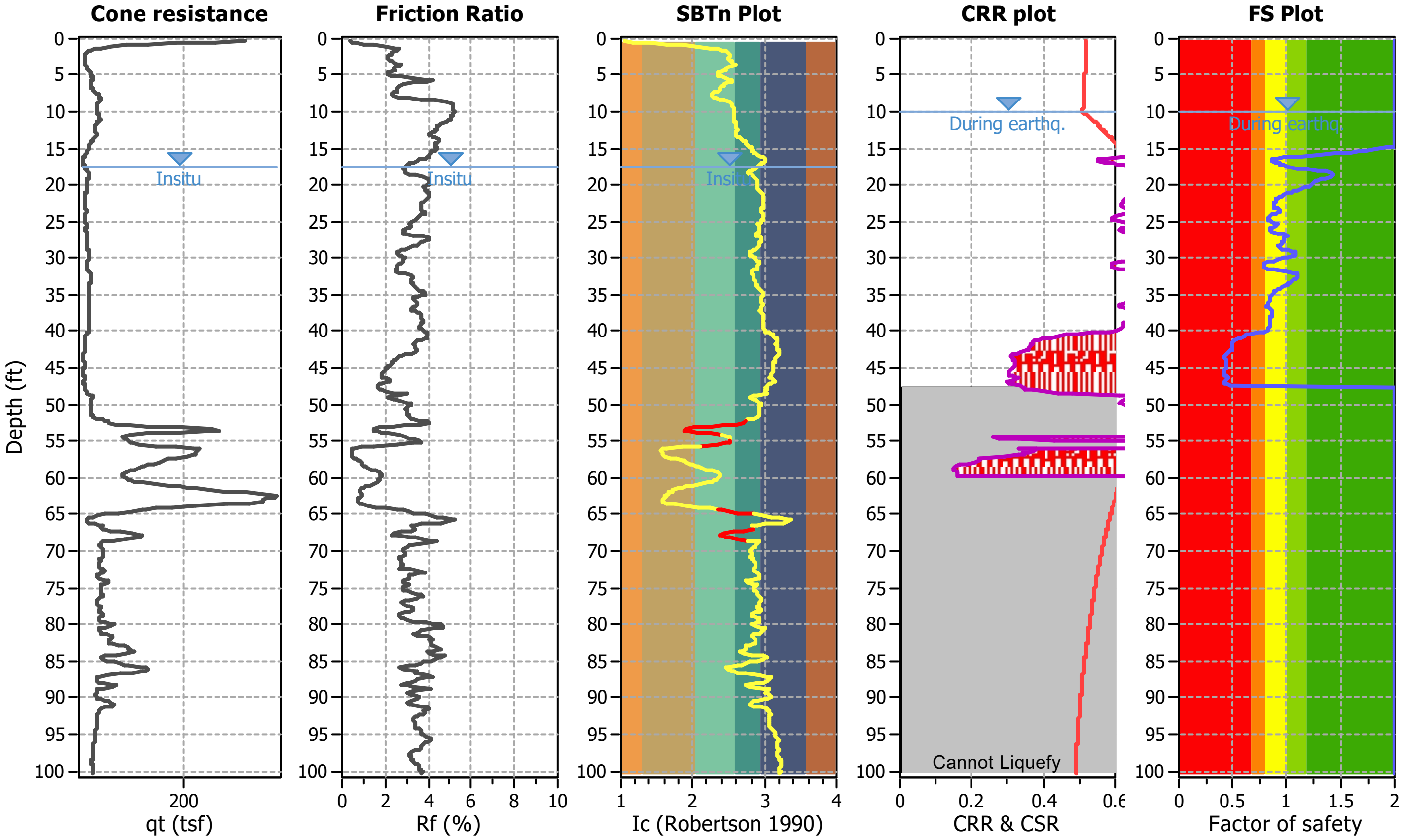
# LIQUEFACTION ANALYSIS REPORT

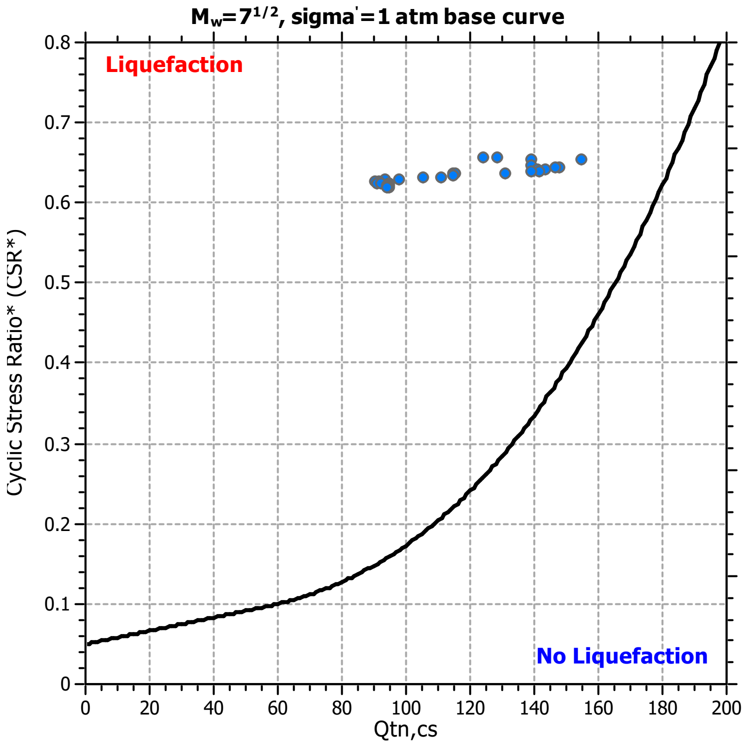
**Project title : Sherman Recovery Center** **Location :**

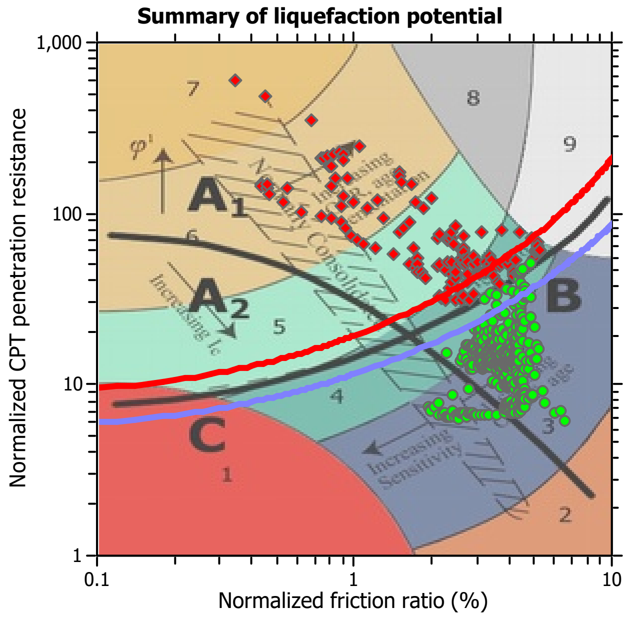
**CPT file : CPT-01**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 17.60 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 10.00 ft | Fill height: | N/A | Limit depth applied: | Yes |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | 60.00 ft |
| Earthquake magnitude $M_w$: | 6.67 | Ic cut-off value: | 2.60 | Trans. detect. applied: | Yes | MSF method: | Method based |
| Peak ground acceleration: | 0.83 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | No | | |

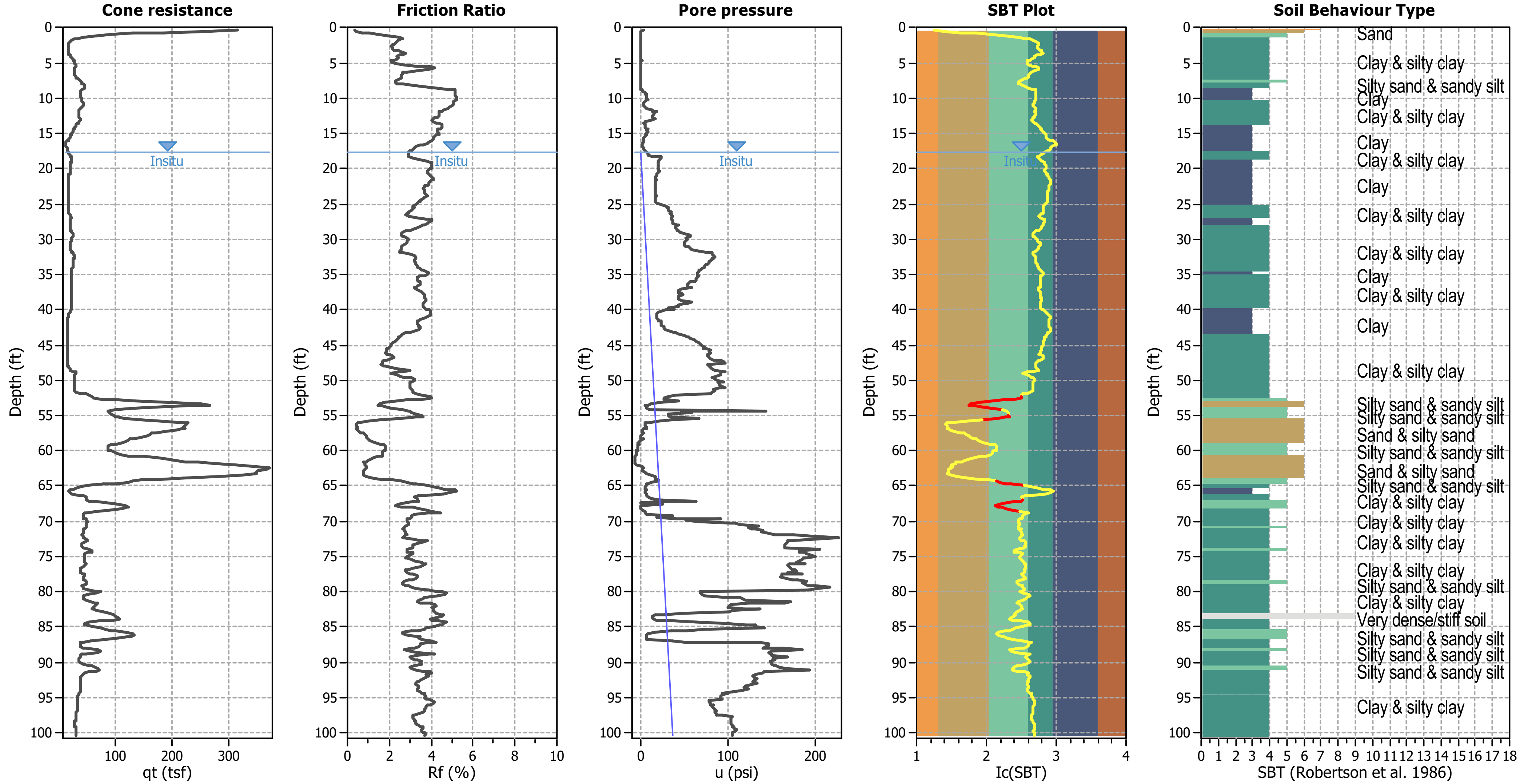



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction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

# CPT basic interpretation plots



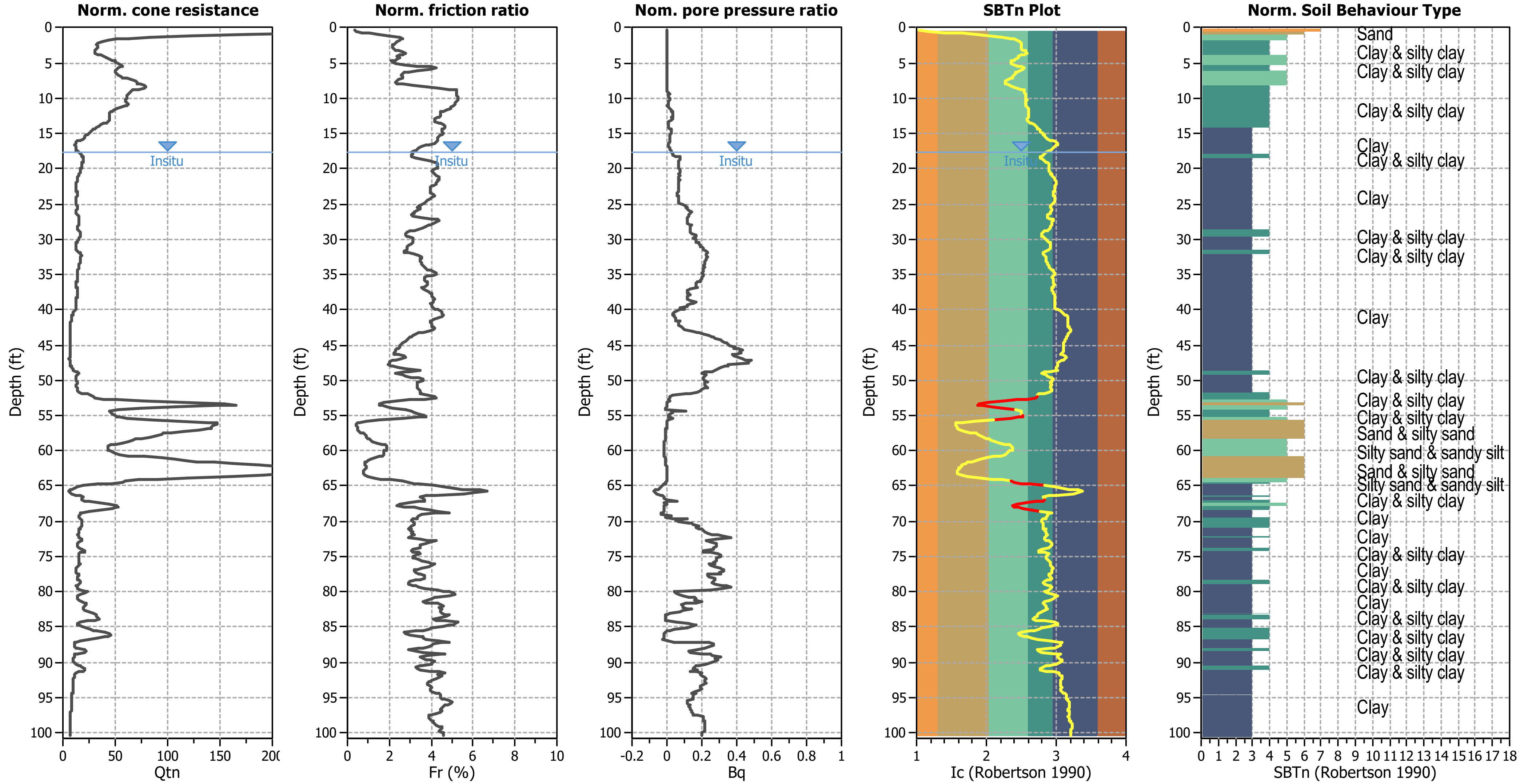

## Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 6.67 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.83 | Use fill: | No | Limit depth applied: | Yes |
| Depth to water table (insitu): | 17.60 ft | Fill height: | N/A | Limit depth: | 60.00 ft |

## SBT legend

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

## CPT basic interpretation plots (normalized)



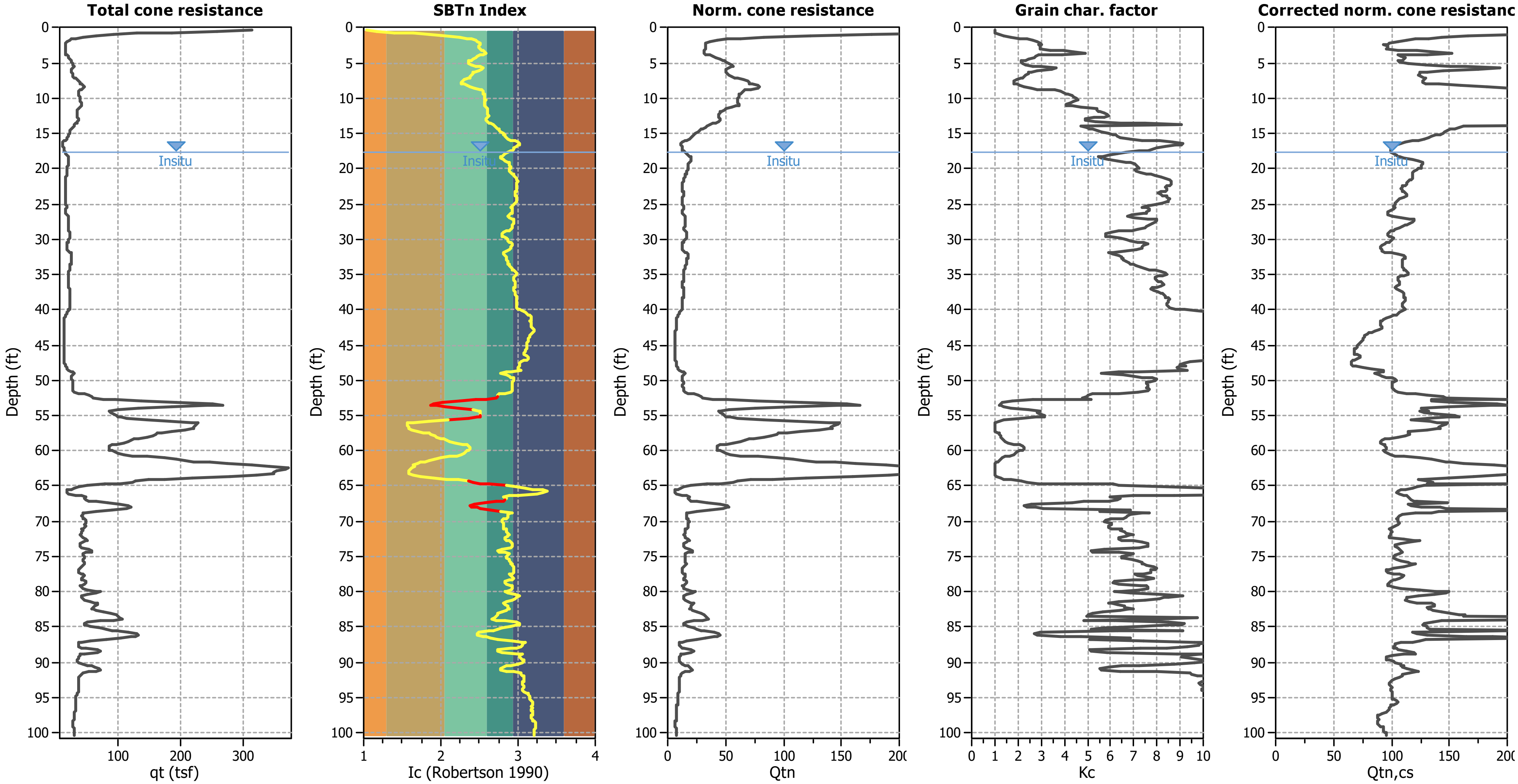

### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 6.67 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.83 | Use fill: | No | Limit depth applied: | Yes |
| Depth to water table (insitu): | 17.60 ft | Fill height: | N/A | Limit depth: | 60.00 ft |

### SBTn legend

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

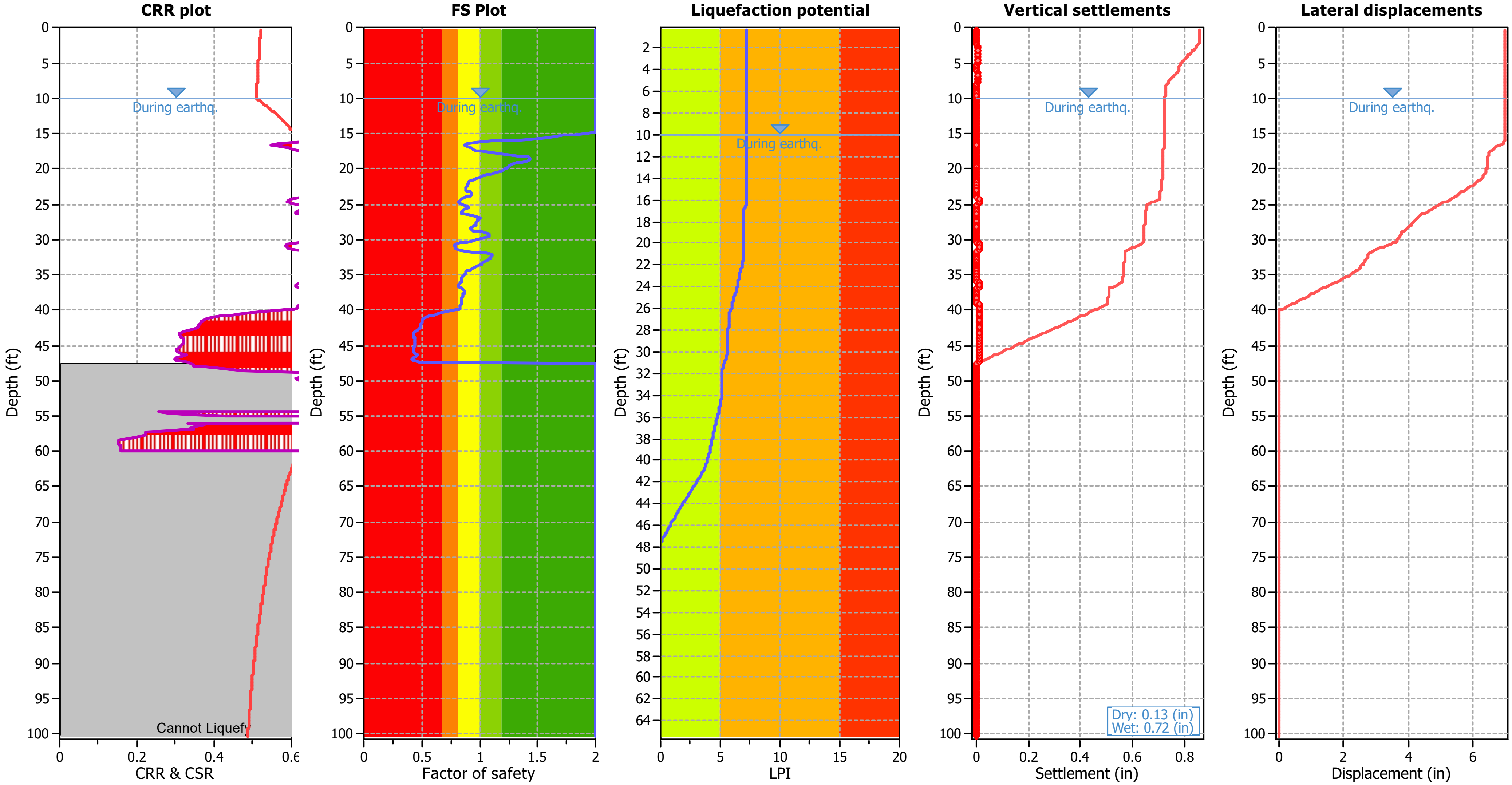
# Liquefaction analysis overall plots (intermediate results)




## Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 6.67 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.83 | Use fill: | No | Limit depth applied: | Yes |
| Depth to water table (insitu): | 17.60 ft | Fill height: | N/A | Limit depth: | 60.00 ft |

## Liquefaction analysis overall plots




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 6.67 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.83 | Use fill: | No | Limit depth applied: | Yes |
| Depth to water table (insitu): | 17.60 ft | Fill height: | N/A | Limit depth: | 60.00 ft |

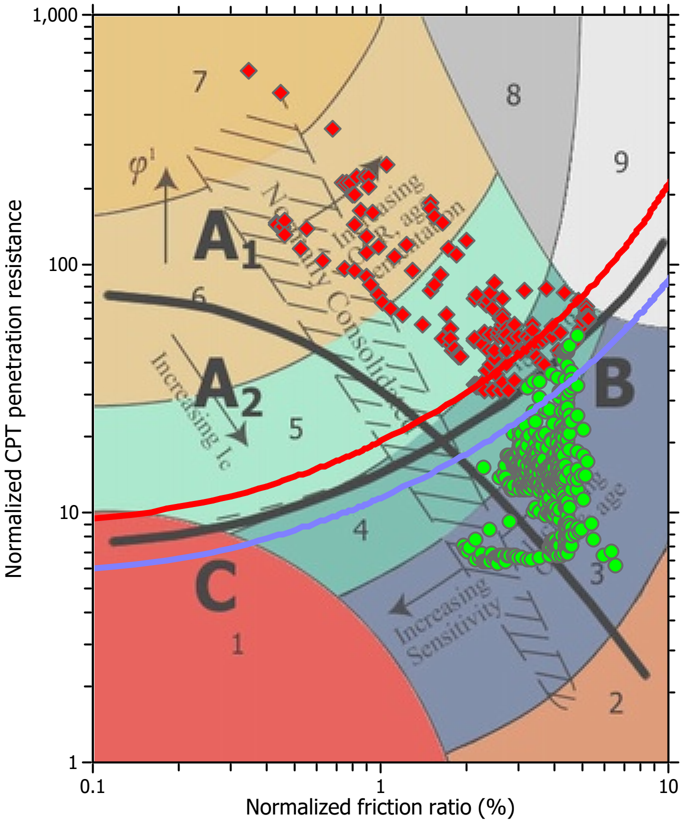
**F.S. color scheme**

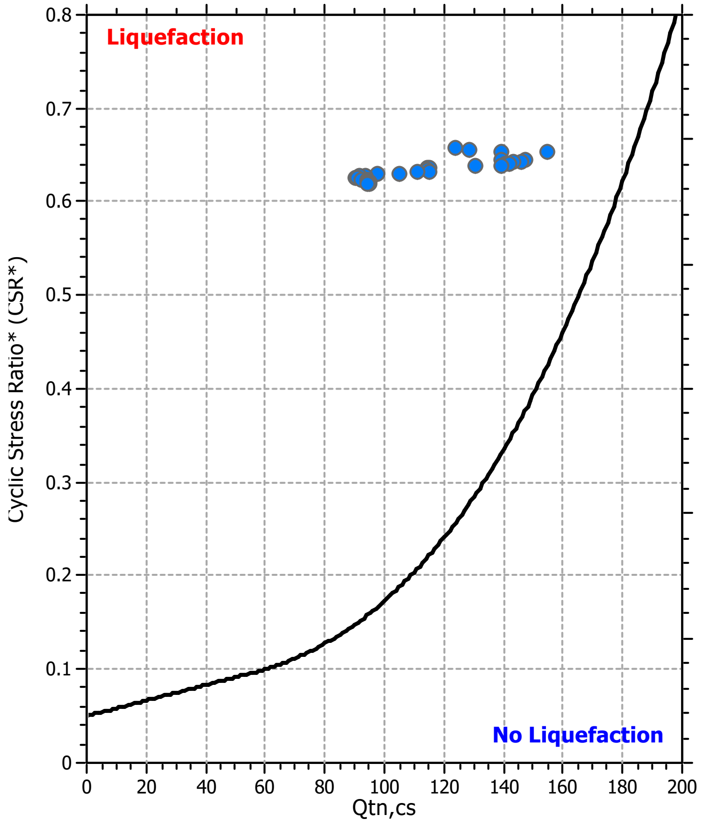
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

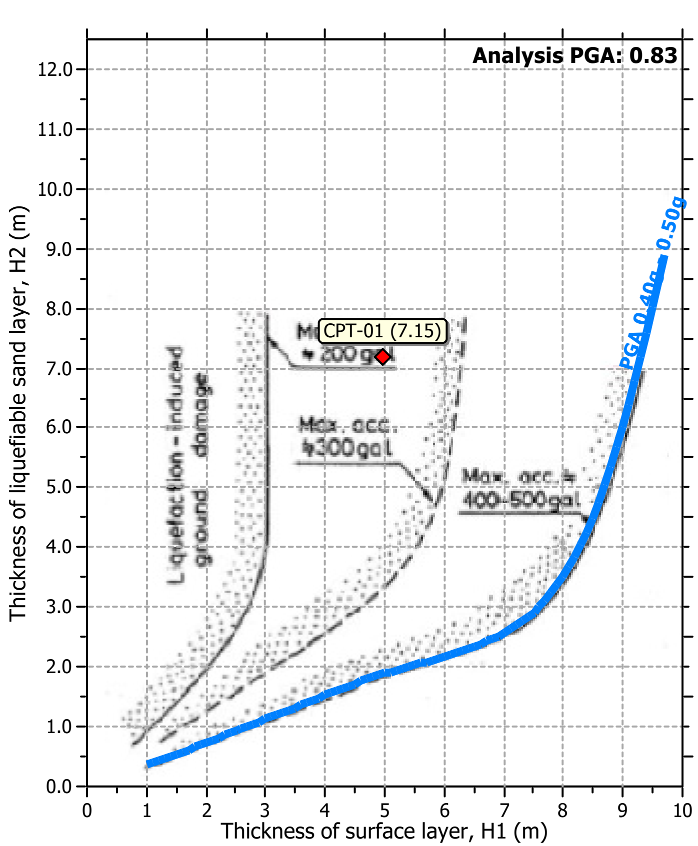
**LPI color scheme**

- Very high risk
- High risk
- Low risk

# Liquefaction analysis summary plots



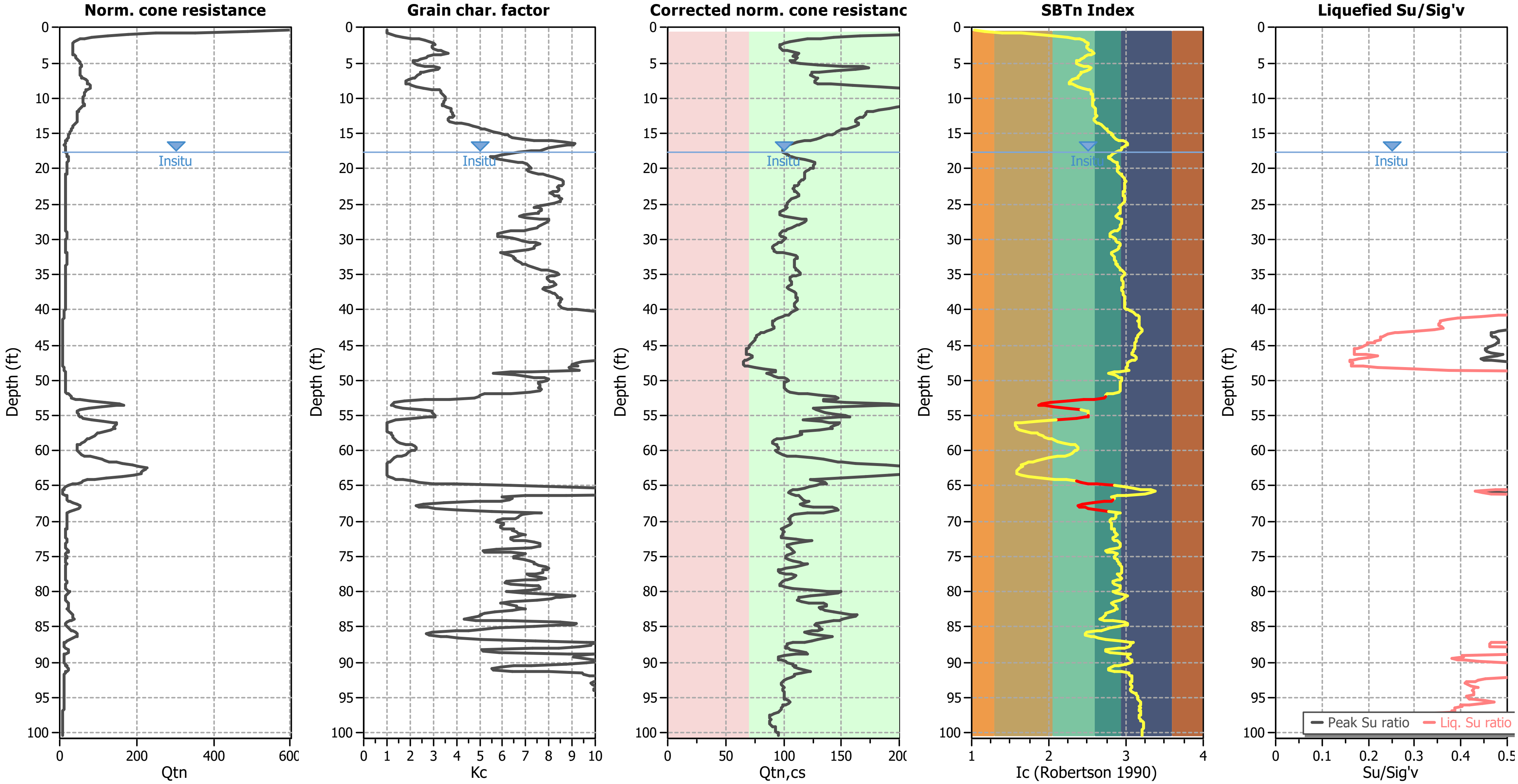




## Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_{\sigma}$ applied: | No |
| Earthquake magnitude $M_w$: | 6.67 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.83 | Use fill: | No | Limit depth applied: | Yes |
| Depth to water table (insitu): | 17.60 ft | Fill height: | N/A | Limit depth: | 60.00 ft |

# Check for strength loss plots (Robertson (2010))




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 10.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | Yes |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 6.67 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.83 | Use fill: | No | Limit depth applied: | Yes |
| Depth to water table (insitu): | 17.60 ft | Fill height: | N/A | Limit depth: | 60.00 ft |