# APPENDIX E

# GEOTECHNICAL REPORT

Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

# GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT

Santa Ana River Bottom (SARB) Maintenance Facility
4600 Crestmore Road
City of Riverside, Riverside County, California

*CONVERSE PROJECT NO. 23-81-234-01*

*Prepared For:*
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*Presented By:*
CONVERSE CONSULTANTS
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November 16, 2023

November 16, 2023

Mr. David M. Beckwith, PE, PLS, QSD, QSP, QISP
CEO/President
David Beckwith and Associates, Inc.
9431 Haven Avenue, Suite 232
Rancho Cucamonga, CA 91730

Subject: **GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT**
**Santa Ana River Bottom (SARB) Maintenance Facility**
4600 Crestmore Road
City of Riverside, Riverside County, California
Converse Project No. 23-81-234-01

Dear Mr. Beckwith:

Converse Consultants (Converse) is pleased to submit this Geotechnical Investigation and Water Percolation Test Report to assist with the design and construction of the proposed maintenance yard and other associated improvements located at 4600 Crestmore Road in the City of Riverside, Riverside County, California. This report was prepared in accordance with our proposal dated July 21, 2023, and our Acceptance of Agreement and Authorization to Proceed signed by you and dated September 9, 2023.

Based upon our field investigation, laboratory data, and analyses, the proposed project is considered feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and construction of the project.

We appreciate the opportunity to be of service to David Beckwith and Associates, Inc., and Riverside County Parks and Open Space District. Should you have any questions, please do not hesitate to contact me at 909-474-2847.

**CONVERSE CONSULTANTS**

Hashmi S. E. Quazi, PhD, PE, GE
Principal Engineer

Dist.: 1-Electronic-Pdf/ Addressee
AZ/CN/RLG/HSQ/kvg

## PROFESSIONAL CERTIFICATION

This report has been prepared by the following professionals whose seals and signatures appear herein.

The findings, recommendations, specifications and professional opinions contained in this report were prepared in accordance with the generally accepted professional engineering and engineering geologic principle and practice in this area of Southern California. We make no other warranty, either expressed or implied.

Catherine Nelson

Aleksey Zhukov
Staff Engineer

Catherine Nelson, GIT
Senior Staff Geologist

Hashmi S. E. Quazi, PhD, PE, GE
Principal Engineer

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## APPENDICES

# 1.0 INTRODUCTION

This report presents the results of our geotechnical investigation and water percolation testing performed for the Santa Ana River Bottom (SARB) Maintenance Facility located at 4600 Crestmore Road in the City of Riverside, Riverside County, California. The project location is shown in Figure No. 1, *Approximate Site Location Map*.

The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils, and to provide design and construction recommendations for the project.

This report is prepared for the project described herein and is intended for use solely by David Beckwith and Associates, Inc., Riverside County Parks and Open Space District, and their authorized agents for design purposes. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

# 2.0 PROJECT DESCRIPTION

The proposed project will include the following.

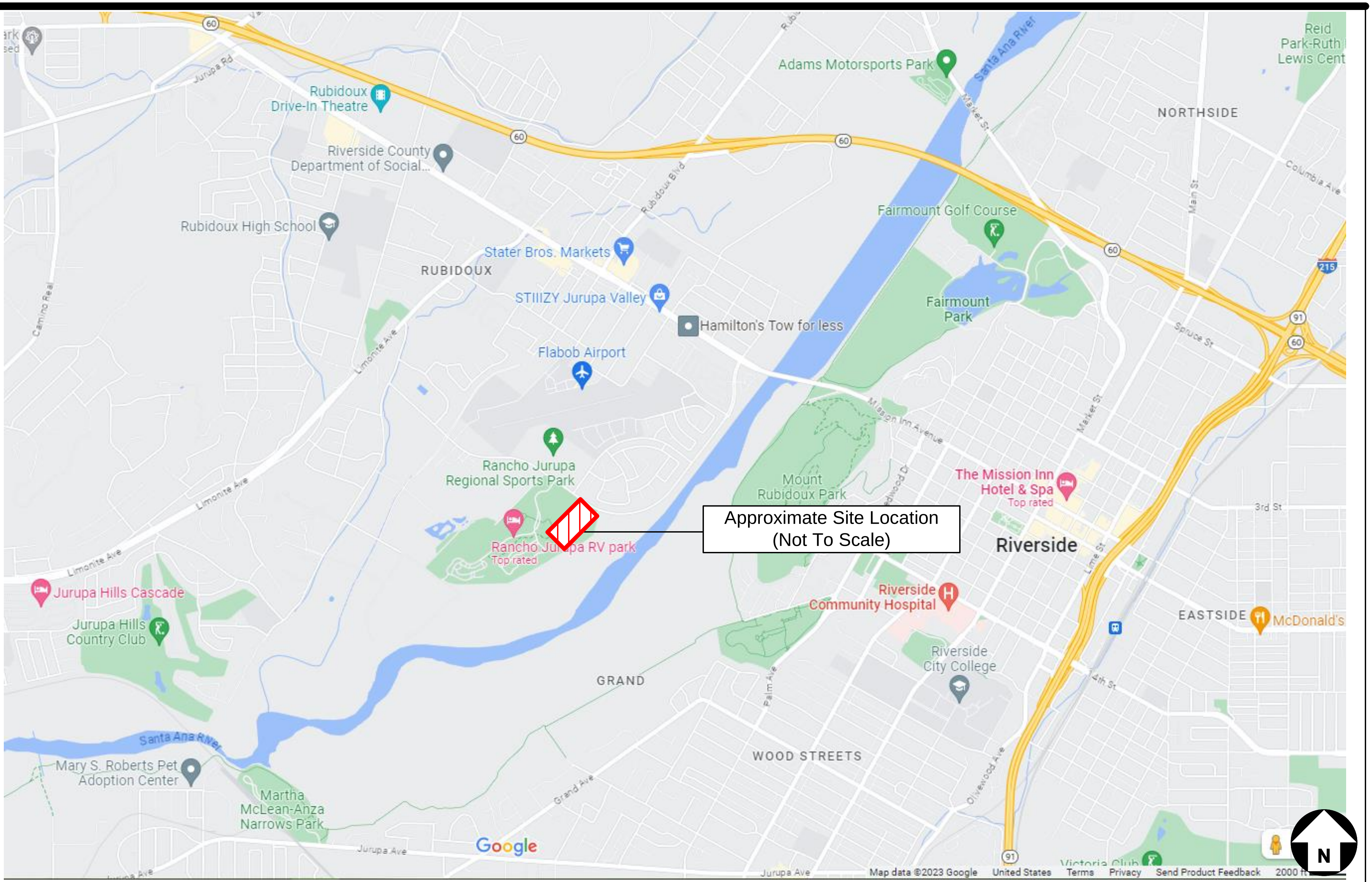
Demolition and Grading
Before construction the site will undergo necessary demolition and grading to create a suitable foundation. It will include the following.

- Level and grade of the SARB building location
- Maintenance road, approximately 1,129 feet long
- Maintenance yard
- Remove the fence line around the D Building area

Class II Base
Create a stable and durable surface for the maintenance building, surrounding area and maintenance road by the following.

- Maintenance Road with 6 -inches of Class II Base
- Maintenance Yard with 6-inches of Class II Base

Cement Slab
Installation of concrete slabs for the maintenance building and hazmat area. It will include the following.




Project: Santa Ana River Bottom (SARB) Maintenance Facility

Location: 4600 Crestmore Road
City of Riverside, Riverside County, California

For: David Beckwith and Associates, Inc.

# Approximate Site Location Map

Project No.
**23-81-234-01**

**Converse Consultants**

Figure No.
**1**

- 60' x 60' concrete slab at least 6 inches with No. 4 rebar at 18-inch interval from the center, 4,000 psi concrete mix
- 10' x 12' concrete slab to the left of the building with 4-inches thick with microfiber mesh

Maintenance Building
- Approximately 2,400 (60' x 40') square-foot maintenance building to accommodate the SARB unit and their day-to-day needs. The building will be CMU concrete blocks or comparable cost effective and sturdy material
- Two bay doors at least 10' by 10' to accommodate heavy equipment
- Security camera
- Electrical panel
- Eye wash station

Hazmat Area
- 10' x 12' x 10' metal canopy to accommodate hazardous materials and sustain windy conditions
- Spill containment workstation with ramp

Security Fencing
- Approximately 800 linear feet of CMU block wall around the maintenance yard
- Approximately 2,700 linear feet from Maintenance yard to existing fence along Rancho Jurupa Park
- Approximately 16 feet wide gate at the rear

Utilities
- Water line from Building D or where feasible
- Connection for fire hydrant
- Electrical line
- Sewer Line to Building D and maintenance yard
- Broadband/internet

Retrofitting Building D
- Demolition of walls to fit shower, toilet, and sink
- Minimum 15 size lockers
  Security cameras to observe the yard

## 3.0 SITE DESCRIPTION

The proposed project site is located on and surrounding the existing Crestmore Manor property located at 4600 Crestmore Road, Riverside, California. It is bounded on the west by Rancho Jurupa Regional Park and on the north, east and south by vacant land.

Currently the proposed project site is being utilized as the Riverside County Regional Parks and Open Space District Headquarters Facility. The existing structures include a large main office/event facility surrounded by manicured grounds, a large parking lot, and a few small outbuildings all secured by standard chain link fencing. The proposed improvements will be located to the south and east of the existing buildings and parking lot. These areas are covered with a sparse to moderate layer of weeds and grasses, with a few large trees and shrubs scattered around. Some proposed project areas can be accessed by an access road around the back of the facility and some need to be accessed through locked gates or through the Rancho Jurupa Regional Park RV Resort. The proposed project area is relatively flat with surface elevations ranging from 752 feet to 754 feet above mean sea level (amsl). Surface drainage at the project site flows to the south.

Photograph Nos. 1 and 2 show the current conditions at the site.

*Photograph No. 1: Present site conditions, facing northeast.*

*Photograph No. 2: Present site conditions, facing southwest.*

# 4.0 SCOPE OF WORK

The scope of this investigation included project set-up, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report, as described in the following sections.

## *4.1 Project Set-up*

As part of the project set-up, our staff will conduct the following.

- Plotted the proposed boring locations on an exhibit and submitted for your review and approval.
- Conducted a site reconnaissance to stake the boring locations to ensure that drill rig access to all the locations was available.
- Notified Underground Service Alert (USA) 48 hours prior to drilling to clear the boring's locations of any conflict with existing underground utilities.
- Engaged a California Licensed drill rig to drill the borings.

## *4.2 Subsurface Exploration*

Eight exploratory borings (BH-01 through BH-08) were drilled on September 9, 2023, to investigate the subsurface conditions at the site. The borings were drilled using an 8-inch diameter hollow stem auger to the maximum explored depth of 51.5 feet below existing ground surface (bgs).

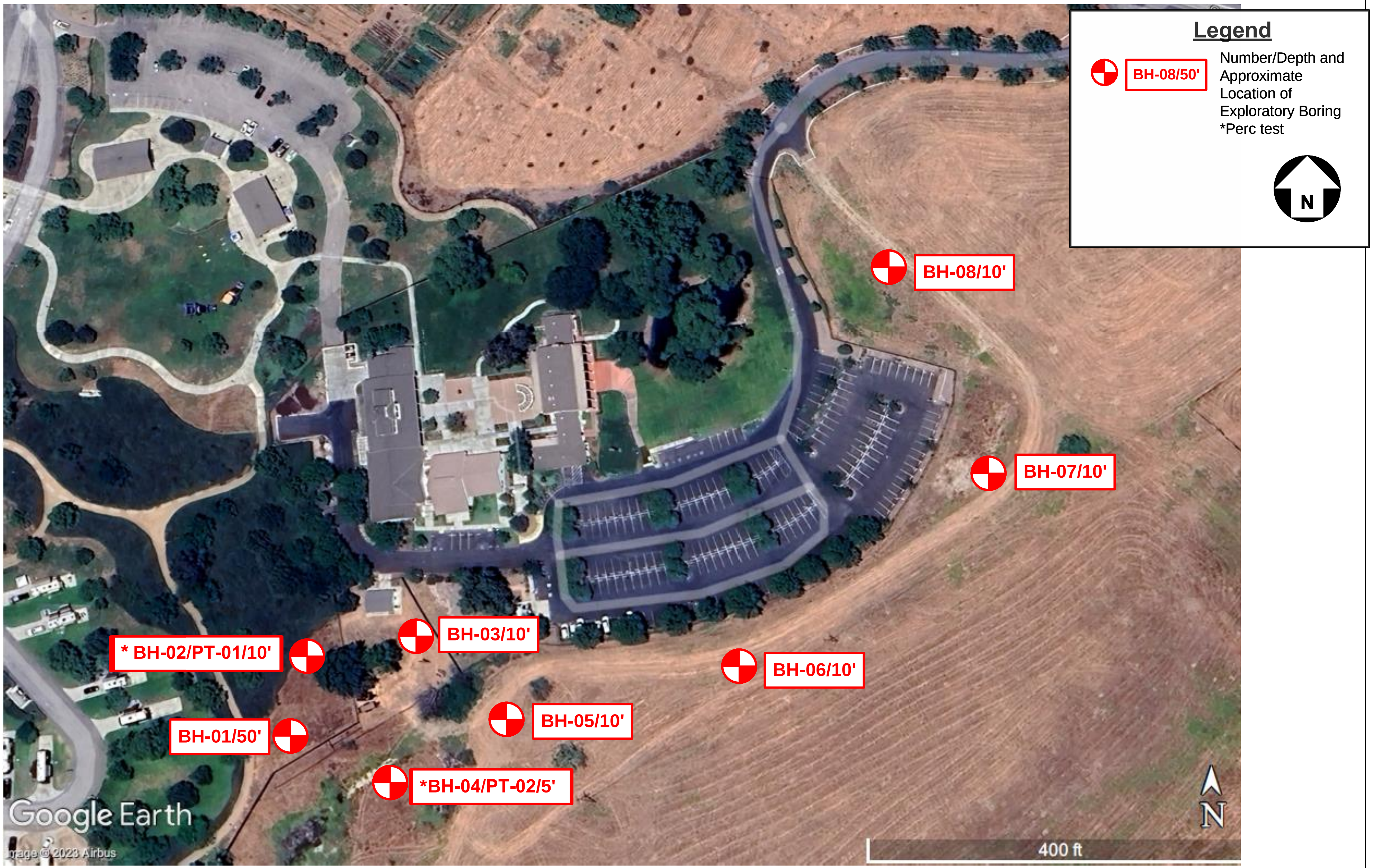
Two borings were utilized to perform percolation testing (BH-02/PT-01 and BH-04/PT-02). The depth of the percolation tests were 10.2 and 5.7 feet bgs, respectively.

Approximate boring and percolation test locations are indicated in Figure No. 2, *Approximate Borings and Percolation Test Locations Map*. For a description of the field exploration and sampling program, see Appendix A, *Field Exploration*.

## *4.3 Laboratory Testing*

Representative soil samples from the project site were tested in the laboratory to aid in the soils classification and to evaluate the relevant engineering properties of the soil. These tests included the following.

- *In*-situ moisture contents and dry densities (ASTM D2216 and ASTM D2937)
- Expansion index (ASTM D4829)
- R-value (California Test 301)
- Soil corrosivity (California Tests 643, 422, and 417)
- Grain size distribution (ASTM D6913)




Project: Santa Ana River Bottom (SARB) Maintenance Facility

Location: 4600 Crestmore Road
City of Riverside, Riverside County, California

For: David Beckwith and Associates, Inc.

# Approximate Boring and Percolation Test Locations Map

Project No.
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**Converse Consultants**

Figure No.
**2**

- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

For *in-situ* moisture and dry density data, see the Logs of Borings in Appendix A, *Field Exploration*. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program.*

### *4.4 Analysis and Report Preparation*

Data obtained from the field exploration and laboratory testing program was assembled and evaluated. Geotechnical analyses of the compiled data were performed, followed by the preparation of this report to present our findings, conclusions, and recommendations for the project.

## 5.0 SITE CONDITIONS

A general description of the surface and subsurface conditions, various materials and groundwater conditions encountered at each location during our field exploration is discussed below.

### *5.1 Subsurface Profile*

Based on our field exploration and laboratory test results, the subsurface soil at the project site consisted of alluvium. This alluvium is Holocene to late Pleistocene-aged axial-channel deposits and was encountered in all of the exploratory borings from the surface to the maximum drilled depths ranging from 6.5 feet to 51.5 feet bgs. This material was generally comprised of sand, sand with silt, silty sand, clayey sand, sandy clay, and sandy silt, which was fine-grained in most units, but was also fine to medium-grained and fine to coarse-grained in some units, had some pinhole porosity, oxidation staining, and trace caliche, was slightly to very desiccated, loose/soft to medium dense/medium stiff, moist and various shades of brown, orange, gray and green. Interbedded/alternating layers of varying material were encountered in each boring from 0.5 inches to 6.0 inches thick. As a result of the project site being located in a historic river channel and current regulatory floodway, subsurface soil conditions with a wide variation of characteristics are expected.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-9, *Logs of Borings*, in Appendix A, *Field Exploration.*

### *5.2 Groundwater*

Groundwater was encountered during the investigation in BH-01 at a depth of 13.7 feet bgs. For comparison, national and regional groundwater databases were accessed as detailed below.

Regional groundwater data from the GeoTracker database (SWRCB, 2023) was reviewed to evaluate the current and historical groundwater levels. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Data from that record is detailed below.

- Poly-Fiber Facility (SL0606515945) is located approximately 3,520 feet north-northwest of the project site. Groundwater was reported at this site at a depth of 10.0 feet bgs in 2007.

Regional groundwater data from the USGS National Water Information System (USGS, 2023a) was reviewed to evaluate the current and historical groundwater levels. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Data from that record is detailed below.

**Table No. 1, Summary of USGS Groundwater Depth Data**

| Site No. | Location | Groundwater Depth Range (ft. bgs) | Date Range |
|---|---|---|---|
| 335843117243502 | South of Rancho Jurupa Regional Park; approx. 755 feet southwest of the project site | 11.03 | 2001 |
| 335843117243501 | South of Rancho Jurupa Regional Park; approx. 755 feet southwest of the project site | 11.08 | 2001 |

Regional groundwater data from the California Department of Water Resources database (DWR, 2023) was reviewed to evaluate the current and historical groundwater levels. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Data from that record is detailed below.

- Flory (Station 339950N1174230W001), located approximately 5,000 feet north of the project site, reported groundwater at depths ranging from 66.34 to 72.88 feet bgs between 2011 and 2023.

Based on the depth at which groundwater was encountered during our investigation, as well as current and historical data in the vicinity of the project site, groundwater is expected to be deeper than 13.7 feet bgs. Groundwater is not expected to be encountered during construction. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in each site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

### *5.3 Expansive Soils*

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade. Depending on the extent and location below finish subgrade, expansive soils can have a detrimental effect on structures.

Based on the laboratory test results, the expansion index was 0, corresponding to very low expansion potential.

### *5.4 Excavatability*

The surface and subsurface soil materials at the site are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. However, excavation will be difficult if a concentration of gravel is encountered.

The phrase “conventional heavy-duty excavation equipment” is intended to include commonly used equipment such as excavators and trenching machines. It does not include hydraulic hammers (“breakers”), jackhammers, blasting, or other specialized equipment and techniques used to excavate hard earth materials. Selection of an appropriate excavation equipment model should be done by an experienced earthwork contractor and may require test excavations in representative areas.

### *5.5 Subsurface Variations*

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations.

### *5.6 Flooding*

Review of National Flood Insurance Rate Maps (FEMA, 2023) indicates that the project site is within an area defined as a Flood Hazard Zone “AE” with the criteria of “Regulatory Floodway”. Areas with this zone determination are considered Special Hazard Floodways.

# 6.0 ENGINEERING GEOLOGY

The regional and local geology within the proposed project area is discussed below.

## *6.1 Regional Geology*

The proposed project site is located within the northern Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the southwest by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto, Elsinore, and San Andreas fault zones, all of which have been known to be active during Quaternary time.

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This northwest-trending linear fabric is created by the regional faulting within the granitic basement rock of the Southern California Batholith. Broad, linear, alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

The proposed project site is located within the north-central portion of the Perris Block region of the Peninsular Ranges province. The Perris Block is a relatively stable structural block bounded by the active Elsinore and San Jacinto fault zones to the west and east, and the Chino and Temecula basins to the north and south, respectively. The Perris Block has low relief and is roughly rectangular in shape.

## *6.2 Local Geology*

Review of geologic mapping indicates that the project site is underlain locally by young (Holocene and late Pleistocene aged) axial-channel deposits. These deposits consist of slightly to moderately consolidated silt, sand, and gravel (Morton and Miller, 2006).

# 7.0 FAULTING AND SEISMICITY

The approximate distance and seismic characteristics of nearby faults as well as seismic design coefficients are presented in the following subsections.

### *7.1 Faulting*

The proposed site is situated in a seismically active region. As is the case for most areas of Southern California, ground-shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site. Review of recent seismological and geophysical publications indicates that the seismic hazard for the project is high.

The project site is not located within a currently mapped State of California Earthquake Fault Zone for surface fault rupture. Table No. 2, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 100 kilometers of the site. The data presented below was calculated using the National Seismic Hazard Maps Database (USGS, 2008) and other published geologic data.

**Table No. 2, Summary of Regional Faults**

| Fault Name and Section | Closest Distance (km) | Slip Sense | Length (km) | Slip Rate (mm/year) | Maximum Magnitude |
|---|---|---|---|---|---|
| San Jacinto | 14.66 | strike slip | 241 | n/a | 7.88 |
| Cucamonga | 21.64 | thrust | 28 | 5 | 6.70 |
| Chino, alt 2 | 22.26 | strike slip | 29 | 1 | 6.80 |
| Chino, alt 1 | 22.46 | strike slip | 24 | 1 | 6.70 |
| Elsinore | 22.59 | strike slip | 241 | n/a | 7.85 |
| S. San Andreas | 24.75 | strike slip | 548 | n/a | 8.18 |
| San Jose | 29.87 | strike slip | 20 | 0.5 | 6.70 |
| Cleghorn | 33.94 | strike slip | 25 | 3 | 6.80 |
| Sierra Madre Connected | 34.41 | reverse | 76 | 2 | 7.30 |
| Sierra Madre | 34.41 | reverse | 57 | 2 | 7.20 |
| North Frontal (West) | 39.51 | reverse | 50 | 1 | 7.20 |
| Puente Hills (Coyote Hills) | 43.35 | thrust | 17 | 0.7 | 6.90 |
| San Joaquin Hills | 47.78 | thrust | 27 | 0.5 | 7.10 |
| Clamshell-Sawpit | 49.53 | reverse | 16 | 0.5 | 6.70 |
| Puente Hills (Santa Fe Springs) | 56.53 | thrust | 11 | 0.7 | 6.70 |
| Raymond | 57.32 | strike slip | 22 | 1.5 | 6.80 |
| Newport Inglewood Connected alt 2 | 63.24 | strike slip | 208 | 1.3 | 7.50 |
| Newport Inglewood Connected alt 1 | 63.27 | strike slip | 208 | 1.3 | 7.50 |
| Newport-Inglewood (Offshore) | 63.27 | strike slip | 66 | 1.5 | 7.00 |

| Fault Name and Section | Closest Distance (km) | Slip Sense | Length (km) | Slip Rate (mm/year) | Maximum Magnitude |
|---|---|---|---|---|---|
| Newport-Inglewood, alt 1 | 63.43 | strike slip | 65 | 1 | 7.20 |
| Pinto Mtn | 64.15 | strike slip | 74 | 2.5 | 7.30 |
| Elysian Park (Upper) | 64.54 | reverse | 20 | 1.3 | 6.70 |
| Puente Hills (LA) | 66.26 | thrust | 22 | 0.7 | 7.00 |
| Helendale-So Lockhart | 66.6 | strike slip | 114 | 0.6 | 7.40 |
| North Frontal (East) | 68.7 | thrust | 27 | 0.5 | 7.00 |
| Verdugo | 70.74 | reverse | 29 | 0.5 | 6.90 |
| Hollywood | 77.36 | strike slip | 17 | 1 | 6.70 |
| Palos Verdes Connected | 81.22 | strike slip | 285 | 3 | 7.70 |
| Palos Verdes | 81.22 | strike slip | 99 | 3 | 7.30 |
| Lenwood-Lockhart-Old Woman Springs | 81.67 | strike slip | 145 | 0.9 | 7.50 |
| Santa Monica Connected alt 2 | 82.04 | strike slip | 93 | 2.4 | 7.40 |
| Sierra Madre (San Fernando) | 88.16 | thrust | 18 | 2 | 6.70 |
| San Gabriel | 88.61 | strike slip | 71 | 1 | 7.30 |
| Johnson Valley (No) | 89.8 | strike slip | 35 | 0.6 | 6.90 |
| Coronado Bank | 90.87 | strike slip | 186 | 3 | 7.40 |
| Burnt Mtn | 92.46 | strike slip | 21 | 0.6 | 6.80 |
| Landers | 92.87 | strike slip | 95 | 0.6 | 7.40 |
| Santa Monica Connected alt 1 | 93.12 | strike slip | 79 | 2.6 | 7.30 |
| Santa Monica, alt 1 | 93.12 | strike slip | 14 | 1 | 6.60 |
| Eureka Peak | 94.99 | strike slip | 19 | 0.6 | 6.70 |
| Rose Canyon | 95.15 | strike slip | 70 | 1.5 | 6.90 |
| Northridge | 96.54 | thrust | 33 | 1.5 | 6.90 |

(Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/)

### *7.2 CBC Seismic Design Parameters*

Seismic parameters based on the 2022 California Building Code (CBSC, 2022) and ASCE 7-16 are provided in the following table. These parameters were determined using the generalized coordinates for the location and the Seismic Design Maps ATC online tool.

**Table No. 3, CBC Seismic Design Parameters**

| Seismic Parameters | |
|---|---|
| Site Coordinates | 33.9805 N, 117.4105 W |
| Site Class | D |
| Risk Category | II* |
| Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_S$ | 1.500g |
| Mapped 1-second Spectral Response Acceleration, $S_1$ | 0.600g |
| Site Coefficient (from Table 11.4-1), $F_a$ | 1.0 |
| Site Coefficient (from Table 11.4-2), $F_v$ | 1.7 |
| MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$ | 1.500g |
| MCE 1-second period Spectral Response Acceleration, $S_{M1}$ | 1.020g |
| Design Spectral Response Acceleration for short period $S_{DS}$ | 1.000g |
| Design Spectral Response Acceleration for 1-second period, $S_{D1}$ | 0.680g |
| Site Modified Maximum Peak Ground Acceleration, $PGA_M$ | 0.550g |

* Risk category may increase to III or IV based on quantities of hazardous materials.

### *7.3 Secondary Effects of Seismic Activity*

In general, secondary effects of seismic activity include surface fault rupture, soil liquefaction, landslides, lateral spreading, and settlement due to seismic shaking, tsunamis, seiches, and earthquake-induced flooding. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture: The project site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2023). There are no known active faults projecting toward or extending across the project site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

Liquefaction: Liquefaction is defined as the phenomenon in which a cohesionless soil mass within the upper 50.0 feet of the ground surface suffers a substantial reduction in its shear strength, due to the improvement of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction generally occurs in submerged granular soils and non-plastic silts during or after strong ground shaking. There are several general requirements for liquefaction to occur and they are as follows.

- Soils must be submerged.
- Soils must be loose to medium-dense.
- Ground motion must be intense.
- Duration of shaking must be sufficient for the soils to lose shear resistance.

Based on review of hazard maps, the project site is located within an area not evaluated for liquefaction by State of California (CGS, 2007). However, the project site is located within a Riverside County liquefaction zone determined to have a liquefaction potential of very high (Riverside County, 2023). A site-specific liquefaction analysis is presented in Appendix C, *Liquefaction and Settlement Analyses*, the potential for liquefaction included settlement at the site is expected to be up to 3.26 inches.

Seismic Settlement: Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. Based on a site-specific liquefaction analysis presented in Appendix C, *Liquefaction and Settlement Analyses*, the project site has the potential for up to 0.99 inches of dry seismic settlement.

Landslides: Seismically induced landslides and slope failures are common occurrences during or soon after large earthquakes. Due to the flat nature of the site, the potential for seismically induced landslides affecting the proposed site is considered to be low.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over underlying materials which are liquefied due to ground shaking. It differs from slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Due to the relatively flat nature of the site, the risk of lateral spreading is considered very low.

Tsunamis: Tsunamis are large waves generated in open bodies of water by fault displacement or major ground movement. Due to the inland location of the site, tsunamis are not considered to be a risk.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Due to the distance from any enclosed bodies of water, seiching is not considered to be a risk.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is not located within a State of California or Riverside County designated dam inundation zone (DSOD, 2023) The risk for earthquake-induced flooding to affect the project site is considered low.

# 8.0 LABORATORY TEST RESULTS

Results of physical and chemical tests performed for this project are presented below.

## *8.1 Physical Testing*

Results of the various laboratory tests are presented in Appendix B, *Laboratory Testing Program*, except for the results of in-situ moisture and dry density tests which are presented on the Logs of Borings in Appendix A, *Field Exploration*. The results are also discussed below.

- In-situ Moisture and Dry Density: *In-situ* dry densities and moisture contents of the alluvium soils were determined in accordance with ASTM Standard D2216 and D2937. Dry densities of onsite soils ranged from 71 to 131 pounds per cubic foot (pcf) with moisture content of 1 to 49 percent.
- Expansion Index (EI): One representative sample from the upper 5 feet soils was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The EI test result was 0.
- R-Value (R) – One representative bulk sample was tested in accordance with Caltrans Test Method 301. The result of the R-value test was 71.
- Grain Size Analysis (PA): Four representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content (CP): A typical moisture-density relationship test was performed on two representative samples in accordance with ASTM D1557. The results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program*. The laboratory maximum dry densities were 117.0 and 122.0 pcf, and the optimum moisture contents were 12.0 and 11.0 percent respectively.
- Direct Shear (DS): Three direct shear tests were performed on relatively undisturbed representative ring samples under soaked moisture condition in accordance with ASTM Standard D3080. The results are presented in Drawings No. B-3 through B-5, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

## *8.2 Chemical Testing - Corrosivity Evaluation*

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purposes of these tests were to determine the corrosion potential of site soils when placed in contact with common pipe materials. These tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Tests 643, 422, and 417. The test results are presented in Appendix B, *Laboratory Testing Program* and summarized below.

- The pH measurements of the tested samples were 8.2 and 8.5.
- The sulfate contents of the tested samples were 307 ppm and 100 ppm, respectively (0.031 and .010 percent by weight).
- The chloride concentrations of the tested samples were 171 ppm and 75 ppm, respectively.
- The minimum electrical resistivity when saturated was 956 and 1,637 ohm-cm, respectively.

# 9.0 PERCOLATION TESTING

Percolation testing was performed at two locations (BH-02/PT-01 and BH-04/PT-02) on September 29, 2023, to estimate the infiltration rates at the site. Details of the percolation testing are presented in Appendix D, *Percolation Testing.* The estimated infiltration rates at the test holes are presented in the following table.

**Table No. 4, Estimated Infiltration Rates**

| Percolation Test | Depth (feet) | Soil Type | Infiltration Rate (inches/hour) |
|---|---|---|---|
| PT-01 | 10.2 | Silty Sand (SM) and Sand with Silt (SP-SM) | 1.31 |
| PT-02 | 5.7 | Silty Sand (SM) and Sand with Silt (SP-SM) | 0.54 |

Based on the test data, the infiltration rate of 1.31 (inches/hour) is for a depth of 10.2 feet bgs. and the infiltration rate of 0.54 (inches/hour) is for a depth of 5.7 feet bgs. Design infiltration rate should be selected based on infiltration structure depth and soil type at that depth. A factor of safety of 3 was applied to the measured infiltration rate to account for subsurface variations, uncertainty in the test method, and future siltation. Please note that infiltration rates may change if the soil type and location of the proposed system changes. If that is the case, then additional percolation testing should be performed in the required location.

# 10.0 EARTHWORK RECOMMENDATIONS

Earthwork recommendations for the project are presented in the following sections.

## *10.1 General*

This section contains our general recommendations regarding earthwork and grading for the project. These recommendations are based on the results of our field exploration, laboratory tests, our experience with similar projects, and data evaluation as presented in the preceding sections. These recommendations may require modification by the geotechnical consultant based on observation of the actual field conditions during grading.

Prior to the start of construction, all existing underground utilities and appurtenances should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing utilities and structure (if any).

All debris, surface vegetation, deleterious material, and surficial soils containing roots and perishable materials should be stripped and removed from the site.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill. Based on these observations, localized areas may require remedial grading deeper than indicated herein. Therefore, some variations in the depth and lateral extent of excavation recommended in this report should be anticipated.

Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

The present in-situ moisture content of the soils is higher than the optimum moisture content (11-12%). So, drying of the soil will most likely be required during the construction.

It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.

### *10.2 Overexcavation*

The site is generally underlain by potentially compressible soils which may be prone to future settlement under the surcharge of foundation, improvements and/or fill loads. Therefore, these materials should be over-excavated within all areas of proposed structures and other improvements and replaced with compacted fill soils. Provided proposed fill loads above existing grade are not more than 2 feet to 3 feet. Greater proposed fill depths may require greater overexcavation depths.

<u>Building Pad:</u> Within the entire level portions of the building pad areas overexcavations should be at least 6.0 feet below existing grade, as well as 4.0 feet below the lowest proposed building footings, whichever is deeper. All over-excavations should extend

laterally at least 6.0 feet or equal to the depth of over-excavation, whichever is greater, outside the entire portions of the building pad area.

Improvements Outside of the Building Area: For areas of proposed parking, flatwork, walls, and other improvements, overexcavations should be at least 5.0 feet below existing grade. Within wall areas overexcavations should also be a minimum of 3.0 feet below the proposed wall footings. All over-excavations should extend laterally at least 5.0 feet or equal to the depth of over-excavation, whichever is greater.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill or structures. However, localized, deeper over-excavation could be required based observation made during grading of the final bottom surfaces of all excavations should be approved by the geotechnical consultant.

If isolated pockets of very soft, loose, eroded, or pumping soil are encountered, the unstable soil should be excavated as needed to expose undisturbed, firm, and unyielding soils.

The contractor should determine the best manner to conduct the excavations, such that there are no losses of bearing and/or lateral support to the existing structures or utilities (if any).

Areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

### *10.3 Cut and Shallow Fill Below Building Pad Areas*

Building pads with shallow cut and fill areas should be capped with a minimum of 5.0 feet of engineered structural fill, so that all footings for structures and walls are founded into engineered fill with a minimum of 4.0 feet of fill below footings for proposed structures and 3.0 feet below footings for proposed walls. Over-excavation should extend to the entire level portions of the building pad area with proposed structures or walls, to the depth of fill.

### *10.4 Cut/Fill Transition and Fill Differentials*

To mitigate distress to structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all level portions of the building pad areas. This should be accomplished by overexcavating the entire “cut” portion of the building pad area by at least 5.0 feet below proposed grade and replacing the excavated materials as properly compacted fill, so that all footings for structures and walls are founded into engineered fill with a minimum of 4.0 feet of fill below footings for proposed structures and 3.0 feet below footings for proposed walls. Recommended depths of over-excavation are provided in the following table.

**Table No. 5, Overexcavation Depths for Cut/Fill Transitions**

| Depth of Fill ("Fill" Portion) | Depth of Overexcavation ("Cut" Portion) |
|---|---|
| Up to 5.0 feet | 5.0 feet |
| Greater than 5.0 feet | One-third the maximum thickness of fill placed on the "fill" portion (15 feet maximum) |

### *10.5 Engineered Fill*

No fill should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the project sites are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including removal of roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-sites soils used as fill should meet the following criteria.

- No particles larger than 3 inches in largest dimension.
- Rocks larger than 1 inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all significant organic matter, debris, or other deleterious material.
- Expansion index of 20 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Based on field investigation and laboratory testing results, on-sites soils may be suitable as fill materials provided that appropriate corrosion mitigation and moisture conditioning will be applied.

Imported materials, if required, should meet the above criteria prior to being used as compacted fill. Any imported fills should be tested and approved by geotechnical representative at least 72 hours prior to delivery to the site.

### *10.6 Compacted Fill Placement*

All surfaces to receive structural fills should be scarified to a depth of 12 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

Fill soils should be thoroughly mixed, and moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. Fill soils should be evenly spread in horizontal lifts not exceeding 8 inches in uncompacted thickness.

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein. The upper 12 inches of subgrade soils underneath pavements intended to support vehicle loads should be scarified, moisture conditioned, and compacted to at least 95 percent of the laboratory maximum dry density.

Fill materials should not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations should not resume until the geotechnical consultant approves the moisture and density conditions of the previously placed fill.

Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

Additional expansion index and corrosion testing should be completed after all fill has been placed and compacted at the site in order to confirm the soil properties and revise the foundation design parameters if necessary.

### *10.7 Backfill Recommendations Behind Walls*

Compaction of backfill adjacent to retaining walls, which may be proposed, can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the walls.

### *10.8 Shrinkage and Subsidence*

The volume of excavated and recompacted soils will decrease as a result of grading. The shrinkage would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. Based on our exploration as well as previous experience in other projects in close vicinity of this site, for the preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- The shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) for the upper 10 feet of soils is estimated to range from approximately 10 to 33 percent. An average value of 20 percent may be used for preliminary earthwork planning.
- Subsidence (defined as the settlement of native materials from the equipment load applied during grading and proposed fill loads) would depend on the construction methods including type of equipment utilized. Ground subsidence is estimated to be approximately 0.20 foot to 0.25 foot.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

### *10.9 Site Drainage*

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. A desirable drainage gradient is 1 percent for paved areas and 2 percent for landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

### *10.10 Utility Trench Backfill*

The following sections present earthwork recommendations for utility trench backfill, including subgrade preparation and trench zone backfill.

Open cuts adjacent to existing roadways or structures are not recommended within a 1:1 (horizontal: vertical) plane extending down and away from the roadway or structure perimeter (if any).

Soils from the trench excavation should not be stockpiled more than 6 feet in height or within a horizontal distance from the trench edge equal to the depth of the trench. Soils should not be stockpiled behind the shoring, if any, within a horizontal distance equal to the depth of the trench, unless the shoring has been designed for such loads.

#### *10.10.1 Pipeline Subgrade Preparation*

The final subgrade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than 2 inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

Any loose, soft and/or unsuitable materials encountered at the pipe subgrade should be removed and replaced with an adequate bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom as near its full length as is practicable.

#### *10.10.2 Pipe Bedding*

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Recommendations for pipe bedding are provided below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate or crushed rock may be used as pipe bedding material. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

The type and thickness of the granular bedding placed underneath and around the pipe, if any, should be selected by the pipe designer. The load on the rigid pipes and deflection of flexible pipes and, hence, the pipe design, depends on the type and the amount of bedding placed underneath and around the pipe.

Bedding materials should be vibrated in-place to achieve compaction. Care should be taken to densify the bedding material below the spring line of the pipe. Prior to placing the pipe bedding material, the pipe subgrade should be uniform and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom as near its full length as is practicable.

Based on the design groundwater depth, migration of fines from the surrounding native and/or fill soils may not be considered in selecting the gradation of any imported bedding material.

### *10.10.3 Trench Zone Backfill*

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated sites soil free of oversize particles and deleterious matter may be used to backfill the trench zone. Detailed trench backfill recommendations are provided below.

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.
- Trench zone backfill should be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method. At least the upper 1 foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.
- Particles larger than 1 inch should not be placed within 12 inches of the pavement subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than 3 inches in the largest dimension should not be placed as trench backfill.
- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse-grained soil, and between optimum and 2 percent above optimum for fine-grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.
- The field density of the compacted soil should be measured by the ASTM D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) or equivalent.
- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

# 11.0 DESIGN RECOMMENDATIONS

The various design recommendations provided in this section are based on the assumption that the above earthwork and grading recommendations will be implemented in the project design and construction.

## *11.1 Shallow Foundation Design Parameters*

The proposed improvements may be supported on continuous footing and/or isolated spread footings. The design of the shallow foundations should be based on the recommended parameters presented in the table below.

**Table No. 6, Recommended Foundation Parameters**

| Parameter | Value |
|---|---|
| Minimum continuous or isolated spread footing width | 18 inches |
| Minimum continuous or isolated spread footing depth of embedment below lowest adjacent grade | 24 inches |
| Allowable net bearing capacity | 2,000 psf |

The actual footing dimensions and reinforcement should be based on structural design. The allowable bearing capacity can be increased by 500 pounds per square foot (psf) with each foot of additional embedment and 100 psf with each foot of additional width up to a maximum of 3,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.  If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

## *11.2 Lateral Earth Pressures and Resistance to Lateral Loads*

In the following subsections, the lateral earth pressures and resistance to lateral loads are estimated by using on-site native soils strength parameters obtained from laboratory testing.

### 11.2.1 Active Earth Pressures

The active earth pressure behind any foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, foundation inclination, surcharges, and any hydrostatic pressures.  The lateral earth pressures are presented in the following tables.

**Table No. 7, Active and At-Rest Earth Pressures**

| Loading Conditions | Lateral Earth Pressure[1] (psf) | Lateral Earth Pressure[2] (psf) |
|---|---|---|
| | Level backfill | 2:1 backfill |
| Active earth conditions (wall is free to deflect at least 0.001 radian) | 50 | 95 |
| At-rest (wall is restrained) | 70 | 125 |

These pressures assume a level ground surface around the structure for a distance greater than the structure height, no surcharge, and no hydrostatic pressure. If water pressure is allowed to build up behind the walls, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the walls.

### 11.2.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.30 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 215 psf per foot of depth may be used for the sides of the footing poured against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,000 psf.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.

Due to the low overburden stress of the soil at shallow depth, the upper 1 foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

## *11.3 Slabs-on-Grade*

Slabs-on-grade should be supported on properly compacted fill. Compacted fill used to support slabs-on-grade should be placed and compacted in accordance with Section 10.6 *Compacted Fill Placement*.

Structural design elements of slabs-on-grade, including but not limited to thickness, reinforcement, joint spacing of more heavily loaded slabs will be dependent upon the anticipated loading conditions and the modulus of subgrade reaction of the supporting materials and should be designed by a structural engineer.

Slabs should be designed and constructed as promulgated by the American Concrete Institute (ACI) and the Portland Cement Association (PCA). Care should be taken during concrete placement to avoid slab curling. Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

Subgrade for slabs-on-grade should be firm and uniform. All loose or disturbed soils including under-slab utility trench backfill should be recompacted.
In hot weather, the contractor should take appropriate curing precautions after placement of concrete to minimize cracking or curling of the slabs. The potential for slab cracking may be lessened by the addition of fiber mesh to the concrete and/or control of the water/cement ratio.

Concrete should be cured by protecting it against loss of moisture and rapid temperature change for at least 7 days after placement. Moist curing, waterproof paper, white polyethylene sheeting, white liquid membrane compound, or a combination thereof may be used after finishing operations have been completed. The edges of concrete slabs exposed after removal of forms should be immediately protected to provide continuous curing.

### *11.4 Settlement*

The total settlement of shallow footings, designed as recommended above, from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1.0 inch or less. The static differential settlement can be taken as equal to one-half of the static total settlement over a lateral distance of 30 feet.

Our analysis of the potential dynamic settlement is presented in Appendix C, *Liquefaction and Settlement Analysis*. We estimate that the project site has the potential for up to 0.99 inches of dry seismic settlement and 3.26 inches of liquefaction induced settlement.

Generally, static and dynamic settlement does not occur at the same time. For design purposes, the structural engineer should decide whether static and dynamic settlement will be combined or not.

### 11.5 *Liquefaction Mitigation Measures*

Liquefaction Hazard can be mitigated by improving the strength, density, and/or drainage characteristics of the soil. This can be done using a variety of improvement techniques. Brief discussion on the various ground improvement methods. presented below.

**Vibroflotation**

Vibroflotation involves the use of a vibrating probe that can penetrate granular soil to depths of over 100 feet. The vibrations of the probe cause the grain structure to collapse thereby densifying the soil surrounding the probe. To treat potentially liquefiable soil, the vibroflot is raised and lowered in a grid pattern. Vibro Replacement is a combination of vibroflotation with a gravel backfill resulting on stone columns, which not only increases the amount of densification, but provides a degree of reinforcement and a potentially effective means of drainage.

**Dynamic Compaction**

Densification by dynamic compaction is performed by dropping a heavy weight of steel or concrete in a grid pattern from heights of 30 to 100 feet. It provides an economical way of improving soil for mitigation of liquefaction hazard. Local liquefaction can be initiated beneath the drop point making it easier for the sand grain to densify.

**Stone Columns**

Stone columns are columns of gravel constructed in the ground. Stone columns can be constructed by the vibroflotation method. They can be installed in other ways, for example, with the help of a steel casing and a drop hammer. In this approach the steel casing is driven into soil and gravel is filled in from the top and tamped with a drop hammer as the steel casing is successively withdrawn.

**Compaction Piles**

Installing compaction piles is a very effective way of improving soil. Compaction piles are usually made of prestressed concrete or timber. Installation of compaction piles both densifies and reinforces the soils. The piles are generally installed in a grid pattern and are generally driven to depths of up to 60 feet.

**Compaction Grouting**

Compaction grouting is a technique whereby a slow-flowing water/sand/cement mix is injected under pressure into a granular soil. The grout forms a bulb that displaces and hence densifies, the surrounding soil.

**Drainage Technique**

Liquefaction hazard can be reduced by increasing the drainage ability of the soil. Drainage techniques include installation of drains of gravel sand or synthetic materials, Synthetic wick drains can be installed at various angles, in contrast to gravel or sand drains that are usually installed vertically. Drainage techniques are often used in combination with other types of soils improvement techniques for more effective liquefaction hazard reduction.

### *11.6 Pipe Design*

Structural design of pipes requires proper evaluation of all possible loads acting on pipes. The stresses and strains induced on buried pipes depend on many factors, including the type of soil, density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils. The recommended values of the various soil parameters for the pipe design are provided in Table No. 8, *Soil Parameters for Pipe Design*.

Where pipes are connecting to rigid structures near, or at its lower levels, and then are subjected to significant loads as the backfill is placed to finish grade, we recommend that provisions be incorporated in the design to provide support of these pipes where they exit the structure. Consideration can be given to flexible connections, concrete slurry support beneath the pipes where they exit the structures, overlaying and supporting the pipes with a few inches of compressible material (i.e., styrofoam or other materials), or other techniques. Automatic shutoffs should be installed to limit the potential leakage in the event of damage in a seismic event.

**Table No. 8, Soil Parameters for Pipe Design**

| Soil Parameters | Parameters |
|---|---|
| Total unit weight of compacted backfill (assuming 92% average relative compaction), $\gamma$ | 130 pcf |
| Angle of internal friction of soils, $\phi$ | 27º |
| Soil cohesion, c | 50 psf |
| Coefficient of friction between concrete and native soils, fs | 0.30 |
| Coefficient of friction between pipe and native soils, fs | 0.25 for CML&C steel & HDPE pipe |
| Bearing pressure against compacted soils | 1,500 psf |
| Coefficient of passive earth pressure, Kp | 2.66 |
| Coefficient of active earth pressure, Ka | 0.38 |
| *Modulus of Soil Reaction, E' | 1,500 psi |

(* Modulus of soil reaction, E' is provided for native trench wall soil. )

### *11.7 Soil Corrosivity*

Two representative soil samples were evaluated for corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program* and design recommendations pertaining to soil corrosivity are presented below.

The sulfate content of the sampled soils corresponds to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-14, Table 19.3.1.1). No concrete type restrictions are specified for exposure category S0 (ACI 318-14, Table 19.3.2.1). A minimum compressive strength of 2,500 psi is recommended.

We anticipate that concrete structures such as footings, slabs, and flatwork will be exposed to moisture from precipitation and irrigation. Based on the site locations and the results of chloride testing of the site soils, we do not anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

According to Romanoff, 1957, the following table provides general guidelines of soil corrosion based on electrical resistivity.

**Table No. 9, Correlation Between Resistivity and Corrosion**

| Soil Resistivity (ohm-cm) per Caltrans CT 643 | Corrosivity Category |
|---|---|
| Over 10,000 | Mildly corrosive |
| 2,000 – 10,000 | Moderately corrosive |
| 1,000 – 2,000 | Corrosive |
| Less than 1,000 | Severe corrosive |

The measured value of the minimum electrical resistivity of the samples when saturated were 956 and 1,637 ohm-cm for the site. This indicates that the soil tested is severely corrosive to corrosive to ferrous metals in contact with the soil. Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.

### *11.8 Asphalt Concrete Pavement*

Based on the laboratory test result, the R-value of the subgrade soil was 71. For pavement design, we have utilized an R-value of 50 and design Traffic Indices (TIs) ranging from 5 to 8.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2022), Chapter 630 with a safety factor of 0.2 for asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table below.

**Table No. 10, Recommended Preliminary Flexible Pavement Sections**

| R-value | Traffic Index (TI) | Pavement Section | | |
|---|---|---|---|---|
| | | Option 1 | | Option 2 |
| | | Asphalt Concrete (inches) | Aggregate Base (inches) | Full AC Section (inches) |
| 50 | 5.0 | 3.0 | 4.0 | 4.5 |
| | 6.0 | 3.5 | 4.0 | 5.5 |
| | 7.0 | 4.0 | 4.5 | 7.0 |
| | 8.0 | 5.0 | 5.0 | 8.5 |

At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design.

Prior to placement of aggregate base, at least 12 inches below finish grade should be overexcavated, processed and replaced as compacted fill (recompacted to at least 95 percent of the laboratory maximum dry density as defined by ASTM Standard D1557 test method).

Base materials should conform with Section 200-2.2,"*Crushed Aggregate Base*," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2021) and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302.5 of the SSPWC.

### *11.9 Rigid Pavement*

Rigid pavement design recommendations were provided in accordance with the Portland Cement Association's (PCA) Southwest Region Publication P-14, Portland Cement Concrete Pavement (PCCP) for Light, Medium and Heavy Traffic Rigid Pavement. For pavement design, we have utilized a design subgrade R-value of 50 and design Traffic Indices (TIs) ranging from 5 to 8. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table:

**Table No. 11, Recommended Preliminary Rigid Pavement Sections**

| Design R-Value | Design Traffic Index (TI) | PCCP Pavement Section (inches) |
|---|---|---|
| 50 | 5.0 | 6.0 |
| | 6.0 | 6.5 |
| | 7.0 | 6.5 |
| | 8.0 | 7.0 |

The above pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,750 psi. The third point method of testing beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are 3.0 inches to 1.0 inch, respectively.

Transverse contraction joints should not be spaced more than 10 feet and should be cut to a depth of 1/4 the thickness of the slab. Longitudinal joints should not be spaced more than 12 feet apart. A longitudinal joint is not necessary in the pavement adjacent to the curb and gutter section.

Prior to placement of concrete, at least the upper 12 inches of subgrade soils below rigid pavement sections should be compacted to at least 95% relative compaction as defined by the ASTM D 1557 standard test method.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into pavement base and/or subgrade.

### *11.10 Concrete Flatwork*

Except as modified herein, concrete walks, driveways, access ramps, curb and gutters should be constructed in accordance with Section 303-5, *Concrete Curbs, Walks, Gutters, Cross-Gutters, Alley Intersections, Access Ramps, and Driveways*, of the Standard Specifications for Public Works Construction (Public Works Standards, 2021).

The subgrade soils under the above structures should consist of compacted fill placed as described in this report. Prior to placement of concrete, the upper at least 1 foot of subgrade soils should be moisture conditioned to between within 3 percent of optimum moisture content for coarse-grained soils and 0 and 2 percent above optimum for fine-grained soils.

The thickness of driveways for passenger vehicles should be at least 4 inches, or as required by the civil or structural engineer. Transverse control joints for driveways should be spaced not more than 10 feet apart. Driveways wider than 12 feet should be provided with a longitudinal control joint.

Concrete walks subjected to pedestrian and bicycle loading should be at least 4 inches thick, or as required by the civil or structural engineer. Transverse joints should be spaced 15 feet or less and should be cut to a depth of one-fourth the slab thickness.

Positive drainage should be provided away from all driveways and sidewalks to prevent seepage of surface and/or subsurface water into the concrete base and/or subgrade.

# 12.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation and shoring design recommendations are presented in the following sections.

## *12.1 General*

Prior to the start of construction, all existing underground utilities should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications.

Vertical braced excavations can be considered for the foundations. Sloped excavations may not be feasible in locations adjacent to existing utilities, pavement, or structures. Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing structures may require vertical side wall excavation. Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act, and the Construction Safety Act should be met. The soil exposed in cuts should be observed during excavation by the geotechnical consultant and the competent person designated by the contractor. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

## *12.2 Temporary Sloped Excavations*

Temporary open-cut trenches may be constructed with side slopes as recommended in the following table. Temporary cuts encountering soft and wet fine-grained soils; dry loose, cohesionless soils or loose fill from trench backfill may have to be constructed at a flatter gradient than presented below.

**Table No. 12, Slope Ratios for Temporary Excavations**

| Soil Type | OSHA Soil Type | Depth of Excavation (ft) | Recommended Maximum Slope (Horizontal:Vertical)[1] |
|---|---|---|---|
| Sand (SP), Sand with Silt (SP-SM), Silty Sand (SM), Clayey Sand (SC), Sandy Silt (ML), and Sandy Clay (CL) | C | 0-4 | vertical |
| | | 4-10 | 1.5:1 |

[1] Slope ratio assumed to be uniform from top to toe of slope.

For shallow excavations up to 4 feet bgs, excavation can be vertical. For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trenches should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring can be provided if necessary.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction materials, should not be placed within 5 feet of the unsupported slope edge. Stockpiled soils with a height higher than 6 feet will require greater distance from trench edges.

## 13.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should review plans and specifications as the project design progresses. Such review is necessary to identify design elements, assumptions, or new conditions which require revisions or additions to our geotechnical recommendations.

The project geotechnical consultant should be present to observe conditions during construction. Geotechnical observation and testing should be performed as needed to verify compliance with project specifications. Additional geotechnical recommendations may be required based on subsurface conditions encountered during construction.

## 14.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by David Beckwith and Associates, Inc., Riverside County Parks and Open Space District, and their authorized agents to assist in the design and construction of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

Converse Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others. Field exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling and laboratory testing is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. In the event that changes to the project occur, or additional, relevant information about the project is brought to our attention, the recommendations contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report are modified or verified in writing. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

As the project evolves, continued consultation and construction monitoring by a qualified geotechnical consultant should be considered an extension of geotechnical investigation services performed to date. The geotechnical consultant should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid. Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

Design recommendations given in this report are based on the assumption that it will be implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

## 15.0 REFERENCES

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# Appendix A

## Field Exploration

# APPENDIX A

## FIELD EXPLORATION

Our field investigation included site reconnaissance and a subsurface exploration program consisting of drilling soil borings and conducting water percolation tests. During the site reconnaissance, the surface conditions were noted, and the borings were marked at locations approved by Mr. David Beckwith with David Beckwith and Associates, Inc. The boring locations were established in the field using approximate distances from existing features as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Eight exploratory borings (BH-01 through BH-08) were drilled on September 25, 2023, to investigate the subsurface conditions at the site to a maximum depth of 51.5 feet below existing ground surface (bgs). Borings details are presented below in Table No. A-1, *Summary of Borings* and Drawings Nos. A-2 through A-9, *Logs of Borings.*

**Table No. A-1, Summary of Borings**

| Boring No. | Boring Depth (ft, bgs) | | Groundwater Depth (ft, bgs) | Date Completed |
|---|---|---|---|---|
| | Proposed | Completed | | |
| BH-01 | 50.0 | 51.5 | 13.7 | 9/25/2023 |
| BH-02/PT-01 | 10.0 | 11.5 | Not Encountered | 9/25/2023 |
| BH-03 | 10.0 | 11.5 | Not Encountered | 9/25/2023 |
| BH-04/PT-02 | 5.0 | 6.5 | Not Encountered | 9/25/2023 |
| BH-05 | 10.0 | 11.5 | Not Encountered | 9/25/2023 |
| BH-06 | 10.0 | 11.5 | Not Encountered | 9/25/2023 |
| BH-07 | 10.0 | 11.5 | Not Encountered | 9/25/2023 |
| BH-08 | 10.0 | 11.5 | Not Encountered | 9/25/2023 |

After logging and soil sampling, borings BH-02 and BH-04 were set up for percolation testing and will be referred to as BH-02/PT-01 and BH-04/PT-02 hereafter. The depths of BH-02/PT-01 and BH-04/PT-02 were restricted to 10.2 and 5.7 feet bgs respectively due to caving in the test holes. Details about the percolation tests are presented in Appendix D, *Percolation Testing*.

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. Encountered materials were continuously logged by a Converse geologist and classified in the field by visual classification in accordance with the Unified Soil Classification System. Where appropriate, the field descriptions and classifications have been modified to reflect laboratory test results.

Relatively undisturbed samples were obtained using California Modified Samplers (2.4 inches inside diameter and 3.0 inches outside diameter) lined with thin sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches. Blow counts at each sample interval are presented on the boring logs. Samples were retained in brass rings (2.4 inches inside diameter and 1.0 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk samples of typical soil types were also obtained.

Standard Penetration Testing (SPT) was also performed at BH-01 in accordance with the ASTM Standard D1586 test method at 10-foot intervals beginning at 20 feet bgs using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the *Logs of Boring*.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between drive samples are indicated on the logs at the top of the next drive sample.

Following the completion of logging and sampling, borings BH-01, BH-03, and BH-05 through BH-08 were backfilled with soil cuttings and compacted by pushing down with the augers using the weight of the drill rig. After completion of percolation testing, the pipes were removed from BH-02/PT-01 and BH-04/PT-02 and boreholes were backfilled with soil cuttings and compacted. If construction is delayed, the surface of the borings may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing Nos. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawing Nos. A-2 through A-9, *Logs of Borings*. All elevations are based on Google Earth.

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | GRAVEL AND GRAVELLY SOILS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS (LITTLE OR NO FINES) | | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | | | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | CLEAN SANDS (LITTLE OR NO FINES) | | SW | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | | | SP | POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES |
| | | SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | | OL | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | OH | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| HIGHLY ORGANIC SOILS | | | | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

## FIELD AND LABORATORY TESTS

| | |
|---|---|
| C | Consolidation (ASTM D 2435) |
| CL | Collapse Potential (ASTM D 4546) |
| CP | Compaction Curve (ASTM D 1557) |
| CR | Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422) |
| CU | Consolidated Undrained Triaxial (ASTM D 4767) |
| DS | Direct Shear (ASTM D 3080) |
| EI | Expansion Index (ASTM D 4829) |
| M | Moisture Content (ASTM D 2216) |
| OC | Organic Content (ASTM D 2974) |
| P | Permeablility (ASTM D 2434) |
| PA | Particle Size Analysis (ASTM D 6913 [2002]) |
| PI | Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318) |
| PL | Point Load Index (ASTM D 5731) |
| PM | Pressure Meter |
| PP | Pocket Penetrometer |
| R | R-Value (CTM 301) |
| SE | Sand Equivalent (ASTM D 2419) |
| SG | Specific Gravity (ASTM D 854) |
| SW | Swell Potential (ASTM D 4546) |
| TV | Pocket Torvane |
| UC | Unconfined Compression - Soil (ASTM D 2166) |
| | Unconfined Compression - Rock (ASTM D 7012) |
| UU | Unconsolidated Undrained Triaxial (ASTM D 2850) |
| UW | Unit Weight (ASTM D 2937) |
| WA | Passing No. 200 Sieve |

## BORING LOG SYMBOLS

### SAMPLE TYPE

STANDARD PENETRATION TEST
Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method

DRIVE SAMPLE 2.42" I.D. sampler (CMS).

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

### DRILLING METHOD SYMBOLS

Auger Drilling

Mud Rotary Drilling

Dynamic Cone or Hand Driven

Diamond Core

# UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

**Santa Ana River Bottom (SARB) Maintenance Facility**
**4600 Crestmore Road**
**City of Riverside, Riverside County, California**
**For: David Beckwith and Associates, Inc.**

Project No. **23-81-234-01**

Drawing No. **A-1a**

## CONSISTENCY OF COHESIVE SOILS

| Descriptor | Unconfined Compressive Strength (tsf) | SPT Blow Counts | Pocket Penetrometer (tsf) | CA Sampler | Torvane (tsf) | Field Approximation |
|---|---|---|---|---|---|---|
| Very Soft | <0.25 | < 2 | <0.25 | <3 | <0.12 | Easily penetrated several inches by fist |
| Soft | 0.25 - 0.50 | 2 - 4 | 0.25 - 0.50 | 3 - 6 | 0.12 - 0.25 | Easily penetrated several inches by thumb |
| Medium Stiff | 0.50 - 1.0 | 5 - 8 | 0.50 - 1.0 | 7 - 12 | 0.25 - 0.50 | Can be penetrated several inches by thumb with moderate effort |
| Stiff | 1.0 - 2.0 | 9 - 15 | 1.0 - 2.0 | 13 - 25 | 0.50 - 1.0 | Readily indented by thumb but penetrated only with great effort |
| Very Stiff | 2.0 - 4.0 | 16 - 30 | 2.0 - 4.0 | 26 - 50 | 1.0 - 2.0 | Readily indented by thumbnail |
| Hard | >4.0 | >30 | >4.0 | >50 | >2.0 | Indented by thumbnail with difficulty |

## APPARENT DENSITY OF COHESIONLESS SOILS

| Descriptor | SPT $N_{60}$- Value (blows / foot) | CA Sampler |
|---|---|---|
| Very Loose | <4 | <5 |
| Loose | 4- 10 | 5 - 12 |
| Medium Dense | 11 - 30 | 13 - 35 |
| Dense | 31 - 50 | 36 - 60 |
| Very Dense | >50 | >60 |

## MOISTURE

| Descriptor | Criteria |
|---|---|
| Dry | Absence of moisture, dusty, dry to the touch |
| Moist | Damp but no visible water |
| Wet | Visible free water, usually soil is below water table |

## PERCENT OF PROPORTION OF SOILS

| Descriptor | Criteria |
|---|---|
| Trace (fine)/ Scattered (coarse) | Particles are present but estimated to be less than 5% |
| Few | 5 to 10% |
| Little | 15 to 25% |
| Some | 30 to 45% |
| Mostly | 50 to 100% |

## SOIL PARTICLE SIZE

| Descriptor | | Size |
|---|---|---|
| Boulder | | > 12 inches |
| Cobble | | 3 to 12 inches |
| Gravel | Coarse | 3/4 inch to 3 inches |
| | Fine | No. 4 Sieve to 3/4 inch |
| Sand | Coarse | No. 10 Sieve to No. 4 Sieve |
| | Medium | No. 40 Sieve to No. 10 Sieve |
| | Fine | No. 200 Sieve to No. No. 40 Sieve |
| Silt and Clay | | Passing No. 200 Sieve |

## PLASTICITY OF FINE-GRAINED SOILS

| Descriptor | Criteria |
|---|---|
| Nonplastic | A 1/8-inch thread cannot be rolled at any water content. |
| Low | The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit. |
| Medium | The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit. |
| High | It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit. |

## CEMENTATION/ Induration

| Descriptor | Criteria |
|---|---|
| Weak | Crumbles or breaks with handling or little finger pressure. |
| Moderate | Crumbles or breaks with considerable finger pressure. |
| Strong | Will not crumble or break with finger pressure. |

**NOTE:** This legend sheet provides descriptions and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

# UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

# Log of Boring No. BH-01

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 753 Depth to Water (ft, bgs): 13.7

| Depth (ft) | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|
| 0 | **ALLUVIUM**<br>**SILTY SAND (SM):** fine-grained, trace clay, pinhole porosity, moderately desiccated, trace caliche, very loose, moist, dark grayish brown. | 2/2/2 | 32 | 74 | CP, CR, EI, PA |
| 5 | **SANDY SILT (ML):** fine-grained sand, trace clay, trace oxidation staining, medium stiff, moist, brownish gray. | 2/3/5 | 16 | 95 | PA |
| | - @7.5': some dark orange oxidation staining, thin (0.5 to 6.0 inches thick), alternating layers of (SM) and (ML). | 3/3/4 | 26 | 89 | DS |
| 10 | **SILTY SAND (SM):** fine to coarse-grained, loose, moist, brownish gray. | 3/4/6 | 6 | 91 | PA |
| 15 | | 3/5/5 | 14 | 92 | |
| 20 | - @20': medium dense. | 2/5/7 | 13 | | |
| 25 | **SAND WITH GRAVEL (SW):** fine to coarse-grained, gravel up to 1.0" maximum dimension, dense, moist, light grayish brown. | 10/17/22 | 12 | 131 | PA |
| 30 | **SAND WITH GRAVEL (SP):** fine to coarse-grained, gravel up to 1.0" maximum dimension, medium dense, moist, light grayish brown. | 8/11/16 | 18 | | |

# Log of Boring No. BH-01

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 753 Depth to Water (ft, bgs): 13.7

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SAND WITH GRAVEL (SP):** fine to coarse-grained, gravel up to 1.0" maximum dimension, very dense, fully saturated, light grayish brown. | | | 8/14/50-5" | 16 | 107 | |
| 40 | | - @40': dense. | | | 4/12/20 | 11 | | |
| 45 | | - @45': very dense. | | | 6/25/36 | 8 | 116 | |
| 50 | | - @50': medium dense. | | | 4/6/14 | 17 | | |
| | | End of boring at 51.5 bgs.<br>Groundwater encountered at 13.7'; stabilized at 12.8'.<br>Boring backfilled with soil cuttings and tamped with an auger using the weight of the drill rig on 09/25/2023. | | | | | | |

# Log of Boring No. BH-02/PT-01

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 755 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | | | | | | |
| | | **ALLUVIUM** **SILTY SAND (SM):** fine-grained, trace clay, pinhole porosity, moderately desiccated, trace caliche, loose, moist, dark grayish brown. | | | | | | |
| | | | ■ | | 2/3/3 | 7 | 81 | |
| 5 | | **SAND WITH SILT (SP-SM):** fine-grained, trace clay, slightly desiccated, loose, moist, light gray. | ■ | ▨ | 3/3/3 | 3 | 84 | |
| | | - @7.5': alternating layers of (SM) and (SP). | ■ | ▨ | 3/3/5 | | | * no recovery* |
| 10 | | | ■ | | 2/4/4 | 14 | 95 | |

End of boring at 11.5 bgs.
Restricted to 10.2 bgs due to caving.
No groundwater encountered.
Percolation tube installed on 09/25/2023.
Borehole presoaked on 09/28/2023.
After completion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 09/29/2023.

# Log of Boring No. BH-03

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 757 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SILTY SAND (SM):** fine-grained, trace clay, pinhole porosity, moderately desiccated, trace caliche, loose, moist, dark grayish brown. | | | 3/4/4 | 12 | 86 | |
| 5 | | **SAND WITH SILT (SP-SM):** fine-grained, trace clay, slightly desiccated, loose, moist, light gray. | | | 4/5/6 | 21 | 85 | DS |
| | | **SAND (SP):** fine to medium-grained, trace silt, loose, moist, light grayish brown. | | | 3/4/5 | | | *no recovery* |
| 10 | | | | | 3/3/7 | 2 | | |

End of boring at 11.5 bgs.
No groundwater encountered.
Boring backfilled with soil cuttings and tamped with an auger using the weight of the drill rig on 09/25/2023.

# Log of Boring No. BH-04/PT02

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 751 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SILTY SAND (SM):** fine-grained, trace clay, pinhole porosity, moderately desiccated, trace caliche, loose, moist, dark grayish brown. | | | | | | |
| | | | | | 2/3/4 | 15 | | *disturbed* |
| 5 | | **SAND WITH SILT (SP-SM):** fine-grained, trace clay, loose, moist, light gray. | | | 2/2/4 | 10 | 90 | |

End of boring at 6.5 bgs.
Restricted to 5.7 bgs due to caving.
No groundwater encountered.
Percolation tube installed on 09/25/2023.
Borehole presoaked on 09/28/2023.
After completion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 09/29/2023.

# Log of Boring No. BH-05

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 752 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES: DRIVE | SAMPLES: BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SAND WITH SILT (SP-SM):** fine-grained, trace clay, moist, grayish brown. | | | | | | |
| | | **SANDY CLAY (CL):** fine-grained sand, few silt, slightly to moderately desiccated, trace caliche spots, medium stiff, moist, brown. | | | 2/4/4 | 26 | 82 | |
| 5 | | **CLAYEY SAND (SC):** fine-grained, few silt, slightly desiccated, orange oxidation swirls, medium stiff, moist, brownish gray. | | | 2/5/7 | 1 | 98 | CR |
| | | **SAND (SW):** fine to coarse-grained, trace silt, dark orange oxidation spots, loose, moist, gray. | | | 2/5/5 | 2 | 95 | |
| 10 | | **SAND WITH SILT (SP-SM):** fine-grained, orange oxidation swirls, loose, moist, grayish brown. | | | 3/5/6 | 8 | 95 | |

End of boring at 11.5 bgs.
No groundwater encountered.
Boring backfilled with soil cuttings and tamped with an auger using the weight of the drill rig on 09/25/2023.

# Log of Boring No. BH-06

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 753 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SAND (SP):** fine to medium-grained, trace silt, trace clay, slightly desiccated, trace oxidation swirls, moist, grayish brown.<br>- @2.5': loose. | | | 3/4/4 | 2 | | *disturbed* |
| 5 | | **CLAYEY SAND (SC):** fine-grained, pinhole porosity, slightly desiccated, dark orange oxidation swirls, soft, moist, greenish brown. | | | 2/2/3 | 27 | 85 | DS |
| | | **SANDY SILT (ML):** fine-grained sand, few clay, moderately desiccated, dark orange oxidation spots, micaceous, soft, moist, brown. | | | 2/2/3 | 47 | 75 | |
| 10 | | **SANDY CLAY (CL):** fine-grained sand, few silt, slightly, desiccated, trace oxidation swirls, micaceous, soft, moist, dark brown. | | | 2/2/3 | 49 | 71 | |

End of boring at 11.5 bgs.
No groundwater encountered.
Boring backfilled with soil cuttings and tamped with an auger using the weight of the drill rig on 09/25/2023.

# Log of Boring No. BH-07

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 754 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SAND WITH SILT (SP-SM):** fine to medium-grained, loose, moist, light grayish brown. | | | | | | |
| | | | X | | 3/3/3 | 14 | 95 | |
| 5 | | **SANDY CLAY (CL):** fine-grained sand, trace silt, pinhole porosity, very desiccated, dark orange oxidation swirls, black oxidation spots, medium stiff, moist, greenish brown. | X | X | 2/7/5 | 7 | 94 | CP |
| | | - @ 7.5': soft. | X | X | 2/3/3 | 8 | | *disturbed* |
| 10 | | - @ 10.0': medium stiff. | X | | 3/3/5 | 36 | 84 | |

End of boring at 11.5 bgs.
No groundwater encountered.
Boring backfilled with soil cuttings and tamped with an auger using the weight of the drill rig on 09/25/2023.

# Log of Boring No. BH-08

Date Drilled: 9/25/2023 Logged by: CATHERINE NELSON Checked By: HASHMI QUAZI

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 753 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered. | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE (%) | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ALLUVIUM**<br>**SAND WITH SILT (SP-SM):** fine to medium-grained, loose, moist, brown. | | | | | | |
| | | | | | 3/3/4 | 4 | 92 | R |
| 5 | | **SANDY SILT (ML):** fine-grained sand, trace clay, moderately desiccated, trace orange oxidation swirls, medium stiff, moist, brown. | | | 2/6/6 | 2 | | *disturbed* |
| | | **SAND WITH SILT (SP-SM):** fine to medium-grained, loose, moist, grayish brown. | | | 3/3/4 | 13 | 86 | |
| 10 | | - @10.0': pinhole porosity, slightly desiccated, heavy oxidation staining. | | | 3/3/3 | 3 | 90 | |

End of boring at 11.5 bgs.
No groundwater encountered.
Boring backfilled with soil cuttings and tamped with an auger using the weight of the drill rig on 09/25/2023.

# Appendix B

## Laboratory Testing Program

# APPENDIX B

# LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters required for this project. Test results are presented herein and on the *Logs of Borings*, in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

## ***In-Situ*** **Moisture Content and Dry Density**

In-situ dry density and moisture content tests were performed on relatively undisturbed ring samples, in accordance with ASTM Standard D2216 and D2937 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the site soils. For test results, see the *Logs of Borings* in Appendix A, *Field Exploration.*

## Expansion Index (EI)

One representative bulk sample was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result is presented in the following table.

**Table No. B-1, Expansion Index Test Result**

| Boring No. | Depth (feet) | Soil Description | Expansion Index | Expansion Potential |
|---|---|---|---|---|
| BH-01 | 0-5 | Silty Sand (SM) | 0 | Very Low |

## R-value (R)

One representative bulk soil sample was tested in accordance with California Test Method CT301 for resistance value (R-value). The test provides a relative measure of soil strength for use in pavement design. The test result is presented in the following table.

**Table No. B-2, R-Value Test Result**

| Boring No. | Depth (feet) | Soil Classification | Measured R-value |
|---|---|---|---|
| BH-08 | 0-4 | Sand with Silt (SP-SM) | 71 |

## Soil Corrosivity (CR)

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. The tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with Caltrans Test Methods 643, 422 and 417. The tests results are presented in the following table.

**Table No. B-3, Summary of Soil Corrosivity Test Results**

| Boring No. | Depth (feet) | pH | Soluble Sulfates (CA 417) (ppm) | Soluble Chlorides (CA 422) (ppm) | Min. Resistivity (CA 643) (Ohm-cm) |
|---|---|---|---|---|---|
| BH-01 | 0-5 | 8.2 | 307 | 171 | 956 |
| BH-05 | 5-10 | 8.5 | 100 | 75 | 1,637 |

## Grain-Size Analysis (PA)

To assist in classification of soils, mechanical grain-size analyses were performed on four representative samples in accordance with the ASTM Standard D6913 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results* and results are presented in the below table.

**Table No. B-4, Grain Size Distribution Test Results**

<table>
<tr><th>Boring No.</th><th>Depth (ft)</th><th>Soil Classification</th><th>% Gravel</th><th>% Sand</th><th>%Silt</th><th>%Clay</th></tr>
<tr><td>BH-01</td><td>0-5</td><td>Silty Sand (SM)</td><td>0.0</td><td>51.1</td><td colspan="2">48.9</td></tr>
<tr><td>BH-01</td><td>5-10</td><td>Sandy Silt (ML)</td><td>0.0</td><td>49.4</td><td colspan="2">50.6</td></tr>
<tr><td>BH-01</td><td>10-15</td><td>Silty Sand (SM)</td><td>0.0</td><td>58.0</td><td colspan="2">42.0</td></tr>
<tr><td>BH-01</td><td>25.0-26.5</td><td>Sand with Gravel (SW)</td><td>20.0</td><td>75.3</td><td colspan="2">4.7</td></tr>
</table>

## Maximum Density and Optimum Moisture Content (CP)

Laboratory maximum dry density-optimum moisture content relationship tests were performed on two representative bulk samples. The tests were conducted in accordance with the ASTM Standard D1557 test method. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Results,* and are summarized in the following table.

**Table No B-5, Summary of Moisture-Density Relationship Results**

| Boring No. | Depth (feet) | Soil Description | Optimum Moisture (%) | Maximum Density (lb/cft) |
|---|---|---|---|---|
| BH-01 | 0-5 | Silty Sand (SM), Dark Grayish Brown | 12.0 | 117.0 |
| BH-07 | 5-10 | Sandy Clay (CL), Greenish Brown | 11.0 | 122.0 |

## Direct Shear (DS)

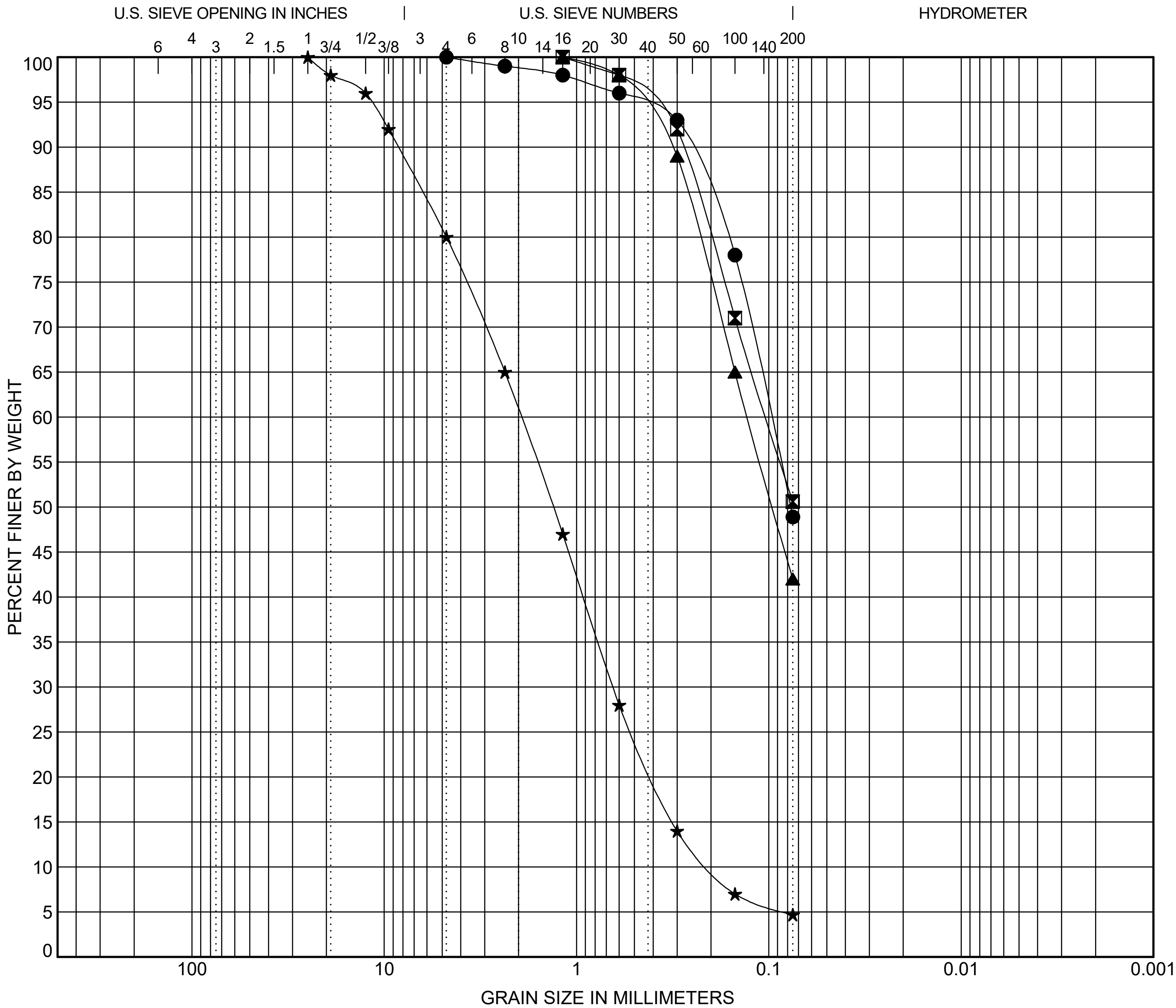
Three direct shear tests were performed on relatively undisturbed representative ring samples under soaked moisture condition in accordance with the ASTM D3080 procedure. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate between 0.004 and 0.010 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawings No. B-3 through B-5, *Direct Shear Test Results,* and the following table.

**Table No. B-6, Summary of Direct Shear Test Results**

| Boring No. | Depth (feet) | Soil Description | Ultimate Strength Parameters | |
|---|---|---|---|---|
| | | | Friction Angle (degrees) | Cohesion (psf) |
| BH-01 | 7.5-9.0 | Sandy Silt (ML) | 27 | 290 |
| BH-03 | 5.0-6.5 | Sand with Silt (SP-SM) | 27 | 120 |
| BH-06 | 5.0-6.5 | Clayey Sand (SC) | 30 | 50 |

## Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Boring No. | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **BH-01** | **0-5** | **SILTY SAND (SM)** | | | | | |
| ⧗ | **BH-01** | **5-10** | **SANDY SILT (ML)** | | | | | |
| ▲ | **BH-01** | **10-15** | **SILTY SAND (SM)** | | | | | |
| ★ | **BH-01** | **25.0-26.5** | **SAND WITH GRAVEL (SW)** | | | | **1.06** | **9.64** |
| | | | | | | | | |

| | Boring No. | Depth (ft) | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **BH-01** | **0-5** | **4.75** | **0.098** | | | **0.0** | **51.1** | **48.9** | |
| ⧗ | **BH-01** | **5-10** | **1.18** | **0.103** | | | **0.0** | **49.4** | **50.6** | |
| ▲ | **BH-01** | **10-15** | **1.18** | **0.129** | | | **0.0** | **58.0** | **42.0** | |
| ★ | **BH-01** | **25.0-26.5** | **25** | **1.947** | **0.644** | **0.202** | **20.0** | **75.3** | **4.7** | |
| | | | | | | | | | | |

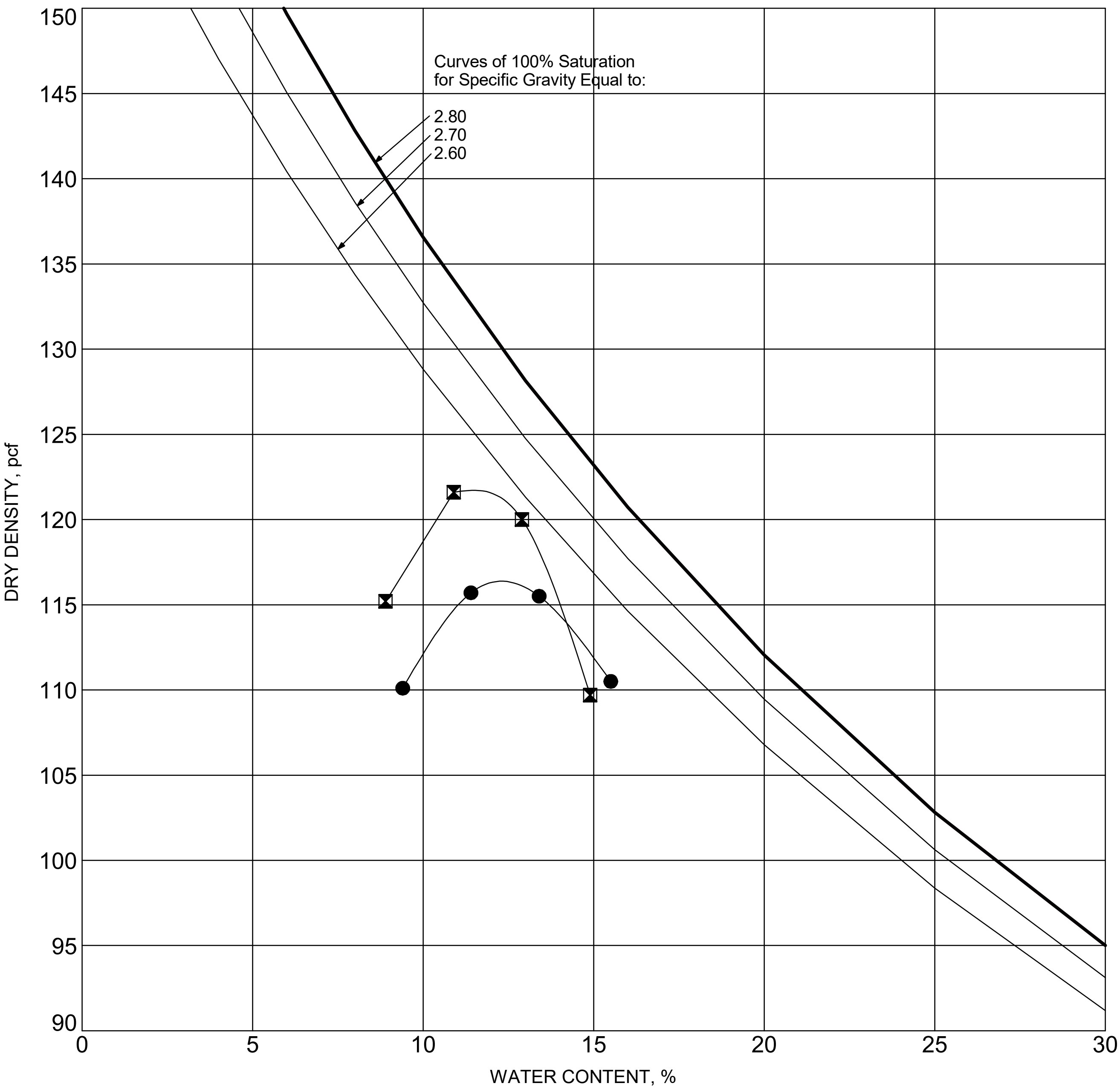
# GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Santa Ana River Bottom (SARB) Maintenance Facility
4600 Crestmore Road
City of Riverside, Riverside County, California
For: David Beckwith and Associates, Inc.

Project No. **23-81-234-01**

Drawing No. **B-1**




| SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf |
|---|---|---|---|---|---|---|
| ● | **BH-01** | **0-5** | **SILTY SAND (SM), DARK GRAYISH BROWN** | **D1557 Method A** | **12.0** | **117.0** |
| ⧗ | **BH-07** | **5-10** | **SANDY CLAY (CL), GREENISH BROWN** | **D1557 Method A** | **11.0** | **122.0** |
| | | | | | | |
| | | | | | | |
| | | | | | | |

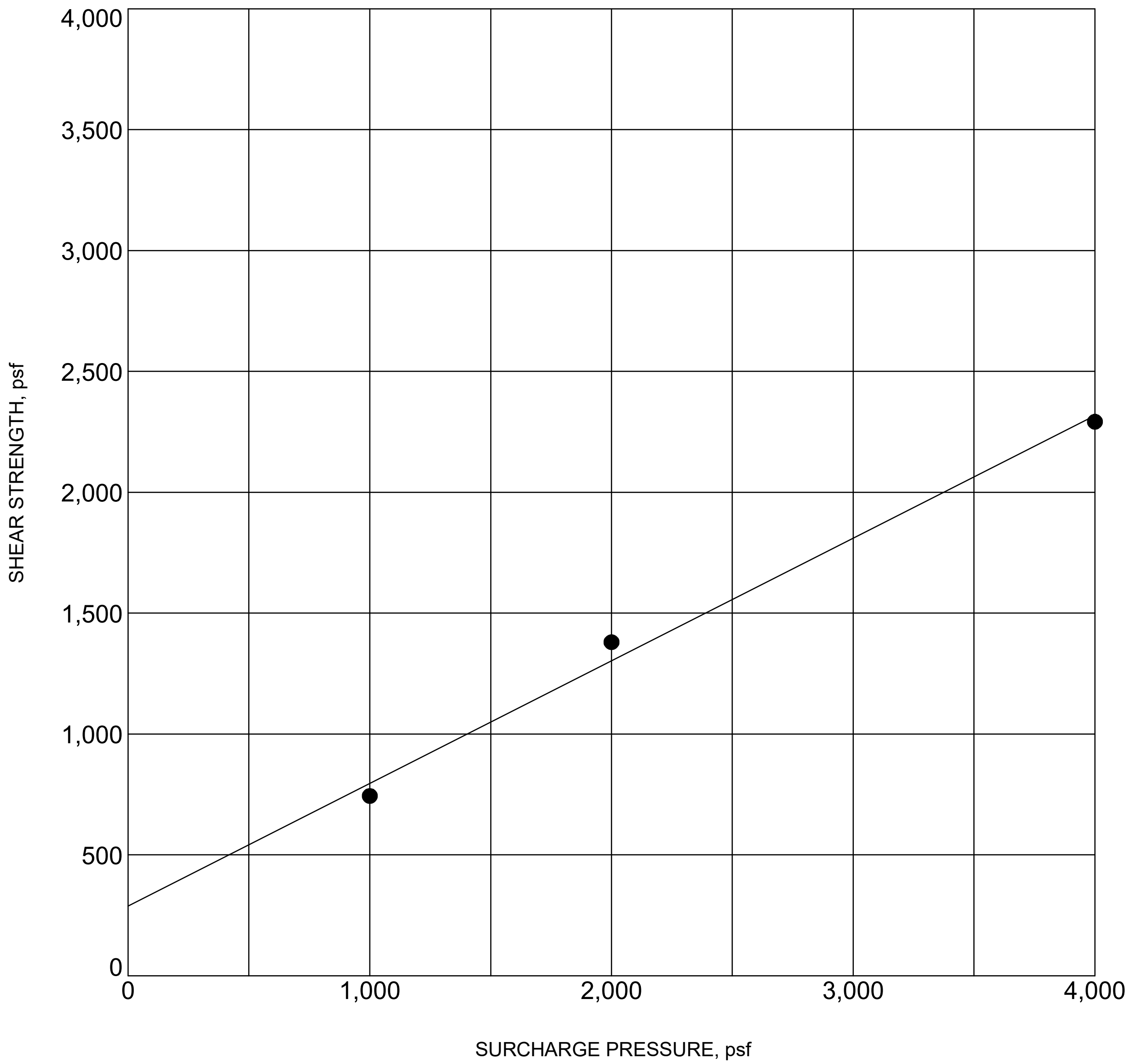
# MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Santa Ana River Bottom (SARB) Maintenance Facility
4600 Crestmore Road
City of Riverside, Riverside County, California
For: David Beckwith and Associates, Inc.

Project No. **23-81-234-01**

Drawing No. **B-2**




| | | | |
|---|---|---|---|
| BORING NO. : | **BH-01** | DEPTH (ft) : | **7.5-9.0** |
| DESCRIPTION : | **SANDY SILT (ML)** | | |
| COHESION (psf) : | **290** | FRICTION ANGLE (degrees): | **27** |
| MOISTURE CONTENT (%) : | **26** | DRY DENSITY (pcf) : | **89** |

NOTE: Ultimate Strength.

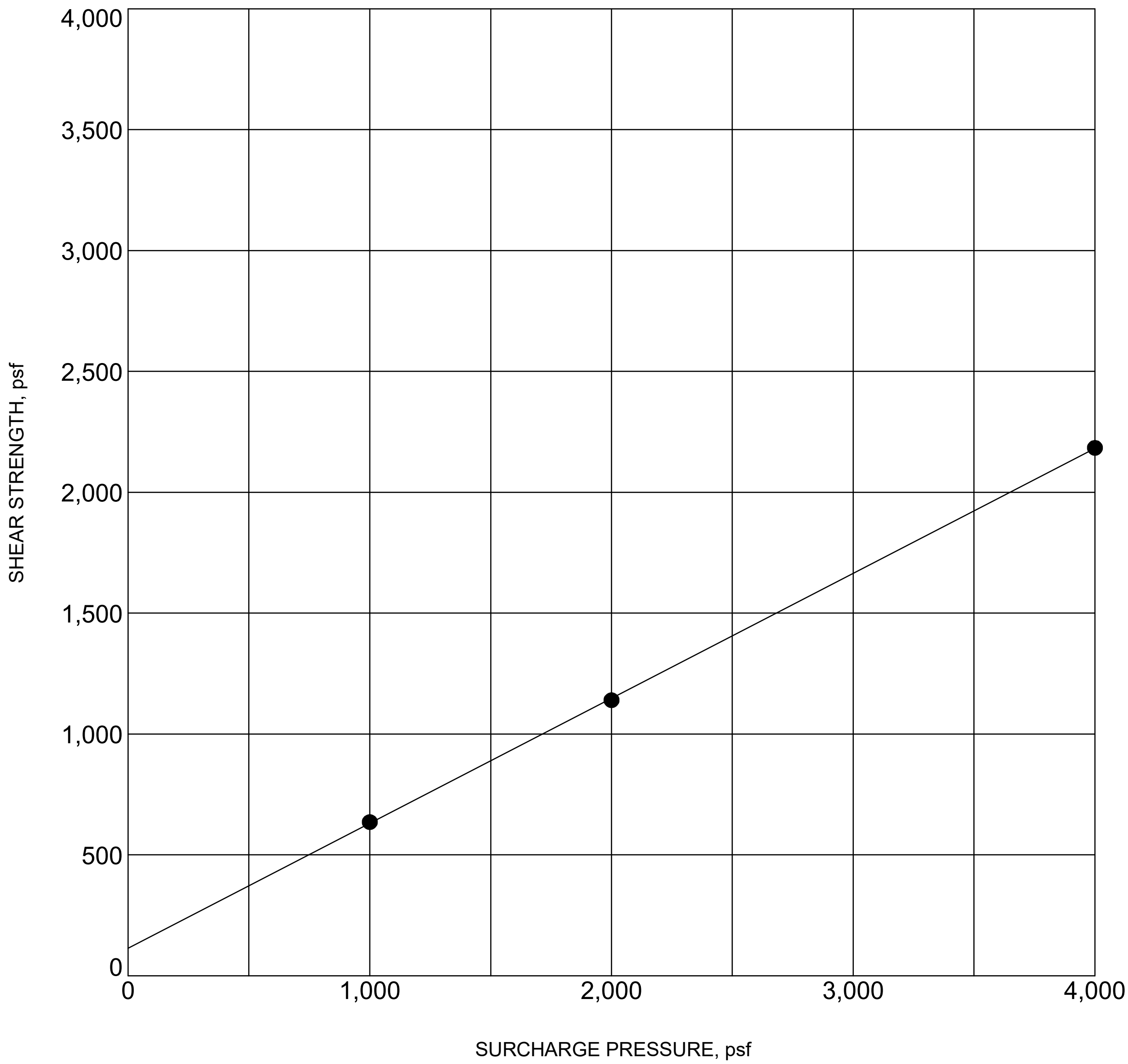
# DIRECT SHEAR TEST RESULTS

Converse Consultants

Santa Ana River Bottom (SARB) Maintenance Facility
4600 Crestmore Road
City of Riverside, Riverside County, California
For: David Beckwith and Associates, Inc.

Project No. **23-81-234-01**

Drawing No. **B-3**




| | | | | | |
|---|---|---|---|---|---|
| BORING NO. | : | **BH-03** | DEPTH (ft) | : | **5.0-6.5** |
| DESCRIPTION | : | **SAND WITH SILT (SP-SM)** | | | |
| COHESION (psf) | : | **120** | FRICTION ANGLE (degrees): | | **27** |
| MOISTURE CONTENT (%) | : | **21** | DRY DENSITY (pcf) | : | **85** |

NOTE: Ultimate Strength.

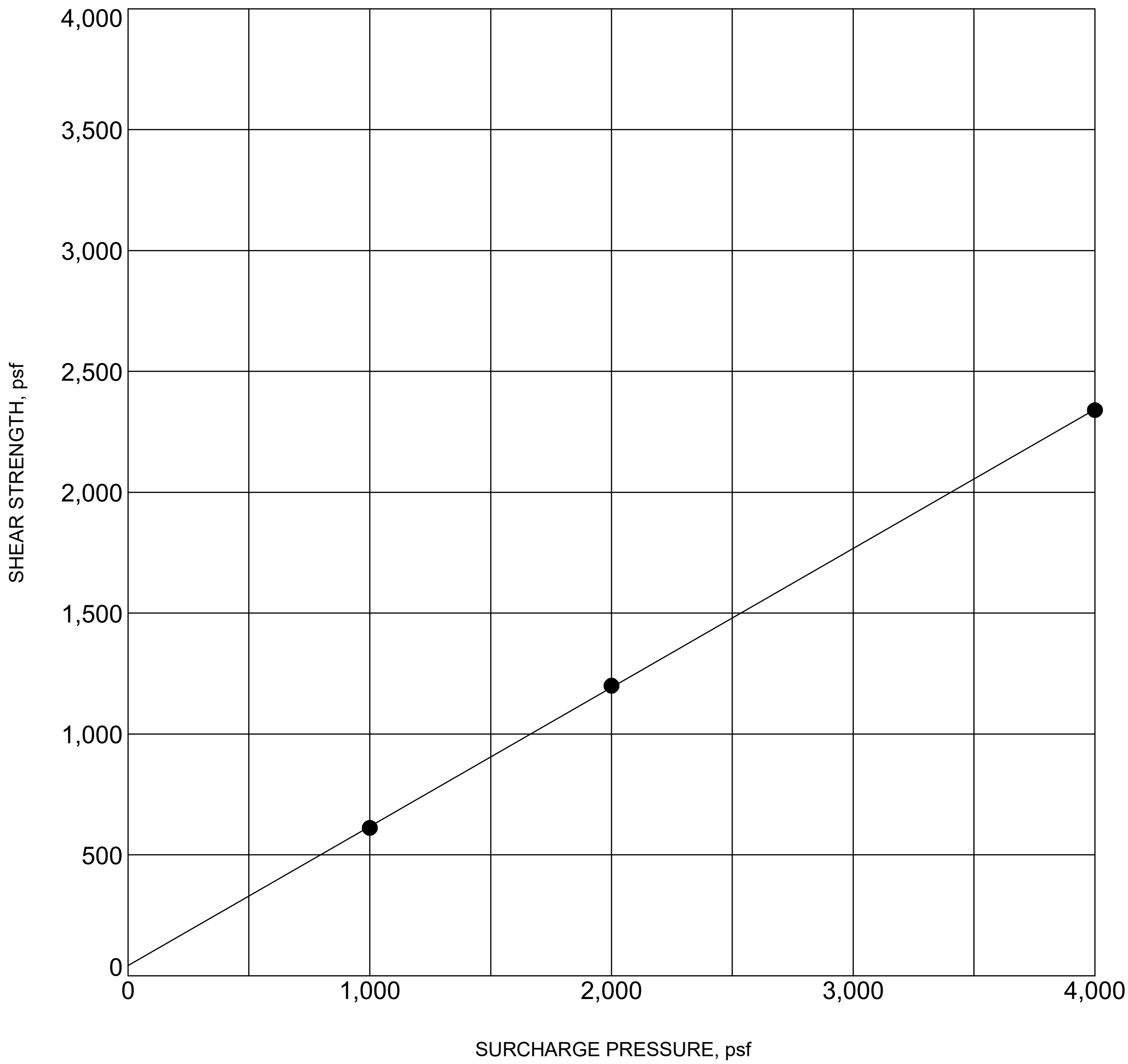
# DIRECT SHEAR TEST RESULTS

Converse Consultants

Santa Ana River Bottom (SARB) Maintenance Facility
4600 Crestmore Road
City of Riverside, Riverside County, California
For: David Beckwith and Associates, Inc.

Project No. **23-81-234-01**

Drawing No. **B-4**




| | | | |
|---|---|---|---|
| BORING NO. : | **BH-06** | DEPTH (ft) : | **5.0-6.5** |
| DESCRIPTION : | **CLAYEY SAND (SC)** | | |
| COHESION (psf) : | **50** | FRICTION ANGLE (degrees): | **30** |
| MOISTURE CONTENT (%) : | **27** | DRY DENSITY (pcf) : | **85** |

NOTE: Ultimate Strength.

# DIRECT SHEAR TEST RESULTS

Converse Consultants

Santa Ana River Bottom (SARB) Maintenance Facility
4600 Crestmore Road
City of Riverside, Riverside County, California
For: David Beckwith and Associates, Inc.

Project No. **23-81-234-01**

Drawing No. **B-5**

# Appendix C

## Liquefaction and Settlement Analysis

# APPENDIX C

## LIQUEFACTION AND SETTLEMENT ANALYSIS

The subsurface data obtained from the boring BH-01 was used to evaluate the liquefaction potential and associated dry seismic settlement when subjected to ground shaking during earthquakes.

A simplified liquefaction hazard analysis was performed using the program SPTLIQ (InfraGEO Software, 2020) using the liquefaction triggering analysis method by Boulanger and Idriss (2014). A modal earthquake magnitude of M 8.11 was selected based on the results of seismic deaggregation analysis using the USGS interactive online tool (https://earthquake.usgs.gov/hazards/interactive/).

A peak ground acceleration ($PGA_M$) of 0.550g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA was based on the CBC seismic design parameters presented in Section 7.2, *CBC 2022 Seismic Parameters.*

The result of our analysis is presented on Plates No. C-1 through C-3 and summarized in the following table.

**Table C-1, Estimated Dynamic Settlements**

| Location | Groundwater Conditions | Groundwater Depth (feet bgs) | Dry Seismic Settlement (inches) | Liquefaction Induced Settlement (inches) |
|---|---|---|---|---|
| BH-01 | Current | 12.8 | 0.99 | 3.26 |
| | Historical | > 11.0 | | |

Based on our analysis, the project site has the potential for up to 1.69 inches of dry seismic settlement and 4.51 inches of liquefaction induced settlement.

# SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA



## PROJECT INFORMATION

| | |
|---|---|
| Project Name | Santa Ana River Bottom (SARB) Maintenance Facility |
| Project No. | 23-81-234-01 |
| Project Location | City of Riverside, Riverside County, California |
| Analyzed By | Aleksey Zhukov |
| Reviewed By | Hashmi Quazi |

## SELECTED METHODS OF ANALYSIS

| | |
|---|---|
| Analysis Description | Liquefaction |
| Triggering of Liquefaction | Boulanger-Idriss (2014) |
| Severity of Liquefaction | LPI: Liquefaction Potential Index based on Iwasaki et al. (1978) |
| Seismic Compression Settlement (Dry/Unsaturated Soil) | Pradel (1998) |
| Liquefaction-Induced Settlement (Saturated Soil) | Tokimatsu and Seed (1987) |
| Liquefaction-Induced Lateral Spreading | Zhang et al. (2004) |
| Residual Shear Strength of Liquefied Soil | Idriss and Boulanger (2008) |

## SEISMIC DESIGN PARAMETERS

| | |
|---|---|
| Earthquake Moment Magnitude, $M_w$ | 8.11 |
| Peak Ground Acceleration, $A_{max}$ | 0.55 g |
| Factor of Safety Against Liquefaction, FS | 1.20 |

## BORING DATA AND SITE CONDITIONS

| | |
|---|---|
| Boring No. | BH-01 |
| Ground Surface Elevation | 753.00 feet |
| Proposed Grade Elevation | 753.00 feet |
| GWL Depth Measured During Test | 12.80 feet |
| GWL Depth Used in Design | 11.00 feet |
| Borehole Diameter | 8.00 inches |
| Hammer Weight | 140.00 pounds |
| Hammer Drop | 30.00 inches |
| Hammer Energy Efficiency Ratio, ER (%) | 86.00 % |
| Hammer Distance to Ground Surface | 5.00 feet |
| Topographic Site Condition: | TSC1 (Level Ground with No Nearby Free Face) |
| - Ground Slope, S (%) | <<= Leave this blank |
| - Free Face Distance to Slope Height Ratio, (L/H) | <<= Leave this blank Set H to zero =>> 0.00 feet |

## INPUT SOIL PROFILE DATA

| Depth to Top of Soil Layer (feet) | Depth to Bottom of Soil Layer (feet) | Material Type USCS Group Symbol (ASTM D2487) | Liquefaction Screening *Susceptible Soil? (Y, N)* | Total Soil Unit Weight $\gamma_t$ (pcf) | Type of Soil Sampler | Field Blow Count $N_{field}$ (blows/ft) | Fines Content FC (%) |
|---|---|---|---|---|---|---|---|
| 0.00 | 5.00 | SM | Y | 97.7 | MCal | 4.0 | 48.90 |
| 5.00 | 7.50 | ML | Y | 110.2 | MCal | 8.0 | 50.60 |
| 7.50 | 10.00 | ML | Y | 112.1 | MCal | 7.0 | 50.60 |
| 10.00 | 15.00 | SM | Y | 96.5 | MCal | 10.0 | 42.00 |
| 15.00 | 20.00 | SM | Y | 104.8 | MCal | 10.0 | 42.00 |
| 20.00 | 25.00 | SM | Y | 104.8 | SPT1 | 12.0 | 42.00 |
| 25.00 | 30.00 | SW | Y | 132.1 | MCal | 39.0 | 4.70 |
| 30.00 | 35.00 | SP | Y | 130.0 | SPT1 | 27.0 | 4.70 |
| 35.00 | 40.00 | SP | Y | 124.1 | MCal | 64.0 | 4.70 |
| 40.00 | 45.00 | SP | Y | 124.1 | SPT1 | 32.0 | 4.70 |
| 45.00 | 50.00 | SP | Y | 125.3 | MCal | 61.0 | 4.70 |
| 50.00 | 51.50 | SP | Y | 125.3 | SPT1 | 20.0 | 4.70 |

# SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA



## PROJECT INFORMATION

| | |
|---|---|
| **Project Name** | Santa Ana River Bottom (SARB) Maintenance Facility |
| **Project No.** | 23-81-234-01 |
| **Project Location** | City of Riverside, Riverside County, California |
| **Analyzed By** | Aleksey Zhukov |
| **Reviewed By** | Hashmi Quazi |

## SEISMIC DESIGN PARAMETERS

| | |
|---|---|
| **Earthquake Moment Magnitude, $M_w$** | 8.11 |
| **Peak Ground Acceleration, $A_{max}$** | 0.55 g |
| **Factor of Safety Against Liquefaction, FS** | 1.20 |

## BORING DATA AND SITE CONDITIONS

| | |
|---|---|
| **Boring No.** | BH-01 |
| **Ground Surface Elevation** | 753.00 feet |
| **Proposed Grade Elevation** | 753.00 feet |
| **GWL Depth Measured During Test** | 12.80 feet |
| **GWL Depth Used in Design** | 11.00 feet |
| **Borehole Diameter** | 8.00 inches |
| **Hammer Weight** | 140.00 pounds |
| **Hammer Drop** | 30.00 inches |
| **Hammer Energy Efficiency Ratio, ER** | 86.00 % |
| **Hammer Distance to Ground Surface** | 5.00 feet |
| **Topographic Site Condition:** | TSC1 (Level Ground with No Nearby Free Face) |
| **- Ground Slope, S** | 0.00 % |
| **- Free Face (L/H) Ratio** | N/A H = 0 feet |
| **Average Total Unit Weight of New Fill** | 120.00 pcf (assumed) |

## SUMMARY OF RESULTS

**Severity of Liquefaction:**

Total Thickness of Liquefiable Soils: 16.50 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 25.94 *** (Very high risk, with major liquefaction effects)

| **Seismic Ground Settlements:** | **Analysis Method** | **Upper 30 feet** | **Upper 50 feet** | **Upper 65 feet** | |
|---|---|---|---|---|---|
| Seismic Compression Settlement: | Pradel (1998) | 0.99 inches | 0.99 inches | 0.99 inches | (Dry/Unsaturated Soils) |
| Liquefaction-Induced Settlement: | Tokimatsu and Seed (1987) | 3.08 inches | 3.08 inches | 3.26 inches | (Saturated Soils) |
| Total Seismic Settlement: | | 4.07 inches | 4.07 inches | 4.25 inches | |

| **Seismic Lateral Displacements:** | **Analysis Method** | **Upper 30 feet** | **Upper 50 feet** | **Upper 65 feet** | |
|---|---|---|---|---|---|
| Cyclic Lateral Displacement: | Tokimatsu and Asaka (1998) | 1.75 inches | 1.75 inches | 1.80 inches | (During Ground Shaking) |
| Lateral Spreading Displacement: | Zhang et al. (2004) | 0.00 inches | 0.00 inches | 0.00 inches | (After Ground Shaking) |

## NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\ C_N\ C_E\ C_B\ C_R\ C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008).

* $FS_{liq}$ = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7.5}$ MSF $K_\sigma$ $K_\alpha$ , MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, $K_\alpha$ =1.0, (level ground), CSR = Cyclic Stress Ratio = 0.65 $A_{max}$ $(\sigma_{vo}/\sigma'_{vo})$ $r_d$ , and $CRR_{7.5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude $M_w$ of 7.5.

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008).

*** Based on Iwasaki et al. (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.

## INPUT SOIL PROFILE DATA

| Depth to Top of Soil Layer (feet) | Depth to Bottom of Soil Layer (feet) | Material Type USCS Group Symbol (ASTM D2487) | Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N) | Total Soil Unit Weight $\gamma_t$ (pcf) | Type of Soil Sampler | Field SPT Blow Count $N_{field}$ (blows/ft) | Fines Content FC (%) |
|---|---|---|---|---|---|---|---|
| 0.00 | 5.00 | SM | Y | 97.70 | MCal | 4.00 | 48.90 |
| 5.00 | 7.50 | ML | Y | 110.20 | MCal | 8.00 | 50.60 |
| 7.50 | 10.00 | ML | Y | 112.10 | MCal | 7.00 | 50.60 |
| 10.00 | 15.00 | SM | Y | 96.50 | MCal | 10.00 | 42.00 |
| 15.00 | 20.00 | SM | Y | 104.80 | MCal | 10.00 | 42.00 |
| 20.00 | 25.00 | SM | Y | 104.80 | SPT1 | 12.00 | 42.00 |
| 25.00 | 30.00 | SW | Y | 132.10 | MCal | 39.00 | 4.70 |
| 30.00 | 35.00 | SP | Y | 130.00 | SPT1 | 27.00 | 4.70 |
| 35.00 | 40.00 | SP | Y | 124.10 | MCal | 64.00 | 4.70 |
| 40.00 | 45.00 | SP | Y | 124.10 | SPT1 | 32.00 | 4.70 |
| 45.00 | 50.00 | SP | Y | 125.30 | MCal | 61.00 | 4.70 |
| 50.00 | 51.50 | SP | Y | 125.30 | SPT1 | 20.00 | 4.70 |

## LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

| Depth to Top of Soil Layer (feet) | Total Vert. Stress (Design) $\sigma_{vo}$ (psf) | Effective Vert. Stress (Design) $\sigma'_{vo}$ (psf) | SPT Corr. for Vert. Stress $C_N$ | SPT Corr. for Hammer Energy $C_E$ | SPT Corr. for Borehole Size $C_B$ | SPT Corr. for Rod Length $C_R$ | SPT Corr. for Sampling Method $C_S$ | Corrected SPT Blow Count $N_{60}$ | Normalized SPT Blow Count $(N_1)_{60}$ | Fines Corrected SPT Blow Count $(N_1)_{60cs}$ | Shear Stress Reduction Coefficient $r_d$ | Correction for High Overburden Stress $K_\sigma$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 244.25 | 244.25 | 1.700 | 1.433 | 1.150 | 0.750 | 0.650 | 3.2 | 5.5 | 11.1 | 1.000 | 1.100 |
| 5.00 | 626.25 | 626.25 | 1.700 | 1.433 | 1.150 | 0.800 | 0.650 | 6.9 | 11.7 | 17.3 | 0.997 | 1.100 |
| 7.50 | 904.13 | 904.13 | 1.539 | 1.433 | 1.150 | 0.850 | 0.650 | 6.4 | 9.8 | 15.4 | 0.993 | 1.073 |
| 10.00 | 1,285.50 | 1,160.70 | 1.295 | 1.433 | 1.150 | 0.850 | 0.650 | 9.1 | 11.8 | 17.4 | 0.986 | 1.049 |
| 15.00 | 1,788.75 | 1,383.15 | 1.163 | 1.433 | 1.150 | 0.950 | 0.650 | 10.2 | 11.8 | 17.4 | 0.976 | 1.029 |
| 20.00 | 2,312.75 | 1,595.15 | 1.072 | 1.433 | 1.150 | 0.950 | 1.000 | 18.8 | 20.1 | 25.7 | 0.965 | 1.021 |
| 25.00 | 2,905.00 | 1,875.40 | 1.002 | 1.433 | 1.150 | 0.950 | 0.650 | 39.7 | 39.8 | 39.8 | 0.953 | 1.002 |
| 30.00 | 3,560.25 | 2,218.65 | 0.958 | 1.433 | 1.150 | 1.000 | 1.000 | 44.5 | 42.6 | 42.6 | 0.940 | 0.954 |
| 35.00 | 4,195.50 | 2,541.90 | 0.955 | 1.433 | 1.150 | 1.000 | 0.650 | 68.6 | 65.5 | 65.5 | 0.925 | 0.915 |
| 40.00 | 4,816.00 | 2,850.40 | 0.905 | 1.433 | 1.150 | 1.000 | 1.000 | 52.7 | 47.7 | 47.7 | 0.910 | 0.882 |
| 45.00 | 5,439.50 | 3,161.90 | 0.910 | 1.433 | 1.150 | 1.000 | 0.650 | 65.4 | 59.5 | 59.5 | 0.895 | 0.852 |
| 50.00 | 5,846.73 | 3,366.33 | 0.807 | 1.433 | 1.150 | 1.000 | 1.000 | 33.0 | 26.6 | 26.6 | 0.884 | 0.904 |

| Depth to Top of Soil Layer (feet) | Cyclic Stress Ratio CSR | Cyclic Resistance Ratio CRR | Factor of Safety * $FS_{liq}$ | Liquefaction Analysis Results | Residual Shear Strength ** $S_r$ (psf) | Seismic Porewater Pressure Ratio $r_u$ (%) | Cumulative Seismic Settlement (inches) | Cumulative Cyclic Lateral Displacement (inches) | Cumulative Lateral Spreading Displacement (inches) |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.358 | | | Unsaturated Soil | | | 4.25 | 1.80 | 0.00 |
| 5.00 | 0.356 | | | Unsaturated Soil | | | 3.51 | 1.68 | 0.00 |
| 7.50 | 0.355 | | | Unsaturated Soil | | | 3.41 | 1.64 | 0.00 |
| 10.00 | 0.391 | 0.173 | 0.44 | LIQUEFY | 150.75 | 100.00 | 3.26 | 1.60 | 0.00 |
| 15.00 | 0.451 | 0.170 | 0.38 | LIQUEFY | 178.78 | 100.00 | 2.12 | 0.99 | 0.00 |
| 20.00 | 0.500 | 0.270 | 0.54 | LIQUEFY | 361.76 | 100.00 | 0.98 | 0.29 | 0.00 |
| 25.00 | 0.528 | | | Dense Soil | | | 0.18 | 0.05 | 0.00 |
| 30.00 | 0.539 | | | Dense Soil | | | 0.18 | 0.05 | 0.00 |
| 35.00 | 0.546 | | | Dense Soil | | | 0.18 | 0.05 | 0.00 |
| 40.00 | 0.550 | | | Dense Soil | | | 0.18 | 0.05 | 0.00 |
| 45.00 | 0.550 | | | Dense Soil | | | 0.18 | 0.05 | 0.00 |
| 50.00 | 0.549 | 0.256 | 0.47 | LIQUEFY | 1,124.04 | 100.00 | 0.18 | 0.05 | 0.00 |

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# SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA



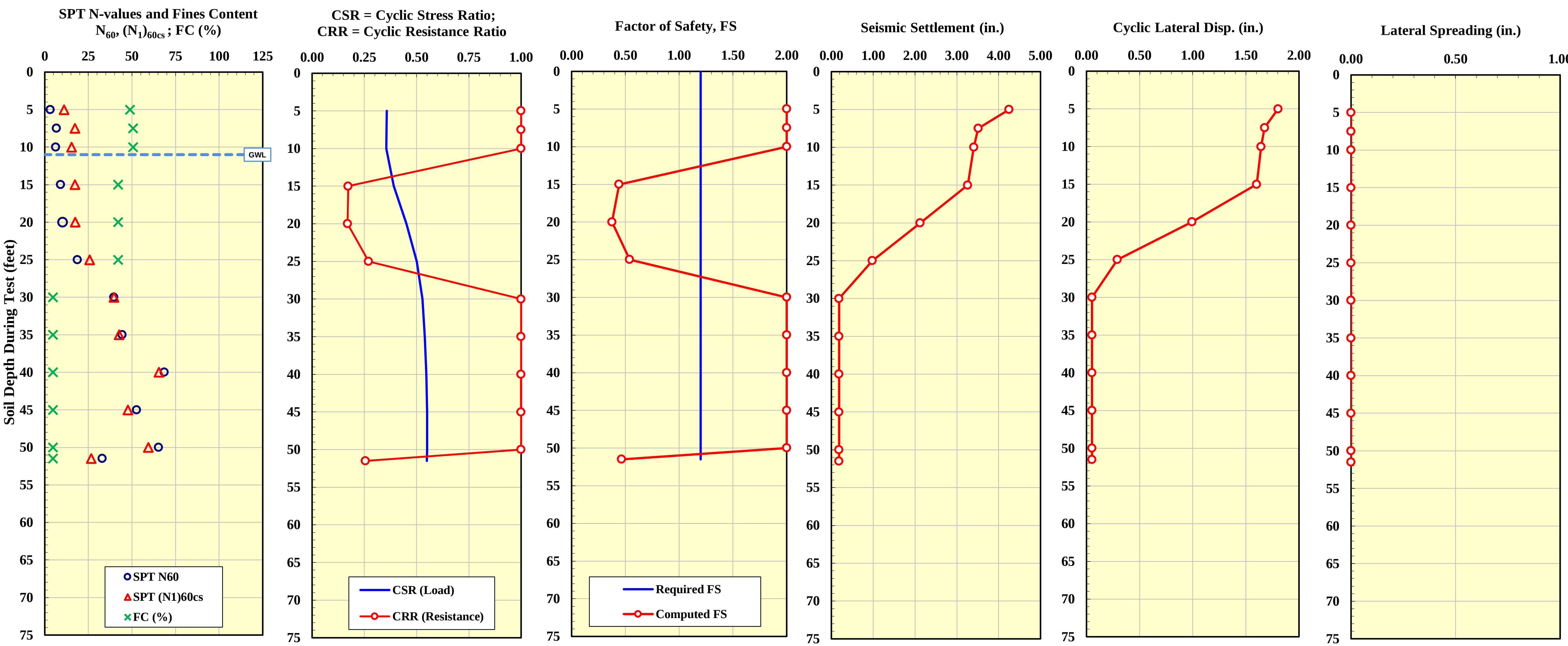
| PROJECT INFORMATION | |
|---|---|
| Project Name | Santa Ana River Bottom (SARB) Maintenance Facility |
| Project No. | 23-81-234-01 |
| Project Location | City of Riverside, Riverside County, California |
| Analyzed By | Aleksey Zhukov |
| Reviewed By | Hashmi Quazi |

| TOPOGRAPHIC CONDITIONS | | |
|---|---|---|
| Ground Slope, S | 0.00 % | |
| Free Face (L/H) Ratio | N/A | H = 0.00 feet |

| GROUNDWATER DATA | |
|---|---|
| GWL Depth Measured During Test | 12.80 feet |
| GWL Depth Used in Design | 11.00 feet |

| BORING DATA | |
|---|---|
| Boring No. | BH-01 |
| Ground Surface Elevation | 753.00 feet |
| Proposed Grade Elevation | 753.00 feet |
| Borehole Diameter | 8.00 inches |
| Hammer Weight | 140.00 pounds |
| Hammer Drop | 30.00 inches |
| Hammer Energy Efficiency Ratio, ER | 86.00 % |
| Hammer Distance to Ground Surface | 5.00 feet |

| SEISMIC DESIGN PARAMETERS | |
|---|---|
| Earthquake Moment Magnitude, $M_w$ | 8.11 |
| Peak Ground Acceleration, $A_{max}$ | 0.55 g |
| Factor of Safety Against Liquefaction, FS | 1.20 |




**Analysis Methods Used ==>>**

**Liquefaction Triggering:**
Boulanger-Idriss (2014)

**Seismic Settlements:**
Above GWL: Pradel (1998)
Below GWL: Tokimatsu and Seed (1987)

**Cyclic Lateral Displacements:**
Above GWL: Pradel (1998)
Below GWL: Tokimatsu and Asaka (1998)

**Lateral Spreading:**
Zhang et al. (2004)

# Appendix D

## Percolation Testing

# APPENDIX D

## PERCOLATION TESTING

Percolation testing was performed at two locations (BH-02/PT-01 and BH-04/PT-02) on September 29, 2023, in general accordance with the Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011).

Upon completion of drilling the test holes, a 2-inch-thick gravel layer was placed at the bottom and a 3.0-inch diameter perforated pipe was installed above the gravel to the ground surface. The boring annulus around the pipe was filled with gravel. The purpose of the pipe and gravel was to reduce the potential erosion and caving due to the addition of water to the hole.

The test holes were presoaked by filling with water to at least 5 times the radius of the test hole. More than 6 inches of water seeped away from the test holes BH-02/PT-01 and BH-04/PT-02 within 25 minutes for 2 consecutive measurements, therefore the criteria for a "sandy soil" test were utilized. During testing, the water level and total depth of the test hole were measured from the top of the pipe every 10 minutes for up to 2 hours. The water level was refilled to the same measurement following each reading and before the next test is started. Following completion of percolation testing, the pipe was removed, and the percolation test hole was backfilled with excavated soil and compacted.

Percolation rates describe the movement of water horizontally and downward into the soil from a boring. Infiltration rates describe the downward movement of water through a horizontal surface, such as the floor of a retention basin. Percolation rates are related to infiltration rates but are generally higher and require conversion before use in basin design. The percolation test data was used to estimate infiltration rates using the Porchet Inverse Borehole Method, in accordance with the Riverside County guidelines. A factor of safety of 3 was applied to the measured infiltration rates to account for subsurface variations, uncertainty in the test method, and future siltation. The infiltration structure designer should determine whether additional design-related safety factors are appropriate.

The measured percolation test data and calculations for conversion to infiltration rates, porosity correction, and factor of safety are shown on Plates No. 1 and 3, *Estimated Infiltration Rate from Percolation Test Data* are graphically represented on Plates No. 2 and 4, *Infiltration Rate Versus Elapsed Time* in Appendix D, *Percolation Testing*. The estimated infiltration rate at the test hole is presented in the following table.

**Table No. D-1, Estimated Infiltration Rate**

| Infiltration Test | Depth (feet) | Soil Type | Infiltration Rate (inches/hour) |
|---|---|---|---|
| PT-01 | 10.2 | Silty Sand (SM) and Sand with Silt (SP-SM) | 1.31 |
| PT-02 | 5.7 | Silty Sand (SM) and Sand with Silt (SP-SM) | 0.54 |

Based on the test data, the infiltration rate of 1.31 (inches/hour) is for a depth of 10.2 feet bgs. and the infiltration rate of 0.54 (inches/hour) is for a depth of 5.7 feet bgs. Design infiltration rate should be selected based on infiltration structure depth and soil type at that depth. A factor of safety of 3 was applied to the measured infiltration rate to account for subsurface variations, uncertainty in the test method, and future siltation. Please note that infiltration rates may change if the soil type and location of the proposed system changes. If that is the case, then additional percolation testing should be performed in the required location.

**Estimated Infiltration Rate from Percolation Test Data, PT-01**

| | |
|---|---|
| Project Name | Santa Ana River Bottom (SARB) Maintenance Facility |
| Project Number | 23-81-234-01 |
| Test Number | BH-02/PT-01 |
| Test Location | Northeastern Portion of the Site |
| Personnel | Javier Calzada |
| Presoak Date | 9/28/2023 |
| Test Date | 9/29/2023 |

Shaded cells contain calculated values.

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 122 |
| Inside Diameter of Pipe, I (inches) | 3.00 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| | |
| Factor of Safety (FOS), F | 3 |

| Interval No. | Time Interval, $\Delta t$ (min) | Initial Depth to Water, $D_0$ (inches) | Final Depth to Water, $D_f$ (inches) | Elapsed Time (min) | Initial Height of Water, $H_0$ (inches) | Final Height of Water, $H_f$ (inches) | Change in Height of Water, $\Delta H$ (inches) | Average Head Height, $H_{avg}$ (inches) | Infiltration Rate, $I_t$ (inches/hr) | | Infiltration Rate with FOS, $I_f$ (inches/hr) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 0 | | | | | | | 0 |
| 1 | 25.00 | 60.00 | 99.36 | 25.00 | 62.00 | 22.64 | 39.36 | 42.32 | 4.26 | | 1.42 |
| 2 | 25.00 | 60.00 | 91.80 | 50.00 | 62.00 | 30.20 | 31.80 | 46.10 | 3.17 | | 1.06 |
| 3 | 10.00 | 60.00 | 82.32 | 60.00 | 62.00 | 39.68 | 22.32 | 50.84 | 5.07 | | 1.69 |
| 4 | 10.00 | 60.00 | 81.72 | 70.00 | 62.00 | 40.28 | 21.72 | 51.14 | 4.90 | | 1.63 |
| 5 | 10.00 | 60.00 | 79.44 | 80.00 | 62.00 | 42.56 | 19.44 | 52.28 | 4.30 | | 1.43 |
| 6 | 10.00 | 60.00 | 78.00 | 90.00 | 62.00 | 44.00 | 18.00 | 53.00 | 3.93 | | 1.31 |
| 7 | 10.00 | 60.00 | 78.00 | 100.00 | 62.00 | 44.00 | 18.00 | 53.00 | 3.93 | | 1.31 |
| 8 | 10.00 | 60.00 | 78.00 | 110.00 | 62.00 | 44.00 | 18.00 | 53.00 | 3.93 | | 1.31 |
| 9 | | | | | | | | | | | |
| 10 | | | | | | | | | | | |
| 11 | | | | | | | | | | | |
| 12 | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **1.31** |

San Bernardino County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2013)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg}))$

**Plate No.**
1

**Infiltration Rate versus Time, PT-01**

| | |
|---|---|
| Project Name | Santa Ana River Bottom (SARB) Maintenance Facility |
| Project Number | 23-81-234-01 |
| Test Number | BH-02/PT-01 |
| Test Location | Northeastern Portion of the Site |
| Personnel | Javier Calzada |
| Presoak Date | 9/28/2023 |
| Test Date | 9/29/2023 |

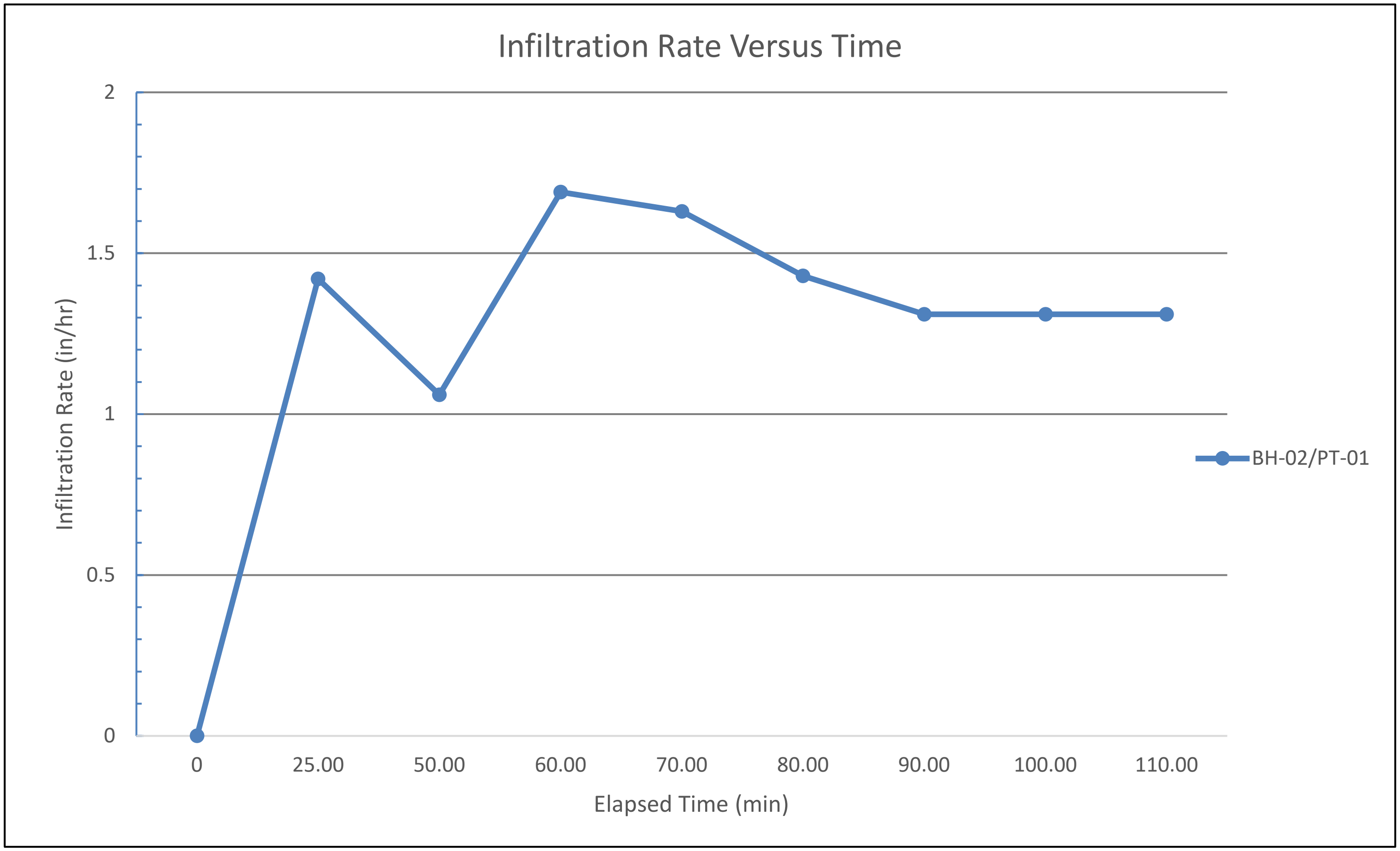



**Plate No.**
2

**Estimated Infiltration Rate from Percolation Test Data, PT-02**

Shaded cells contain calculated values.

| | |
|---|---|
| Project Name | Santa Ana River Bottom (SARB) Maintenance Facility |
| Project Number | 23-81-234-01 |
| Test Number | BH-04/PT-02 |
| Test Location | Southern Portion of Site |
| Personnel | Javier Calzada |
| Presoak Date | 9/28/2023 |
| Test Date | 9/29/2023 |

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 68 |
| Inside Diameter of Pipe, I (inches) | 3.00 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| | |
| Factor of Safety (FOS), F | 3 |

| Interval No. | Time Interval, $\Delta t$ (min) | Initial Depth to Water, $D_0$ (inches) | Final Depth to Water, $D_f$ (inches) | Elapsed Time (min) | Initial Height of Water, $H_0$ (inches) | Final Height of Water, $H_f$ (inches) | Change in Height of Water, $\Delta H$ (inches) | Average Head Height, $H_{avg}$ (inches) | Infiltration Rate, $I_t$ (inches/hr) | | Infiltration Rate with FOS, $I_f$ (inches/hr) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 0 | | | | | | | 0 |
| 1 | 25.00 | 18.00 | 43.92 | 25.00 | 50.00 | 24.08 | 25.92 | 37.04 | 3.19 | | 1.06 |
| 2 | 25.00 | 18.00 | 36.24 | 50.00 | 50.00 | 31.76 | 18.24 | 40.88 | 2.04 | | 0.68 |
| 3 | 10.00 | 18.00 | 27.48 | 60.00 | 50.00 | 40.52 | 9.48 | 45.26 | 2.41 | | 0.80 |
| 4 | 10.00 | 18.00 | 24.72 | 70.00 | 50.00 | 43.28 | 6.72 | 46.64 | 1.66 | | 0.55 |
| 5 | 10.00 | 18.00 | 25.20 | 80.00 | 50.00 | 42.80 | 7.20 | 46.40 | 1.79 | | 0.60 |
| 6 | 10.00 | 18.00 | 24.84 | 90.00 | 50.00 | 43.16 | 6.84 | 46.58 | 1.69 | | 0.56 |
| 7 | 10.00 | 18.00 | 25.08 | 100.00 | 50.00 | 42.92 | 7.08 | 46.46 | 1.75 | | 0.58 |
| 8 | 10.00 | 18.00 | 24.60 | 110.00 | 50.00 | 43.40 | 6.60 | 46.70 | 1.63 | | 0.54 |
| 9 | | | | | | | | | | | |
| 10 | | | | | | | | | | | |
| 11 | | | | | | | | | | | |
| 12 | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **0.54** |

San Bernardino County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2013)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg}))$

**Infiltration Rate versus Time, PT-02**

| | |
|---|---|
| Project Name | Santa Ana River Bottom (SARB) Maintenance Facility |
| Project Number | 23-81-234-01 |
| Test Number | BH-04/PT-02 |
| Test Location | Southern Portion of Site |
| Personnel | Javier Calzada |
| Presoak Date | 9/28/2023 |
| Test Date | 9/29/2023 |

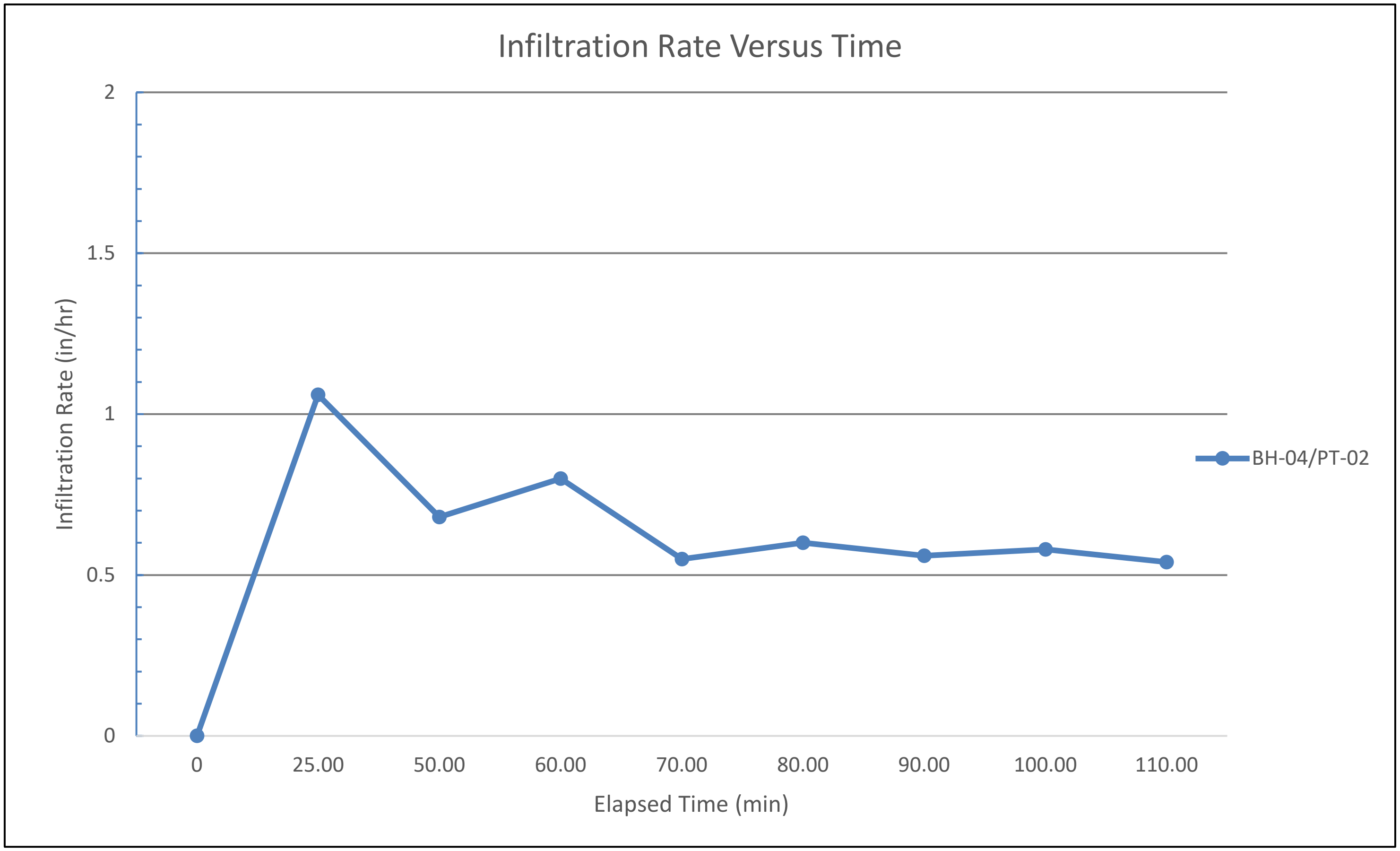



**Plate No.**
4