# Appendix B Geotechnical Investigation and Water Percolation Test Report

# Appendix

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# GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT

## HERITAGE PARK POOL FACILITY AND PARKING LOT
14301 YALE AVENUE
CITY OF IRVINE, ORANGE COUNTY, CALIFORNIA

*CONVERSE PROJECT NO. 22-32-125-01*

*Prepared For:*
MIG
109 West Union Avenue
Fullerton, California 92832

*Presented By:*
CONVERSE CONSULTANTS
3176 Pullman Street, Suite 108
Costa Mesa, CA 92626



December 12, 2022

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

December 12, 2022

Mr. Steve Lang
Principal
MIG
109 West Union Avenue
Fullerton, California 92832

Subject: **GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT**
**Heritage Park Pool Facility and Parking Lot**
14301 Yale Avenue
City of Irvine, Orange County, California
Converse Project No. 22-32-125-01

Dear Mr. Lang:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of the Heritage Park Master Plan Update project, located at 14301 Yale Avenue, City of Irvine, Orange County, California. The report was prepared in accordance with our proposal dated July 19, 2022, and your Acceptance of Agreement and Authorization to Proceed through email dated October 18, 2022.

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

We appreciate the opportunity to be of service to MIG and City of Irvine. Should you have any questions, please do not hesitate to contact us at 909-474-2847.

**CONVERSE CONSULTANTS**

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

Dist.: 1-Electronic Pdf /Addressee
SR/SM/HSQ/kvg

## PROFESSIONAL CERTIFICATION

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

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.

SK Syfur Rahman, PhD, EIT
Senior Staff Engineer

Stephen McPherson
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 test performed for the Heritage Park Pool Facility and Parking Lot project, located at 14301 Yale Avenue, City of Irvine, Orange County, California. The approximate location of the site is shown on 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 recommendations for site earthwork, and design and construction of the proposed improvements.

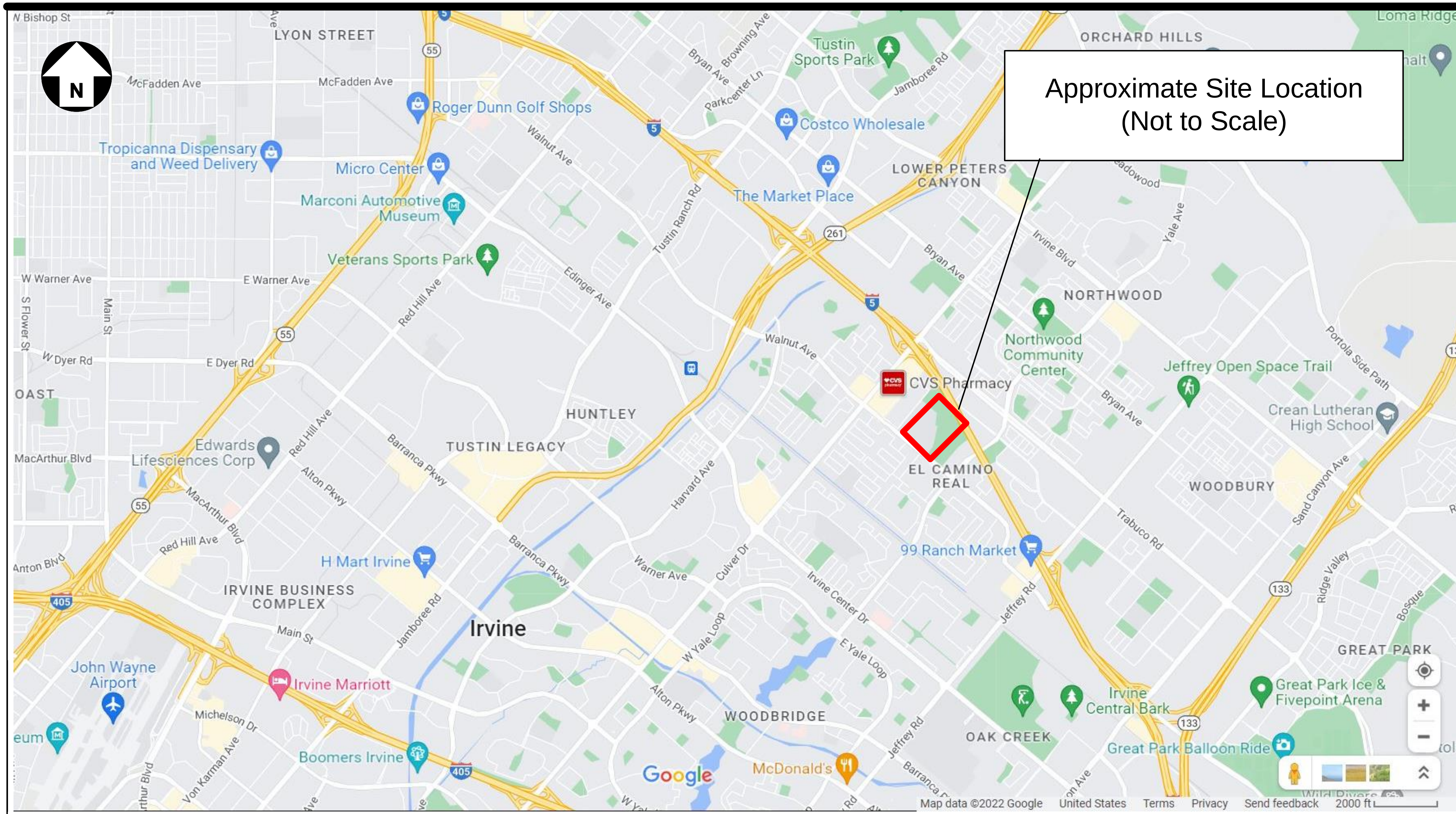
This report is prepared for the project described herein and is intended for use solely by MIG, City of Irvine, and their authorized agents for design purposes. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

# 2.0 PROJECT DESCRIPTION

According to the information provided by MIG, the Heritage Park Pool Facility and Parking Lot project will include design and construction of a pool facility and at-grade parking. The pool facility will be a 65m competition pool with concrete apron, bleachers for 2000 people, and about 6,000 square feet of locker rooms/mechanical. At-grade parking would be an asphalt or concrete parking lot with pedestrian circulation paths that connect to existing sidewalks.

# 3.0 SITE DESCRIPTION

Heritage Community Park is located at 14301 Yale Avenue, City of Irvine, Orange County, California. The park site is bounded by Santa Ana Fwy on the northeast, Yale Avenue on the southeast, Walnut Avenue on the southwest, and Escolar Drive on the northwest. Existing features in the park site include Nature Play, Splash Play, Children’s Play, Pickleball Courts, Water Tower Plaza, Connecting Plaza, Drop-Off Area, Water Features, Fine Arts Center, Group Picnic Areas, Tai Chi/Flexible Workout Area, Parking, Promenade, Library, Open Meadow, Pool, Competition Pool, Locker Rooms/Toilets, Mechanical/Storage/Support Space, Lobby/Entrance etc. Present site conditions are depicted in Photograph Nos. 1 through 3.




Project: Heritage Park Pool Facility and Parking Lot

Location: 14301 Yale Avenue
City of Irvine, Orange County, California

For: MIG

# Approximate Site Location Map

Project No.
22-32-125-01

**Converse Consultants**



Figure No.
1

*Photograph No. 1: Present site conditions at BH-01 facing southwest.*

*Photograph No. 2: Present site conditions at BH-02 facing southeast.*

*Photograph No. 3: Present site conditions at BH-03 facing northwest.*

# 4.0 SCOPE OF WORK

The scope of this investigation includes the following tasks presented below.

## *4.1 Project Set-up*

As part of the project set-up, our staff performed the following tasks.

- Prepared the boring and percolation test locations map and submitted it for your review and approval.
- Conducted a site reconnaissance and staked/marked the boring and percolation test locations such that drill rig access to all the locations was available.
- Provided necessary documentation to City of Irvine for encroachment permit application.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the borings and percolation test locations of any conflict with existing underground utilities.
- Engaged a California-licensed driller to drill exploratory borings.

## *4.2 Subsurface Exploration*

Three exploratory borings (BH-01 through BH-03) were drilled on November 7, 2022, to investigate the subsurface conditions. All borings were drilled to depths between 11.5 and 21.5 feet below ground surface (bgs).

Due to the close proximity of the borings BH-02 and BH-03 to the percolation test locations PT-01 and PT-02, after collection of soil samples, BH-02 and BH-03 were set up for percolation testing and also refereed as PT-01 and PT-02 respectively. The depth of PT-01 and PT-02 were 10.0 feet bgs. Details about the percolation tests are presented in Appendix C, Percolation Testing.

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling.

The approximate locations of the borings are shown on Figure No. 2, *Approximate Boring and Percolation Test Locations Map.* A detailed discussion of the subsurface exploration is presented in Appendix A, *Field Exploration*.

### *4.3 Laboratory Testing*

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

- *In-situ* moisture contents and dry density (ASTM D2216 and ASTM D2937)
- Soil expansion Index (ASTM D4829)
- R-value (California Test 301)
- Collapse potential (ASTM D4546)
- Soil corrosivity (California Tests 422, 417, and 643)
- Grain size distribution (ASTM D6913)
- Maximum dry density and optimum moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

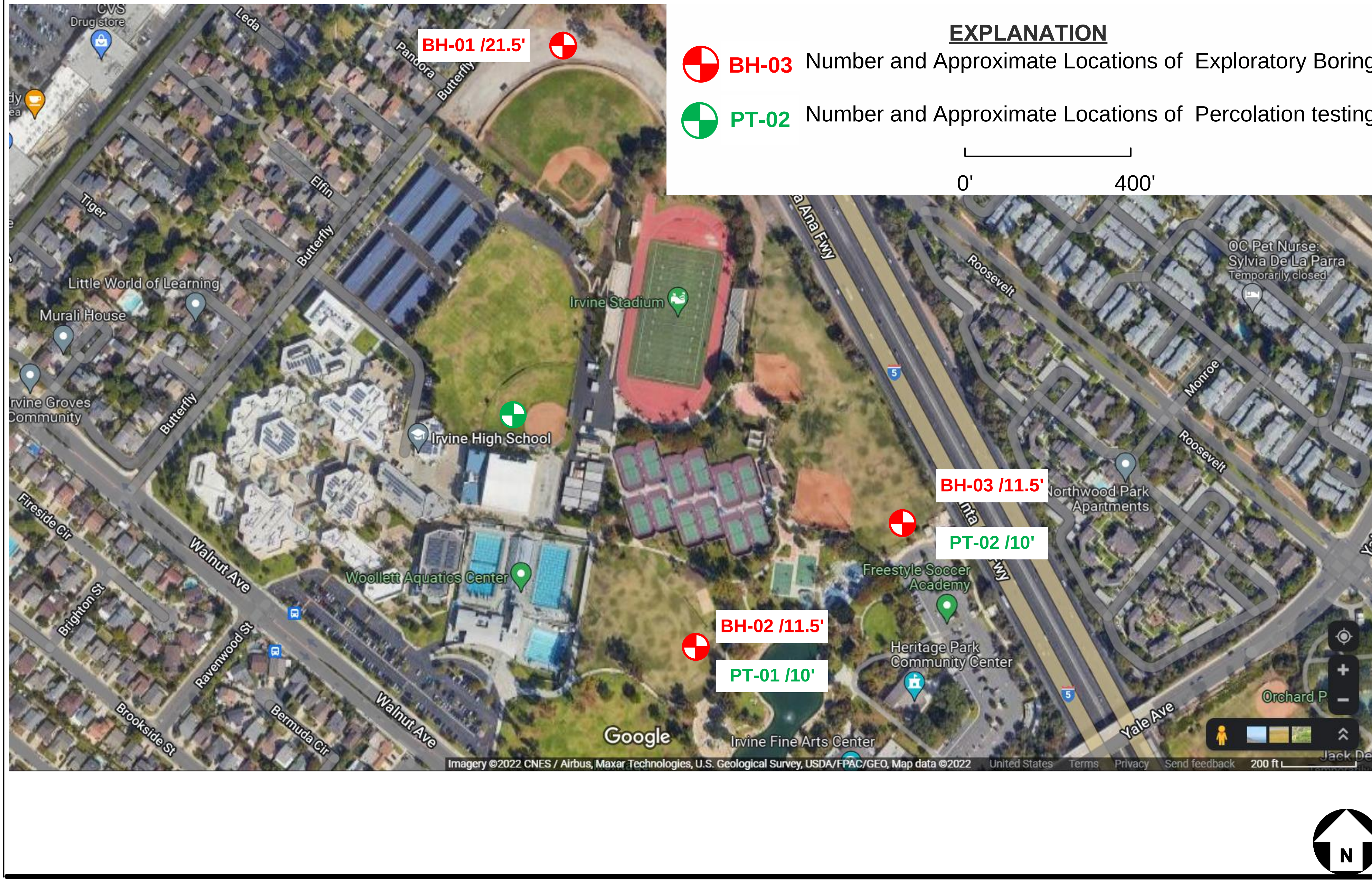
For *in-situ* moisture and dry density data, see the Logs of Borings in Appendix A, *Field Exploration*. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.

### *4.4 Report Preparation*

Data and information obtained from the document review, field exploration, and laboratory testing program were compiled and evaluated. Geotechnical analyses of the compiled data were performed, and this report was prepared to present our findings, conclusions, and recommendations for the proposed improvements.

## 5.0 SITE CONDITIONS

The subsurface conditions encountered at the site during our field investigation are described in the following sections.




Project: Heritage Park Pool Facility and Parking Lot

Location: 14301 Yale Avenue
City of Irvine, Orange County, California

For: MIG

# Approximate Boring and Percolation Test Locations Map

Project No.
22-32-125-01

**Converse Consultants**

Figure No.
2

### *5.1 Subsurface Profile*

Based on the exploratory borings and laboratory test results, the subsurface soils at the site consisted primarily of a mixture of sand, silt, clay, and gravel. Scattered to little gravel up to 2-inches in largest dimension was observed in the borings.

Discernible fill soils were not identified in our subsurface exploration; however, the site may have been previously graded for the existing structures and soccer playground and fill soil is likely present. If present, the fill soils were likely derived from on-site sources and are similar to the native alluvial soils in composition and density.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-4 *Logs of Borings,* in Appendix A, *Field Exploration.*

### *5.2 Groundwater*

Groundwater was not encountered during the field investigation up to the explored depth of 21.5 feet bgs.

For comparison, regional groundwater data from the GeoTracker database (SWRCB, 2022) was reviewed to evaluate the current and historical groundwater levels from sites within approximately a 1.0-mile radius of the project site. Data from search are provided below.

- SHELL OIL (Site No. #T0605999018), located approximately 2,400 feet north of the project site reported groundwater at depths ranging from 15.22 to 21.31 feet bgs between 2000 and 2012.
- TREASURE FARMS/IRVINE COMPANY (Site No. # T0605901258), located approximately 3,400 feet northwest of the project site reported groundwater at a depth of 32.35 feet bgs in 2001.
- ARCO (Site No. # T0605900659), located approximately 2,750 feet northwest of the project site reported groundwater at a depth of 4.0 feet bgs in 1993.
- HERITAGE ECONOMY INC (Site No. # T0605902251), located approximately 2,500 feet northwest of the project site reported groundwater at depths ranging from 7.09 to 11.10 feet bgs between 2000 and 2009.

The National Water Information System (USGS, 2022) was reviewed to evaluate current and historical groundwater levels from sites within approximately a 1.0-mile radius of the project site. Data from that search is provided below.

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

| Alignment No. | Location | Groundwater Depth Range (ft. bgs) | Date Range |
|---|---|---|---|
| 334231117464601 | I-5, Santa Ana Freeway; approximately 1,900 feet north of project site | 95.00 | 2008 |
| 334240117465401 | Culver drive at I-5; approximately 1,400 feet north of project site | 34.00-44.00 | 2008-2022 |
| 334202117460401 | Remington; approximately 3,200 feet southeast of project site | 38.87-172.50 | 1969-1986 |
| 334133117461401 | Remington; approximately 5,000 feet southeast of project site | 45.00 | 1977 |

The California Department of Water Resources database (DWR, 2022) was reviewed for historical groundwater data from sites within a 1.0-mile radius of the project site. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Details of that record are listed below.

- Well No. 05S09W25D001S (Station 337112N1177826W001), located approximately 3,000 feet north of the project site, reported groundwater at a depth ranging from 16.70 to 256.70 feet bgs between 1982 and 2010.
- Well No. 05S09W36C001S (Station 337008N1177759W001), located approximately 750 feet east of the project site, reported groundwater at a depth ranging from 54.00 to 305.00 feet bgs between 2006 and 2010.
- Well No. 05S09W36B001S (Station 337006N1177696W001), located approximately 3,000 feet east of the project site, reported groundwater at a depth ranging from 38.50 to 172.50 feet bgs between 1969 and 1986.
- Well No. 05S09W35J001S (Station 336914N1177833W001), located approximately 4,100 feet southwest of the project site, reported groundwater at a depth ranging from 88.10 to 94.80 feet bgs in 1969.
- Well No. 05S09W35D002S (Station 336983N1177947W001), located approximately 5,000 feet southeast east of the project site, reported groundwater at a depth ranging from -15.25 to 195.90 feet bgs between 1984 and 2010.
- Well No. 05S09W25N001S (Station 337049N1177813W001), located approximately 1,250 feet northeast of the project site, reported groundwater at a depth ranging from 23.00 to 256.60 feet bgs between 2006 and 2010.

Based on available data, the historical high groundwater level reported at wells within approximately one mile of the site was at or above the surface at one location near the Como Channel. However, based on historical high groundwater level reported at wells not located near a water channel, groundwater depth was reported 4.00 feet bgs. Current groundwater is expected to be deeper than about 21.5 feet bgs. Therefore, groundwater is not expected to be encountered during the construction of the project. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation.

### *5.3 Excavatability*

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

The phrase “conventional heavy-duty excavation equipment” is intended to include commonly used equipment such as excavators, scrapers, 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 models should be done by an experienced earthwork contractor.

### *5.4 Subsurface Variations*

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the 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.

## 6.0 LABORATORY TEST RESULTS

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.

### *6.1 Physical Testing*

The results of laboratory tests on samples obtained from the site are presented below.

- *In-situ* Moisture and Dry Density – *In-situ* dry density and moisture content of the site soils were determined in accordance with ASTM Standard D2216 and D2937. The dry densities of upper 0 to 10 feet soils of the site ranged from 107 to 117 pcf with moisture contents ranging from 12 to 20 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Expansion Index (EI) – One representative soil sample was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result showed an EI of 27, corresponding to low expansion potential.
- R-Value – One representative bulk sample was tested in accordance with Caltrans Test Method 301. The result of the R-value test was 15.

- Collapse Potential – The collapse potential of two relatively undisturbed samples were tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The test results showed a collapse potential of 0.01 and 0.05, indicating no collapse potential.
- Grain Size Analysis – Two representative soil samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test result is graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – The moisture-density relationship of a representative soil sample was tested in according to ASTM Standard D1557 and the results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 115.5 pounds per cubic feet (pcf) with optimum moisture content of 12.2 percent.
- Direct Shear – One direct shear test was performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The results of the direct shear tests are presented in Drawing No. B-3, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

### *6.2 Chemical Testing - Corrosivity Evaluation*

One representative soil sample was 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. 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 measurement of the sample was 8.0.
- The soluble sulfate content of the sample was 302 ppm (0.0302 percent by weight).
- The chloride concentration of the sample was 126 ppm.
- The minimum electrical resistivity (wet condition) of the sample when saturated was 1,017 ohm-cm.

## 7.0 PERCOLATION TESTING

Two percolation tests (PT-01 and PT-02) were performed on November 9, 2022, to evaluate water infiltration rate. The measured percolation test data and calculations are represented in Appendix C, *Percolation Testing*. The estimated infiltration rates at each test hole are presented in the following table.

**Table No. 2, Estimated Infiltration Rates**

| Percolation Test | Depth of Boring* (feet) | Predominant Soil Types (USCS) | Average Percolation Rate (inches/hour) | Design Percolation Rate (inches/hour) |
|---|---|---|---|---|
| PT-01 | 10 | Sandy Clay (CL), Clay with Sand (CL), Clay (CL) | 0.01 | 0.01 |
| PT-02 | 10 | Sandy Clay (CL), Clay with Sand (CL) | 0.01 | 0.01 |

(* Approximate depth)

Based on the calculated infiltration rate during the final respective intervals in each test, a design infiltration rate of 0.01 (inches/hour) can be used for depth of 10 feet for specified soil types and selected percolation testing locations. 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.

## 8.0 FAULTING AND SEISMICITY

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

### *8.1 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. 3, *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. 3, Summary of Regional Faults**

| Fault Name and Section | Closest Distance (km) | Slip Sense | Length (km) | Slip Rate (mm/year) | Maximum Magnitude |
|---|---|---|---|---|---|
| San Joaquin Hills | 3.33 | thrust | 27 | 0.5 | 7.10 |
| Newport Inglewood Connected alt 2 | 17.22 | strike slip | 208 | 1.3 | 7.50 |
| Newport Inglewood Connected alt 1 | 17.46 | strike slip | 208 | 1.3 | 7.50 |

| Fault Name and Section | Closest Distance (km) | Slip Sense | Length (km) | Slip Rate (mm/year) | Maximum Magnitude |
|---|---|---|---|---|---|
| Newport-Inglewood, alt 1 | 17.48 | strike slip | 65 | 1.0 | 7.20 |
| Newport-Inglewood | 17.52 | strike slip | 66 | 1.5 | 7.00 |
| Elsinore | 19.87 | strike slip | 241 | n/a | 7.79 |
| Puente Hills (Coyote Hills) | 22.93 | thrust | 17 | 0.7 | 6.90 |
| Chino, alt 1 | 23.67 | strike slip | 24 | 1.0 | 6.70 |
| Chino, alt 2 | 23.82 | strike slip | 29 | 1.0 | 6.80 |
| Puente Hills (Santa Fe Springs) | 33.76 | thrust | 11 | 0.7 | 6.70 |
| Palos Verdes | 36.07 | strike slip | 99 | 3.0 | 7.30 |
| Palos Verdes Connected | 36.07 | strike slip | 285 | 3.0 | 7.70 |
| San Jose | 38.6 | strike slip | 20 | 0.5 | 6.70 |
| Puente Hills (LA) | 43.76 | thrust | 22 | 0.7 | 7.00 |
| Sierra Madre | 46.67 | reverse | 57 | 2.0 | 7.20 |
| Sierra Madre Connected | 46.67 | reverse | 76 | 2.0 | 7.30 |
| Cucamonga | 47.22 | thrust | 28 | 5.0 | 6.70 |
| Coronado Bank | 48.51 | strike slip | 186 | 3.0 | 7.40 |
| Elysian Park (Upper) | 50.41 | reverse | 20 | 1.3 | 6.70 |
| Raymond | 54.8 | strike slip | 22 | 1.5 | 6.80 |
| Clamshell-Sawpit | 56.62 | reverse | 16 | 0.5 | 6.70 |
| Verdugo | 58.96 | reverse | 29 | 0.5 | 6.90 |
| San Jacinto | 60.75 | strike slip | 241 | n/a | 7.88 |
| Hollywood | 62.4 | strike slip | 17 | 1.0 | 6.70 |
| Santa Monica Connected alt 2 | 65.06 | strike slip | 93 | 2.4 | 7.40 |
| S. San Andreas | 69.07 | strike slip | 548 | n/a | 8.18 |
| Santa Monica Connected alt 1 | 71.38 | strike slip | 79 | 2.6 | 7.30 |
| Santa Monica, alt 1 | 71.38 | strike slip | 14 | 1.0 | 6.60 |
| Rose Canyon | 72.19 | strike slip | 70 | 1.5 | 6.90 |
| Cleghorn | 73.59 | strike slip | 25 | 3.0 | 6.80 |
| Malibu Coast, alt 2 | 78.04 | strike slip | 38 | 0.3 | 7.00 |
| Malibu Coast, alt 1 | 78.04 | strike slip | 38 | 0.3 | 6.70 |
| Sierra Madre (San Fernando) | 79.83 | thrust | 18 | 2.0 | 6.70 |
| Anacapa-Dume, alt 2 | 79.85 | thrust | 65 | 3.0 | 7.20 |
| San Gabriel | 82.59 | strike slip | 71 | 1.0 | 7.30 |
| North Frontal (West) | 82.92 | reverse | 50 | 1.0 | 7.20 |
| Northridge | 87.29 | thrust | 33 | 1.5 | 6.90 |
| Anacapa-Dume, alt 1 | 90.42 | thrust | 51 | 3.0 | 7.20 |
| Santa Susana, alt 1 | 95.71 | reverse | 27 | 5.0 | 6.90 |

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

### *8.2 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. 4, CBC Seismic Design Parameters**

| Parameter | Value |
|---|---|
| Site Coordinates | 33.701564 N, 117.779201 W |
| Risk Category | II |
| Site Class | D |
| Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_S$ | 1.258g |
| Mapped 1-second Spectral Response Acceleration, $S_1$ | 0.450g |
| Site Coefficient (from Table 1613.5.3(1)), $F_a$ | 1.0 |
| Site Coefficient (from Table 1613.5.3(2)), $F_v$ | 1.850 |
| MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$ | 1.258g |
| MCE 1-second period Spectral Response Acceleration, $S_{M1}$ | 0.833g |
| Design Spectral Response Acceleration for short period $S_{DS}$ | 0.839g |
| Design Spectral Response Acceleration for 1-second period, $S_{D1}$ | 0.555g |
| Peak Ground Acceleration, $PGA_M$ | 0.577g |

### *8.3 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 Department of Conservation, Geologic Hazards Map (CGS, 2007; DOC, 2022). 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 feet of the ground surface suffers a substantial reduction in its shear strength, due to the development 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.

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

Based on a review of state and county hazard maps, the project site is located within an area at risk for liquefaction by the State of California (CGS, 2007). Due to the limitation of field investigation depth, site specific liquefaction analysis was not performed. However, due to the presence of clayey soil and groundwater not being encountered up to a depth of 21.5 feet bgs, the potential for liquefaction can be considered low.

***Seismic Settlement***: Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. Classification of the samples and sampling blow counts indicate that the site is loose, medium dense, to dense. The potential for dry seismic settlement is not known with certainty, however, the potential is considered low.

***Landslides:*** Seismically induced landslides and slope failures are common occurrences during or soon after large earthquakes. Due to the flat nature of the site and the distance away from any foothills, the potential for seismically induced landslides affecting the proposed site is considered to be very 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. The topography at the project site and in the immediate vicinity is very flat. Under these circumstances, the potential for lateral spreading at the subject site is considered low to moderate.

***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 site being a far distance from bodies of water, seiches are 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 located within a designated dam inundation area (DSOD, 2022). The Syphon Canyon, No. 1029-4, has a potential to inundate the project area.

# 9.0 EARTHWORK RECOMMENDATIONS

Earthwork recommendations for the project are presented below.

## *9.1 General*

This section contains our general recommendations regarding earthwork and grading for the proposed improvements. 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 findings during the final investigation or observation of the actual field conditions during grading.

All existing underground utilities and appurtenances should be located at the 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 structures or utilities.

All debris, deleterious material, demolished material, and artificial fill (if any) and surficial soils containing roots and perishable materials should be stripped and removed from the site. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

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.

## *9.2 Overexcavation/Removal*

Structural footings, slabs, and pavements should be uniformly supported by compacted fill. In order to provide uniform support, structural areas should be overexcavated, scarified, and recompacted as follows.

**Table No. 5, Overexcavation Depths**

| Structure/Pavement | Minimum Excavation Depth |
|---|---|
| Swimming Pool | 18 inches below footing bottoms |
| Bleachers | 15 inches below footing bottoms, or 2 feet below ground surface, whichever is deeper |
| Locker Rooms | 18 inches below footing bottoms, or 3 feet below ground surface, whichever is deeper |
| Slabs, Pavement | 12 inches below finish grade |

The depth of over excavation below the footings, slab, and pavements should be uniform. The over excavation should extend to at least 2 feet beyond the footprint of the footings and slabs and 1-foot beyond the edge of the pavement. The over excavation bottom should be scarified and compacted as described in Section 9.4, *Compacted Fill Placement*.

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. Consideration should be given to using slot cuts or other excavation methods which preserve lateral support during excavation operations near the existing structures.

### *9.3 Engineered/Structural Fill*

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the site are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including cleaning roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-site 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 organic matter, debris, or other deleterious material.
- Expansion index of 30 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-site soils may be suitable as structural/engineered fill materials provided corrosion recommendations presented in

Section 10.10, *Soil Corrosivity* are implemented. Also, since the in-situ moisture within upper 5 feet of soil is higher than the optimum moisture content, moisture conditioning will be required during grading.

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

### *9.4 Compacted Fill Placement*

All surfaces to receive structural fills should be scarified to a depth of 6 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 densities as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein. At least the upper 1 feet 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.

### *9.5 Compaction Behind Wall*

The backfill for the retaining wall of the swimming pool should be compacted to 90 percent of laboratory the maximum dry density. Compaction of backfill adjacent to wall can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the wall. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any wall within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the wall.

### *9.6 Shrinkage and Subsidence*

The volume of excavated and recompacted soils may be expected to 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. For preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- An average shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) of 5 percent can be used for the upper 5 feet of soils for preliminary earthwork planning.
- Subsidence (defined as the settlement of native materials from the equipment load applied during grading) would depend on the construction methods including type of equipment utilized. For estimation purposes, ground subsidence may be taken as 0.1 to 0.15 feet.

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.

### *9.7 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. The building pad should have a gradient of at least 2 percent towards drainage facilities. The drainage gradient should be 1 percent for paved areas and 2 percent in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

## 10.0 DESIGN RECOMMENDATIONS

Design recommendations are presented in the following sections.

### *10.1 General Evaluation*

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the above earthwork recommendations will be implemented.

### *10.2 Shallow Foundation Design Parameters*

The proposed swimming pool may be supported on continuous and/or isolated stiffened 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 spread footing width | 15 inches |
| Minimum isolated footing width | 15 inches |
| Minimum continuous or isolated footing depth of embedment below lowest adjacent grade | 15 inches |
| Allowable net bearing capacity | 2,500 psf |

The allowable bearing capacity can be increased by 500 psf with each foot of additional embedment and 100 psf with each foot of additional width up to a maximum of 3,500 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.

### *10.3 Mat Foundation Design Parameters*

The swimming pool may be supported by mat foundation. The modulus of subgrade reaction (k) for design of flexible mat foundation can be estimated from the available soil compressibility data and published charts. For design of flexible mat foundation, the following equation may be used.

$$k= k_1[(B+1)/2B]^2$$

Where:
k= vertical modulus of subgrade reaction for mat foundation, kips per cubic feet
$k_1$= 200 kcf, normalized modulus of subgrade reaction for 1-square-foot footing
B= foundation width, feet

Other necessary parameters (modulus of elasticity and Poisson's ratio) for mat foundation design are as follows.

$$E= 33\, W_c^{1.5}\, f_c^{\,0.5} \text{ psi}$$

Where, E = Modulus of Elasticity of Concrete (psi)
$W_c$ = weight of concrete (pcf)
$f_c$ = compressive strength of concrete at 28 days (psi)
$\nu$ = 0.35, Poisson's Ratio

An allowable net bearing capacity of 2,500 psf may be used for mat foundations founded on compacted native soil. The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. The mat foundation dimensions, and reinforcement should be based on structural design. For design purposes, the self-weight of the mat foundation can be negligible.

### *10.4 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.

#### 10.4.1 Active Earth Pressures

The active earth pressure behind any buried wall or foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall or foundation inclination, surcharges, and any hydrostatic pressures. The recommended lateral earth pressures without surcharge for the site are presented in the following table.

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

| Loading Conditions | Lateral Earth Pressure (psf/ft depth) |
|---|---|
| Active earth conditions (wall is free to deflect at least 0.001 radian) | 45 |
| At-rest (wall is restrained) | 66 |
| Horizontal Seismic Coefficient | 0.22H* |

*H = height of buried wall

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 structure, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the structure.

#### 10.4.2 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 0.35 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 210 psf per foot of depth may be used for the sides of footings poured against recompacted 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 2,500 psf for compacted fill.

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.

### *10.5 Drilled Pier Foundations*

The Bleachers can also be supported on drilled pier foundations deriving their support primarily through skin friction. The piers may be designed for compression using an allowable skin friction value of 250 psf for a minimum of 12 feet deep below the finished grade. This value may be increased by 33 percent for transient wind and seismic forces. For pier design in tension, 50 percent of the recommended allowable skin friction values in compression may be used. The drilled pier should have a minimum diameter of 24 inches. For design purposes, the upper 2 feet of the soil should be neglected in determining the skin friction and point of fixity can be considered in the toe of pier.

The equivalent lateral earth pressure equal to 210 pounds per square foot per foot of depth may be used for the design.

### *10.6 Drilled Pier Foundation Installation Recommendations*

It is the responsibility of the contractor to select proper construction equipment and method to correctly install the piers based on his own interpretation of the information presented in this report.

Groundwater was not encountered in the exploratory boreholes up to depth of 21.5 feet below existing ground surface and due to the presence of clayey soil, there is less possibility of caving. However, casing, or other methods approved by the project geotechnical consultant, may be used to support the sides of the excavation. Casing should be used at the discretion of the contractor. The casing should be advanced as drilling proceeds by drilling with a flight or bucket auger smaller in diameter than the inside of the casing. Occasional hammering may be required to advance the casing within the excavation. The casing, when used, should not be left in place as the pier designs are based on skin friction only. The casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. The contractor should have equipment on-site with sufficient pulling capacity to pull the casing at the proper time. The casing should have an outside diameter not less than the specified diameter of the pier.

The bottoms of the excavations should be cleaned of any loose cuttings before placing concrete. All applicable state and federal OSHA safety regulations must be satisfied during construction.

Drilled pier installation shall be performed under continuous observation by the project geotechnical consultant to confirm that the subsurface soils are similar to the soils encountered during our field investigation, which have formed the basis of our pier design recommendations. The contractor shall provide access and necessary facilities, including droplights, at his expense, to accommodate pier observations.

Drilled pier installation shall be performed such that compliance with all safety rules and requirements is achieved.

### *10.7 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 9.4 *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. Slab should be monolithically constructed with the footings and grade beams.

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.

If moisture-sensitive flooring or environments are planned, slabs-on-grade should be protected by 10-mil-thick polyethylene vapor barriers. The sub-grade surface should be free of all exposed rocks or other sharp objects prior to placement of the barrier. The barrier should be overlain by 2 inches pf sand, to minimize punctures and to aid in the concrete curing. At discretion of the structure engineer, the sand layer may be eliminated. Converse does not practice in the field of moisture vapor transmission evaluation/mitigation since this does not fall under the geotechnical disciplines. Therefore, we recommend that a qualified person, such as the flooring contractor, structural engineer, and/or architect be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction.

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.

### *10.8 Retaining Wall Drainage*

The recommended lateral earth pressure values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose excess water that may accumulate behind earth retaining structures. Wall drainage may be provided by free-draining gravel surrounded by synthetic filter fabric or by prefabricated, synthetic drain panels or weep holes. In either case, drainage should be collected by perforated pipes and directed to a sump, storm drain, or other suitable location for disposal. We recommend drain rock should consist of durable stone having 100 percent passing the 1-inch sieve and less than 5 percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a minimum flow rate of 110 gallons per minute per square foot of fabric, and a minimum puncture strength of 110 pounds.

### *10.9 Settlement*

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 40 feet.

### *10.10 Soil Corrosivity*

The results of chemical testing of one representative soil sample 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 general discussion pertaining to soil corrosivity is presented below.

The sulfate contents of the sampled soil correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-14, Table 19.3.1.1). 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 guideline of soil corrosion based on electrical resistivity.

**Table No. 8, 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 sample when saturated was 1,017 Ohm-cm. This indicates that the soil tested of the site is corrosive to ferrous metals in contact with the soils (Romanoff, 1957). <u>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 and site soils.</u>

### *10.11 Flexible Pavement Recommendations*

Based on the laboratory test result, the R-value of the subgrade soil was 15. For pavement design, we have utilized an R-value of 15 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, 2021), 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. 9, Recommended Preliminary Pavement Sections**

| R-value | Traffic Index (TI) | Pavement Section | | |
|---|---|---|---|---|
| | | Option 1 | | Option 2 |
| | | Asphalt Concrete (inches) | Aggregate Base (inches) | Full AC Section (inches) |
| 15 | 5 | 4.0 | 6.0 | 4.0 |
| | 6 | 4.5 | 8.5 | 5.0 |
| | 7 | 5.0 | 11.0 | 6.5 |
| | 8 | 5.5 | 14.0 | 7.5 |

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, appropriate earthworks should be performed according to specifications provided in Section 9.0 *Earthwork Recommendations.*

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.

### *10.12 Rigid Pavement Recommendations*

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 15 and design Traffic Indices (TIs) ranging from 5 to 9 for both Bradford and Fairhaven Well sites. 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. 9, Rigid Pavement Structural Sections for Fairhaven and Bradford Well Sites**

| Design R-Value | Design Traffic Index (TI) | PCCP Pavement Section (inches) |
|---|---|---|
| 15 | 5.0 | 7.5 |
| | 6.0 | 7.5 |
| | 7.0 | 8.0 |
| | 8.0 | 8.5 |

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 12.0 inches of subgrade soils below rigid pavement sections should be compacted to at least ninety-five percent (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.

### *10.13 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 2 feet 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 cement concrete 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.

# 11.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

## *11.1 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.

Sloped excavations may not be feasible in locations adjacent to existing utilities or pavement. Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing utilities or pavement 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 soils 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.

## *11.2 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. 10, Slope Ratios for Temporary Excavations**

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

[1] Slope ratio is assumed to be constant from top to toe of slope, with level adjacent ground.

Shallow excavations up to 4 feet bgs 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.

## 12.0 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. Testing should be performed to determine density and moisture of the compacted soils. 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.

## 13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by MIG, City of Irvine, 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. Site 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 is 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 the recommendations contained in this report are 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.

## 14.0 REFERENCES

AMERICAN CONCRETE INSTITUTE (ACI), 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, October 2014.

Appendix VII. Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (City of Santa Ana, 2013)

CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2022, California Building Code (CBC).

CALIFORNIA STATE WATER RESOURCES CONTROL BOARD (SWRCB), 2022, GeoTracker database (http://geotracker.waterboards.ca.gov/), accessed in 2022.

DAS, B.M., 2011, Principles of Foundation Engineering, Seventh Edition, published by Global Engineering, 2011.

The California Department of Water Resources database (DWR, 2022).

ROMANOFF, MELVIN, 1957, Underground Corrosion, National Bureau of Standards Circular 579, dated April 1957.

U.S. GEOLOGICAL SURVEY (USGS), 2022, U.S. Seismic Design Maps Application (http://geohazards.usgs.gov/designmaps/us/application.php), accessed on October 1, 2022.

U.S. GEOLOGICAL SURVEY (USGS), 2022, National Water Information System: Web Interface (https://maps.waterdata.usgs.gov/mapper/index.html), accessed on October 1, 2022.

U.S. GEOLOGICAL SURVEY (USGS), 2008, U.S. Seismic Design Maps Application (http://geohazards.usgs.gov/designmaps/us/application.php), accessed on October 1, 2022.

# Appendix A

## Field Exploration

# APPENDIX A

## FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling soil borings and water percolation test. During the site reconnaissance, the surface conditions were noted, and the borings were marked at locations approved by Ms. Kathleen Haton with City of Irvine. The approximate boring locations were established in the field using approximate distances from local streets as well as existing structures as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Three exploratory borings (BH-01 through BH-03) were drilled on November 7, 2022, to investigate the subsurface conditions. All borings were drilled to depths between 11.5 and 21.5 feet below ground surface (bgs).

Due to the close proximity of the borings BH-02 and BH-03 to the percolation test locations PT-01 and PT-02, after collection of soil samples, BH-02 and BH-03 were set up for percolation testing and also refereed as PT-01 and PT-02, respectively. The depth of PT-01 and PT-02 were 10.0 feet bgs. Details about the percolation tests are presented in Appendix C, *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 Engineer 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 in plastic bags.

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, the boring BH-01 was backfilled with soil cuttings mixed with cement and compacted by pushing down with an auger using the drill rig weight. After completion of percolation testing, the pipes were removed from PT-01 (BH-02) and PT-02 (BH-03), and boreholes were backfilled with soil cuttings and tamped. If construction is delayed, the surface at 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-4, *Logs of Borings*. All elevations are based on Google Earth.

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS | GRAVEL AND GRAVELLY SOILS | 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 |
| | MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | SAND AND SANDY SOILS | 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 |
| | MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS | 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 |
| MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | 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

### DRILLING METHOD SYMBOLS

Auger Drilling

Mud Rotary Drilling

Dynamic Cone or Hand Driven

Diamond Core

### 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

## UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

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Heritage Park Pool Facility and Parking Lot
14301 Yale Avenue
City of Irvine, Orange County, California For:
MIG

**Project No.** 22-32-125-01 **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: 11/7/2022 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 112 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 |
|---|---|---|---|---|---|---|---|---|
| 0 | | **ALLUVIUM:**<br>**CLAYEY SAND/SANDY CLAY WITH GRAVEL (SC/CL):** fine to coarse-grained, little gravel up to 2 inches in maximum dimension, sub-angular to sub-rounded, medium dense, moist, dark brown.<br>- roots and rootlets, dark brown to black | | X | | | | PA |
| 2.5 | | | X | X | 10/13/12 | 10 | 117 | |
| 5 | | **SANDY CLAY (CL):** fine to coarse-grained sand, few gravel up to 1" in maximum dimension, medium stiff to stiff, moist, brown to dark brown. | X | X | 3/5/6 | 15 | 114 | PA |
| 7.5 | | | X | X | 7/8/15 | 18 | 112 | |
| 10 | | **CLAY WITH SAND (CL):** fine to medium-grained sand, tan oxidation streaking, medium plasticity, very stiff, moist, dark brown. | X | | 6/12/18 | 15 | 114 | |
| 15 | | **SAND WITH SILT (SP-SM):** fine to medium-grained, scattered gravel up to 0.3" in maximum dimension - sub-rounded, medium dense, moist, orangish brown. | X | | 5/6/7 | 5 | 106 | |
| 20 | | **SILTY CLAY (CL-ML):** trace fine-grained sand, medium plasticity, stiff, moist, brown to dark brown.<br><br>End of Boring at 21.5 feet bgs.<br>No groundwater was encountered.<br>Borehole was backfilled with soil cuttings and compacted by pushing down with an augers using the drill rig weight on 11/7/2022. | X | | 3/8/12 | 27 | 96 | |

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City of Irvine, Orange County, California
For: MIG

Project No. **22-32-125-01**

Drawing No. **A-2**

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

Date Drilled: 11/7/2022 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 119 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 |
|---|---|---|---|---|---|---|---|---|
| 0 | | **ALLUVIUM:**<br>**SANDY CLAY (CL):** fine to medium-grained sand, scattered gravel up to 0.5 inches in maximum dimension, rootlets, medium plasticity, moist, dark brown. | | X | | | | CP, CR, EI |
| 5 | | **CLAY WITH SAND (CL):** fine to medium-grained sand, trace silt, tan oxidation streaking, stiff, moist, dark brown. | X | | 4/10/15 | 20 | 107 | |
| | | | X | | 4/8/10 | 16 | 107 | CL, DS |
| 10 | | **CLAY (CL):** trace fine to medium-grained sand, trace silt, tan oxidation streaking, very stiff, moist, dark brown to black. | X | | 7/16/22 | 22 | 105 | |

End of Boring at 11.5 feet bgs.
No groundwater was encountered.
Borehole was used for percolation testing. After completion of the percolation testing, pipe was removed, and borehole was backfilled with soil cuttings and tamped on 11/9/2022.

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For: MIG

Project No. 22-32-125-01

Drawing No. A-3

# Log of Boring No. BH-03 / PT-02

Date Drilled: 11/7/2022 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 126 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 |
|---|---|---|---|---|---|---|---|---|
| 0–5 | | **ALLUVIUM:**<br>**CLAYEY SAND/SANDY CLAY (SC/CL):** fine to coarse-grained, roots and rootlets, moist, dark brown. | | X | | | | R |
| 5 | | **SANDY CLAY (CL):** fine to medium-grained sand, scattered gravel up to 2" in maximum dimension - sub-angular to sub-rounded, trace silt, tan oxidation streaking, very stiff, moist, dark brown. | X | | 2/7/13 | 12 | 109 | |
| | | | X | | 8/15/17 | 17 | 114 | |
| 10 | | **CLAY WITH SAND (CL):** fine to medium-grained sand, trace silt, tan oxidation streaking, very stiff, moist, dark brown. | X | | 9/12/21 | 19 | 108 | |

End of Boring at 11.5 feet bgs.
No groundwater was encountered.
Borehole was used for percolation testing. After completion of the percolation testing, pipe was removed, and borehole was backfilled with soil cuttings and tamped on 11/9/2022.

# 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 ASTM 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

One 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-02 | 0-5 | Sandy Clay (CL) | 27 | Low |

### R-value

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-03 | 0-5 | Sandy Clay (CL)/Clayey Sand (SC) | 15 |

### Collapse

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, two collapse tests were performed in accordance with the ASTM Standard D4546 laboratory procedure. The samples were loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test results are presented in the following table.

**Table No. B-3, Collapse Test Results**

| Boring No. | Depth (feet) | Soil Classification | Percent Swell (+) Percent Collapse (-) | Collapse Potential |
|---|---|---|---|---|
| BH-02 | 7.5-9.0 | Clay with Sand (CL) | +0.01 | None |
| BH-03 | 5.0-6.5 | Sandy Clay (CL) | -0.05 | None |

## Soil Corrosivity Test

One representative soil sample was tested to determine minimum electrical resistivity (wet condition), 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 Tests 643, 422 and 417. Test results are presented in the following table.

**Table No. B-4, 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-02 | 0-5 | 8.0 | 302 | 126 | 1,017 |

## Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analyses were performed on two select samples in accordance with the ASTM Standard ASTM D6913 test method. Grain-size curve is shown in Drawing No. B-1, *Grain Size Distribution Result*.

**Table No. B-5, 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>Clayey Sand/Sandy Clay with Gravel (SC/CL)</td><td>23.0</td><td>39.7</td><td colspan="2">37.3</td></tr>
<tr><td>BH-01</td><td>5-10</td><td>Sandy Clay (CL)</td><td>6.0</td><td>44.1</td><td colspan="2">49.9</td></tr>
</table>

## Maximum Density and Optimum Moisture Content

Laboratory maximum dry density-optimum moisture content relationship test was performed on one representative bulk soil sample. The test was 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-6, Summary of Moisture-Density Relationship Results**

| Boring No. | Depth (feet) | Soil Description | Optimum Moisture (%) | Maximum Density (lb/cft) |
|---|---|---|---|---|
| BH-02 | 0-5 | Sandy Clay (CL), Dark Brown | 12.2 | 115.5 |

**Direct Shear**

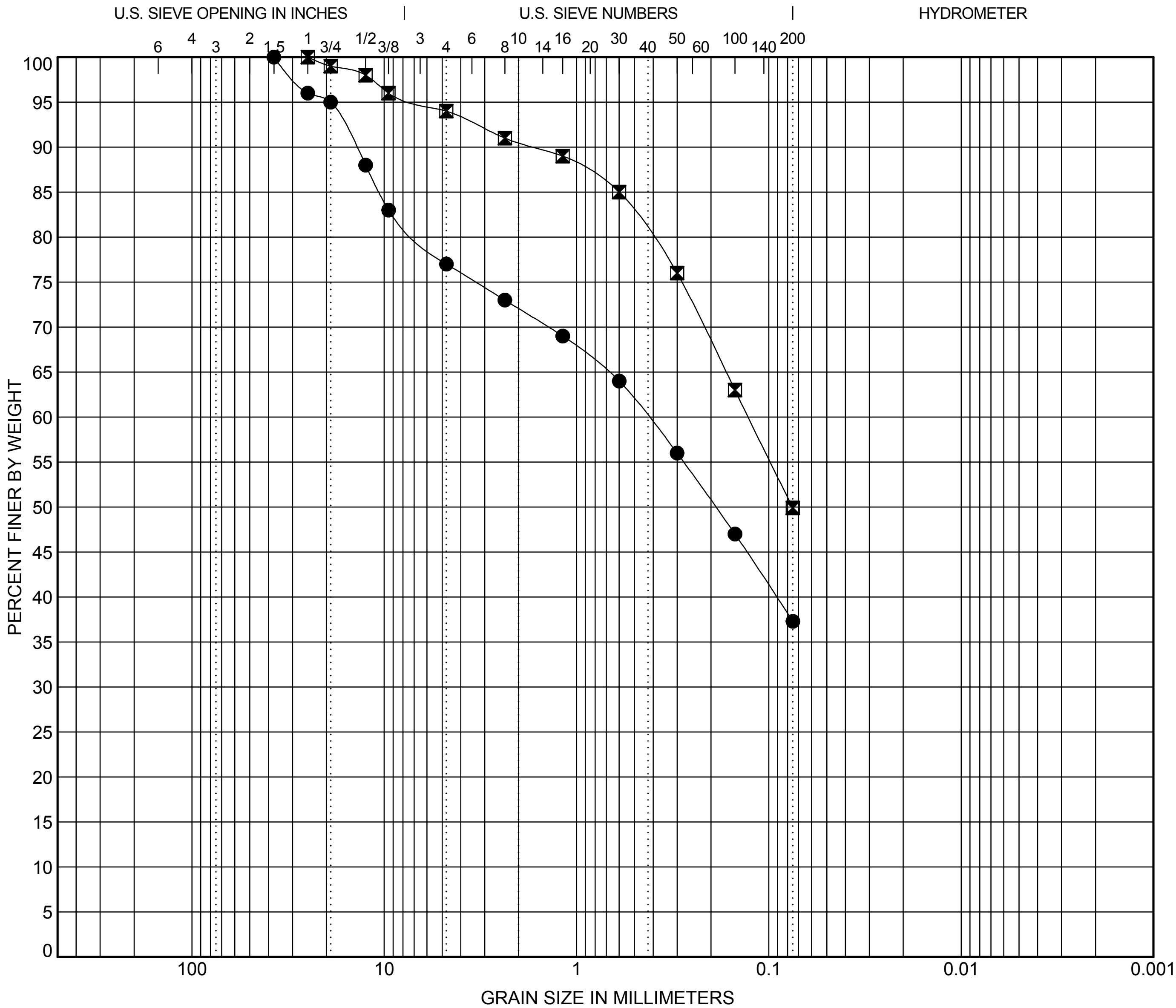
One direct shear test was performed on relatively undisturbed samples under soaked moisture condition in accordance with ASTM D3080. For this test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.004 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 Drawing No. B-3*, Direct Shear Test Result,* and the following table.

**Table No. B-7, Summary of Direct Shear Test Results**

| Boring No. | Depth (feet) | Soil Description | Peak Strength Parameters | |
|---|---|---|---|---|
| | | | Friction Angle (degrees) | Cohesion (psf) |
| BH-02 | 7.5-9.0 | Clay with Sand (CL) | 27 | 220 |

**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** | **0-5.0** | **CLAYEY SAND/SANDY CLAY WITH GRAVEL(SC/CL)** | | | | | |
| ⊠ | **BH-01** | **5.0-10.0** | **SANDY CLAY (CL)** | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| | Boring No. | Depth (ft) | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **BH-01** | **0-5.0** | **37.5** | **0.424** | | | **23.0** | **39.7** | **37.3** | |
| ⊠ | **BH-01** | **5.0-10.0** | **25** | **0.128** | | | **6.0** | **44.1** | **49.9** | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| | | | | | | | | | | |

# GRAIN SIZE DISTRIBUTION RESULTS

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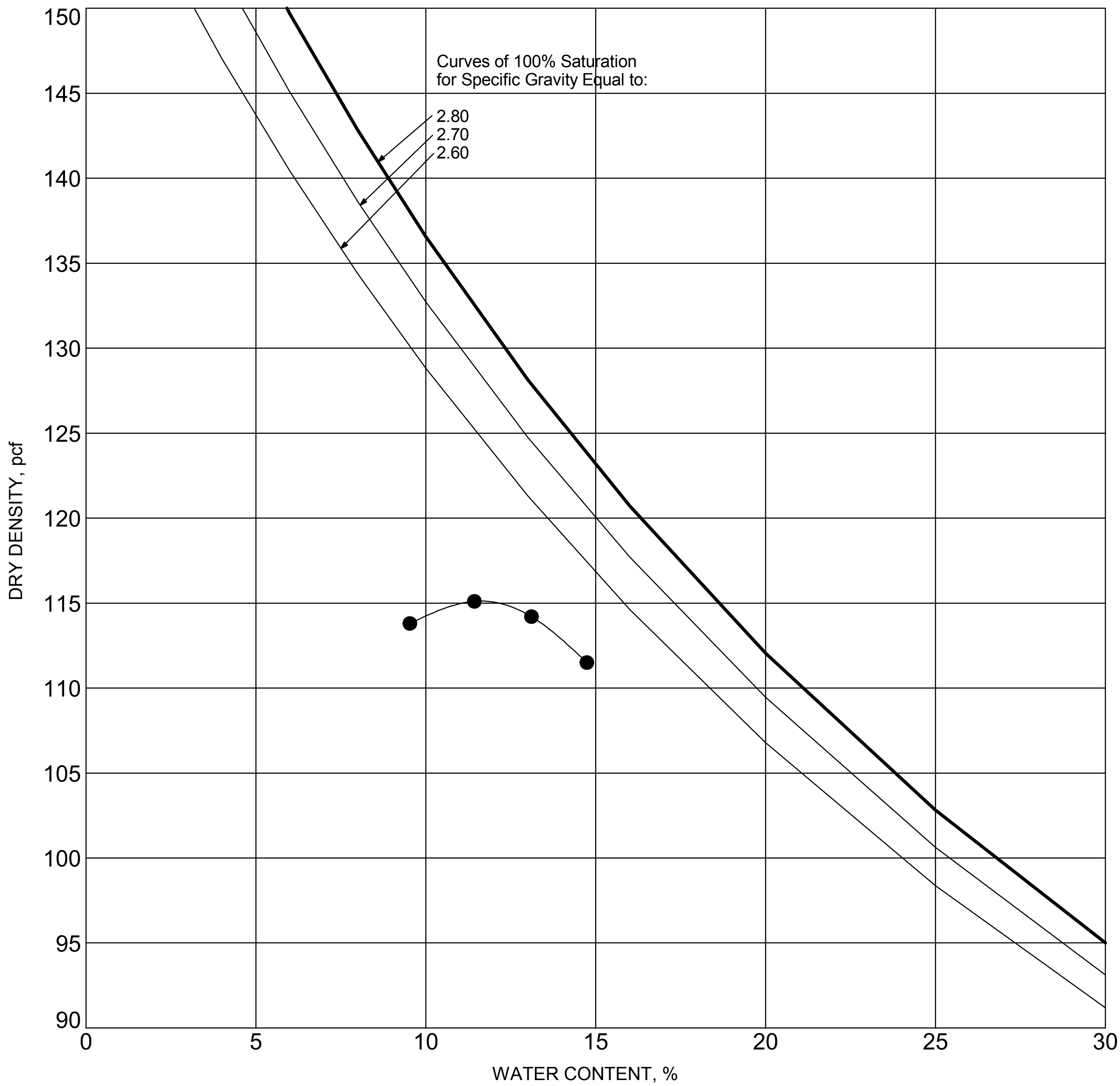
For: MIG

Project No. **22-32-125-01**

Drawing No. **B-1**

Project ID: 22-32-125-01.GPJ; Template: GRAIN SIZE




| SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf |
|---|---|---|---|---|---|---|
| ● | **BH-02 / PT-01** | **0-5.0** | **SANDY CLAY (CL), DARK BROWN** | **D1557 Method B** | **12.2** | **115.5** |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

# MOISTURE-DENSITY RELATIONSHIP RESULTS

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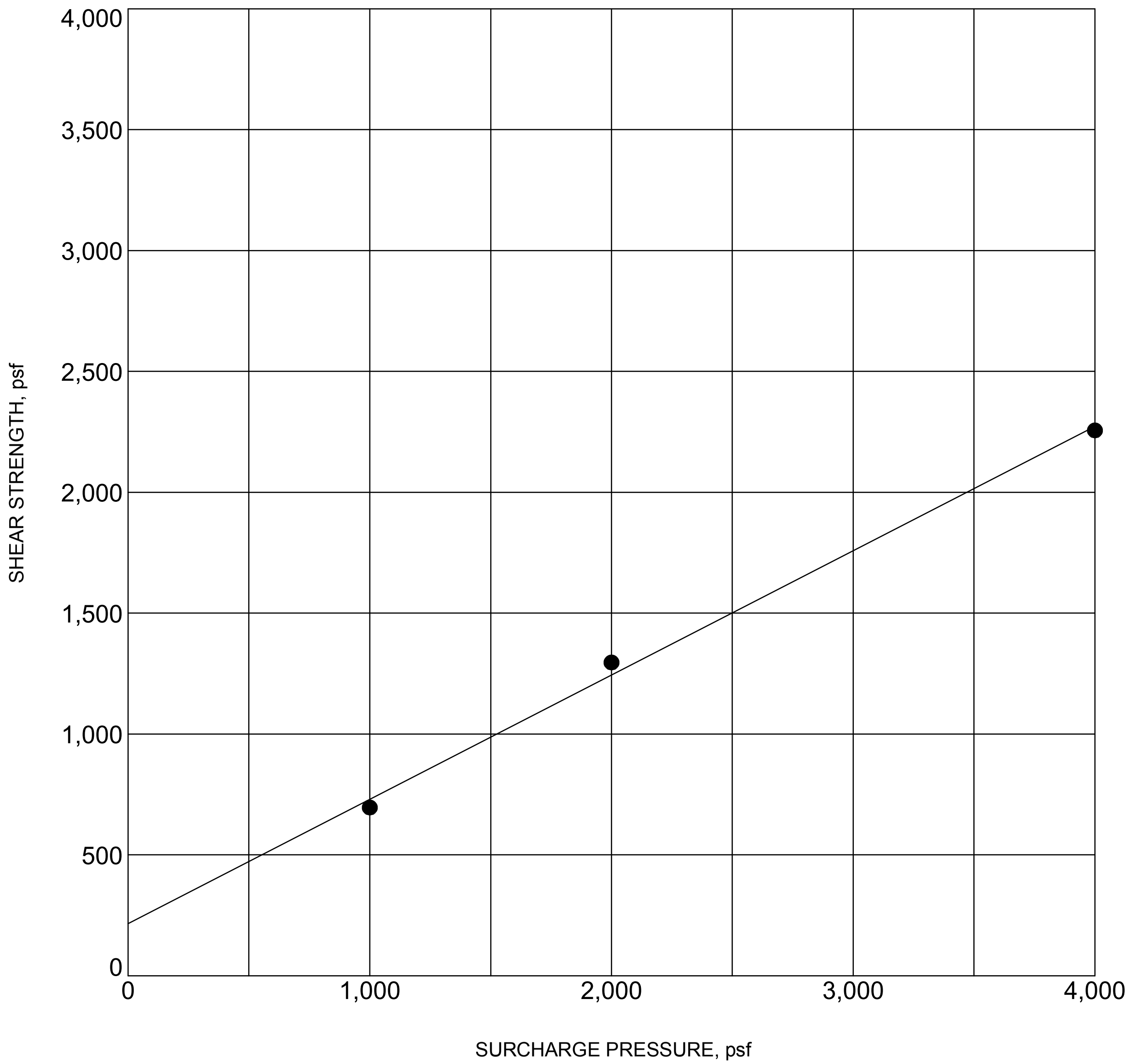
For: MIG

Project No.
**22-32-125-01**

Drawing No.
**B-2**

Project ID: 22-32-125-01.GPJ; Template: COMPACTION




| | | | | | |
|---|---|---|---|---|---|
| BORING NO. | : | **BH-02 / PT-01** | DEPTH (ft) | : | **7.5-9.0** |
| DESCRIPTION | : | **CLAY WITH SAND (CL)** | | | |
| COHESION (psf) | : | **220** | FRICTION ANGLE (degrees): | | **27** |
| MOISTURE CONTENT (%) | : | **16.0** | DRY DENSITY (pcf) | : | **107.0** |

NOTE: Ultimate Strength.

# DIRECT SHEAR TEST RESULTS

Converse Consultants

Heritage Park Pool Facility and Parking Lot
14301 Yale Avenue
City of Irvine, Orange County, California
For: MIG

Project No. **22-32-125-01**

Drawing No. **B-3**

# Appendix C

## Percolation Testing

# APPENDIX C

## PERCOLATION TESTING

Percolation testing was performed at two locations (PT-01 and PT-02) on November 9, 2022, in general accordance with the Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMP), Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (Orange County, 2013) for using a percolation testing method to estimate infiltration rates.

Upon completion of drilling the test holes, an approximately 2-inch-thick gravel layer was placed at the bottom of each hole 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 for erosion and caving due to the addition of water to the hole.

Each test hole was presoaked by filling with water to at least 5 times the radius of the test hole. After pre-soaking, water was added until the water levels were as near the required testing depth as could be achieved. The water levels were measured to the nearest 1/10-foot and recorded every 30 minutes. Following the completion of percolation testing, the pipe was removed from PT-01 and PT-02, and borehole backfilled with soil cuttings and tamped.

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 design. The percolation test data was used to estimate infiltration rates using the Porchet Inverse Borehole Method, in accordance with the Orange County technical guidelines. A factor of safety of 2 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, calculations and estimated infiltration rates are shown on Plates No. 1 and 2. The estimated infiltration rates at the test holes are presented in the following table.

**Table C-1, Estimated Infiltration Rates**

| Percolation Test | Depth (feet) | Soil Type | Infiltration Rate (inches/hour) (FOS 2) |
|---|---|---|---|
| PT-01 | 10 | Sandy Clay (CL), Clay with Sand (CL), Clay (CL) | 0.01 |
| PT-02 | 10 | Sandy Clay (CL), Clay with Sand (CL) | 0.01 |

Based on the calculated infiltration rates during the final respective intervals in each test, a design infiltration rate of 0.01(inches/hour) can be used for depth of 10 feet for specified soil types for selected percolation testing locations. 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 is required to be performed in the required location.

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

| | |
|---|---|
| Project Name | Heritage Park Master Plan Update |
| Project Number | 22-32-125-01 |
| Test Number | PT-01 |
| Test Location | 33.701087, -117.778963 |
| Personnel | Aleksey Zhukov |
| Presoak Date | 11/7/2022 |
| Test Date | 11/9/2022 |

Shaded cells contain calculated values.

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 120 |
| Inside Diameter of Pipe, I (inches) | 3.00 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| | |
| Factor of Safety (FOS), F | 2 |

| 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 | 19.36 | 19.60 | 25.00 | 100.64 | 100.40 | 0.24 | 100.52 | 0.01 | | 0.01 |
| 2 | 30.00 | 8.00 | 8.50 | 55.00 | 112.00 | 111.50 | 0.50 | 111.75 | 0.02 | | 0.01 |
| 3 | 30.00 | 8.00 | 8.63 | 85.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 4 | 30.00 | 8.00 | 8.63 | 115.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 5 | 30.00 | 8.00 | 8.63 | 145.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 6 | 30.00 | 8.00 | 8.63 | 175.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 7 | 30.00 | 8.00 | 8.63 | 205.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 8 | 32.00 | 8.00 | 8.63 | 237.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 9 | 30.00 | 8.00 | 8.63 | 267.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| 10 | 30.00 | 8.00 | 8.63 | 297.00 | 112.00 | 111.37 | 0.63 | 111.69 | 0.02 | | 0.01 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **0.01** |

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 | Heritage Park Master Plan Update |
| Project Number | 22-32-125-01 |
| Test Number | PT-01 |
| Test Location | 33.701087, -117.778963 |
| Personnel | Aleksey Zhukov |
| Presoak Date | 11/7/2022 |
| Test Date | 11/9/2022 |

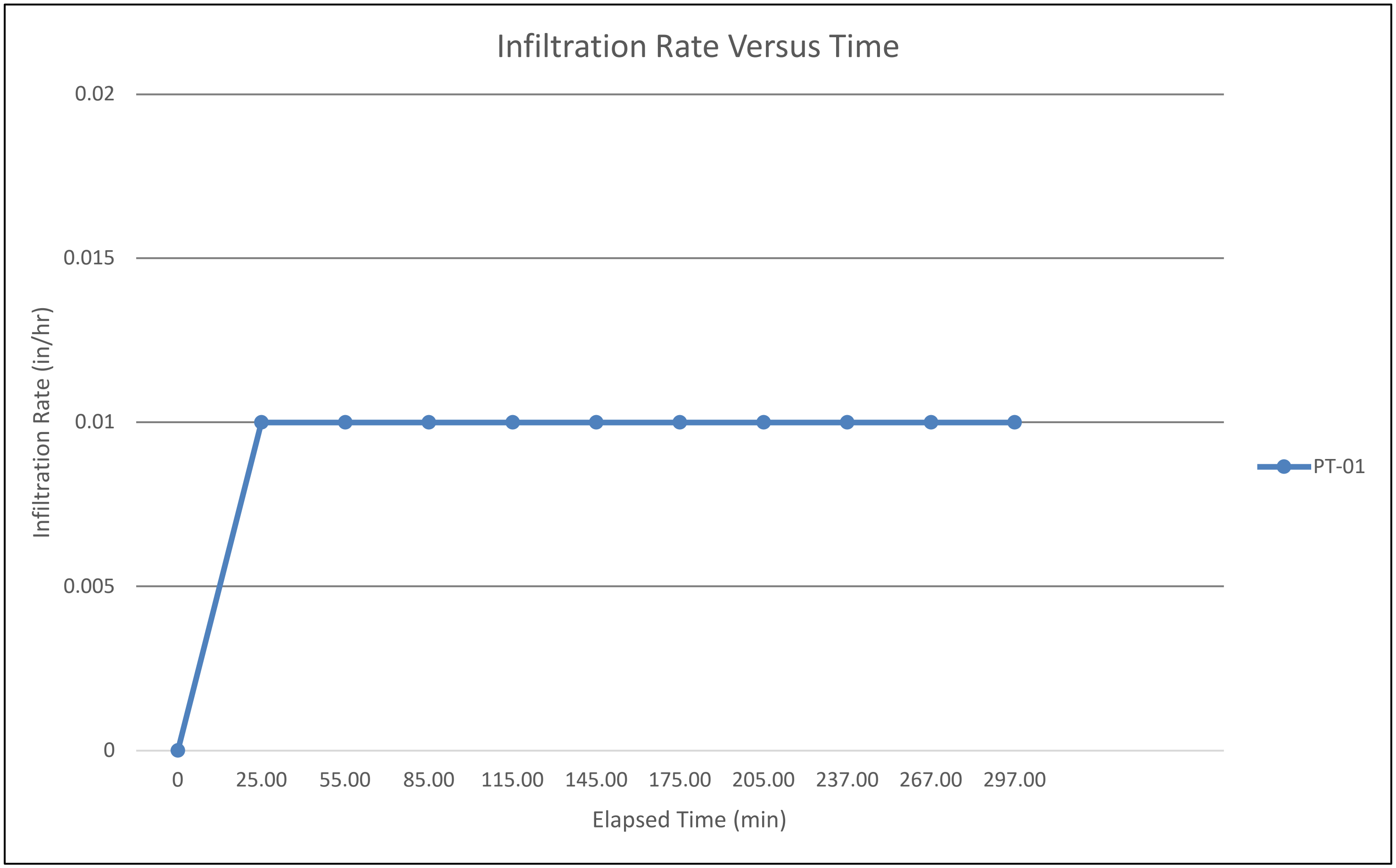



**Plate No. 2**

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

| | |
|---|---|
| Project Name | Heritage Park Master Plan Update |
| Project Number | 22-32-125-01 |
| Test Number | PT-02 |
| Test Location | 33.702110, -117.777103 |
| Personnel | Aleksey Zhukov |
| Presoak Date | 11/7/2022 |
| Test Date | 11/9/2022 |

Shaded cells contain calculated values.

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 120 |
| Inside Diameter of Pipe, I (inches) | 3.00 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| | |
| Factor of Safety (FOS), F | 2 |

| 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 | 14.10 | 14.34 | 25.00 | 105.90 | 105.66 | 0.24 | 105.78 | 0.01 | | 0.01 |
| 2 | 25.00 | 14.34 | 14.58 | 50.00 | 105.66 | 105.42 | 0.24 | 105.54 | 0.01 | | 0.01 |
| 3 | 30.00 | 7.50 | 8.00 | 80.00 | 112.50 | 112.00 | 0.50 | 112.25 | 0.02 | | 0.01 |
| 4 | 30.00 | 7.50 | 8.00 | 110.00 | 112.50 | 112.00 | 0.50 | 112.25 | 0.02 | | 0.01 |
| 5 | 30.00 | 7.50 | 8.00 | 140.00 | 112.50 | 112.00 | 0.50 | 112.25 | 0.02 | | 0.01 |
| 6 | 30.00 | 7.00 | 7.50 | 170.00 | 113.00 | 112.50 | 0.50 | 112.75 | 0.02 | | 0.01 |
| 7 | 30.00 | 7.50 | 7.88 | 200.00 | 112.50 | 112.12 | 0.38 | 112.31 | 0.01 | | 0.01 |
| 8 | 30.00 | 7.50 | 7.88 | 230.00 | 112.50 | 112.12 | 0.38 | 112.31 | 0.01 | | 0.01 |
| 9 | 30.00 | 7.50 | 7.88 | 260.00 | 112.50 | 112.12 | 0.38 | 112.31 | 0.01 | | 0.01 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **0.01** |

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. 3**

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

| | |
|---|---|
| Project Name | Heritage Park Master Plan Update |
| Project Number | 22-32-125-01 |
| Test Number | PT-02 |
| Test Location | 33.702110, -117.777103 |
| Personnel | Aleksey Zhukov |
| Presoak Date | 11/7/2022 |
| Test Date | 11/9/2022 |

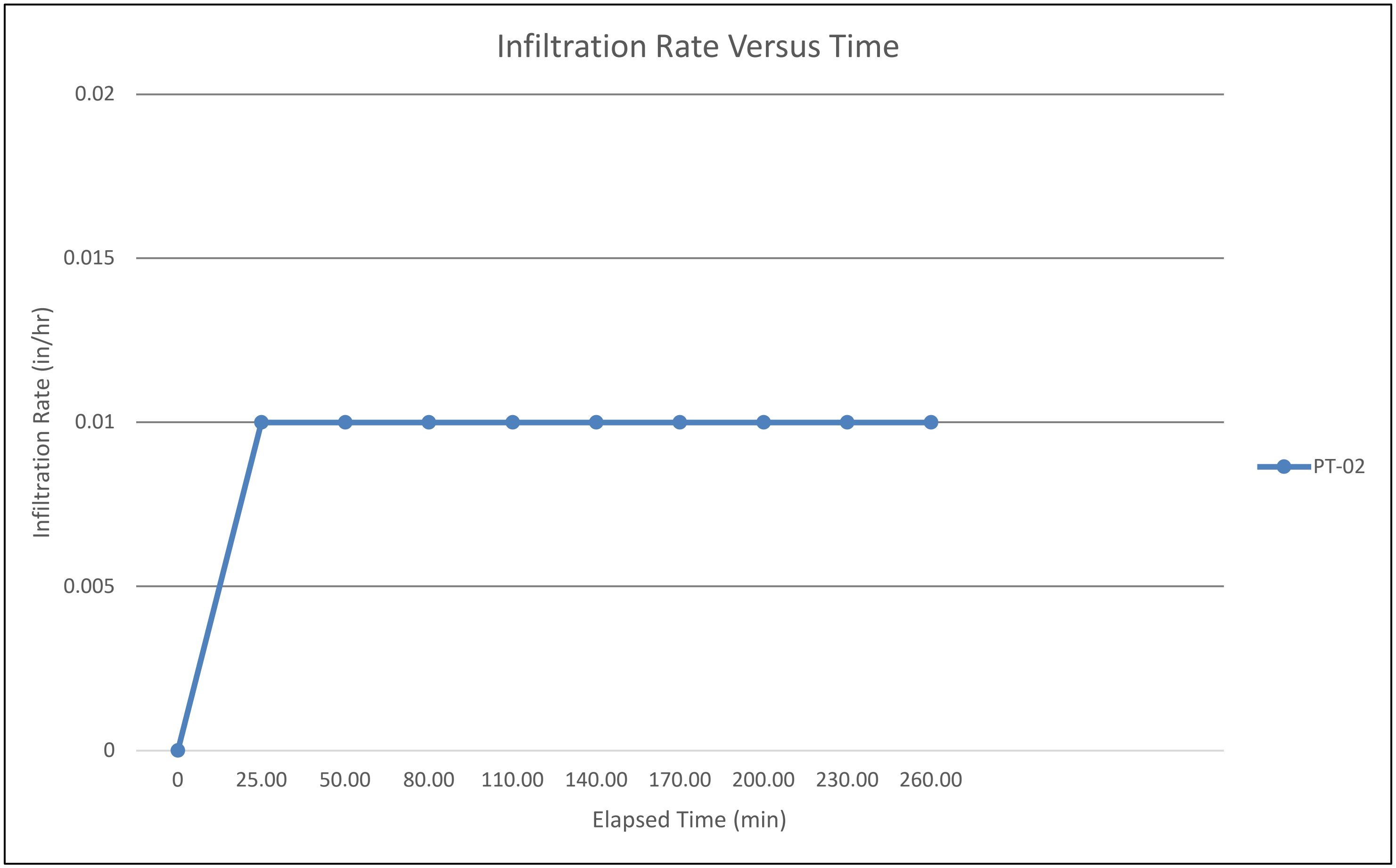



**Plate No. 4**