GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT

10TH AND FLOWER STREET PARK
SOUTHEAST CORNER OF 10TH STREET AND FLOWER STREET
CITY OF SANTA ANA, ORANGE COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 23-32-103-01

Prepared For:

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August 15, 2023

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

August 15, 2023

Mr. Mark S. Oskorus, PE, QSP
Project Manager, Land Development
David Evans and Associates, Inc.
17782 East 17th Street, Suite 200
Tustin, California 92780

Subject: **GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT**
10th Street & Flower Street Park
Southeast Corner of 10th Street and Flower Street
City of Santa Ana, Orange County, California
Converse Project No. 23-32-103-01

Dear Mr. Oskorus:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation and water percolation test report to assist with the design and construction of a new park, located at the southeast corner of the intersection of 10th Street and Flower Street City of Santa Ana, Orange County, California. The report was prepared in accordance with our revised proposal dated February 21, 2023, and your Acceptance of Agreement and Authorization to Proceed through email dated June 19, 2023.

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 David Evans and Associates, Inc. (DEA), and the City of Santa Ana. 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/EH/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

Elizabeth Hernandez
Staff Geologist

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

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation and water percolation tests performed for the 10th and Flower Street Park project, located at the southeast corner of the intersection of 10th Street and Flower Street City of Santa Ana, Orange County, California. The approximate location of the site is shown on Figure No. 1, *Approximate Site Location Map*.

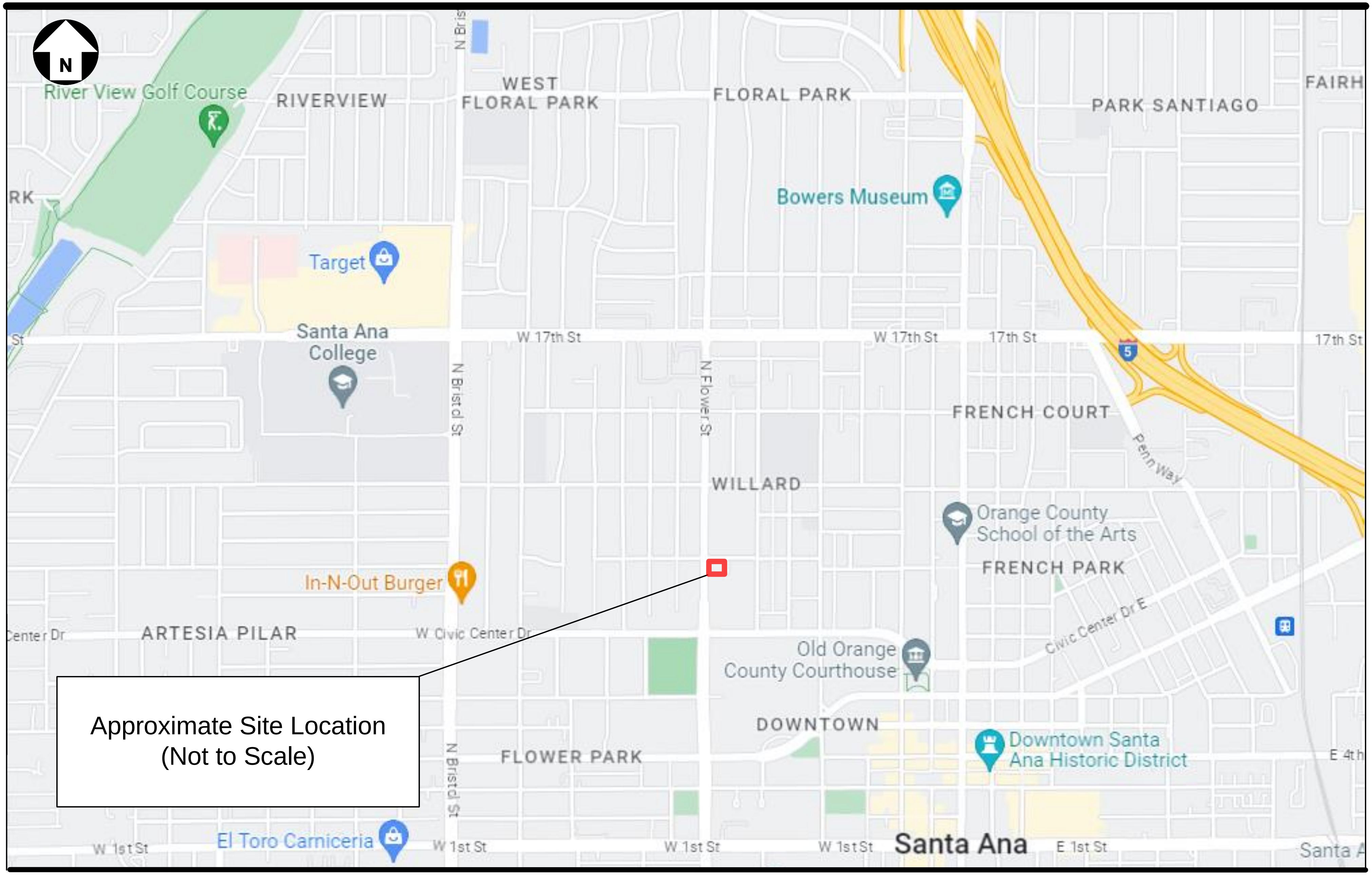
The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils and to provide recommendations for site earthwork, and design and construction of the proposed improvements.

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

2.0 PROJECT DESCRIPTION

The intent of the proposed park improvement is to provide the local community with access to open space, exercise, and recreation facilities. Based on input from the local community, the project will include, but is not limited to, the following project components.

- Playground area, with rubberized play surface
- Exercise area
- Canvas shade structures
- Turf play area
- Site lighting (new electrical service)
- Water wise landscaping, with shade trees
- Automated irrigation system (new water service)
- Site furnishings (benches, bike racks, trash receptacles, drinking fountain, picnic tables)
- Park monument sign
- Perimeter ROW improvements
- WQMP, as applicable
- Potential features may include A skateboard court, basketball court and pedestrian bridge
- Demolition of Garnsey Avenue



Project: 10th and Flower Street Park
Location: Southeast corner of 10th and Flower Streets
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Approximate Site Location Map

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23-32-103-01

Converse Consultants

Figure No.
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3.0 SITE DESCRIPTION

The proposed new 0.65-acre neighborhood park is located at the southeast corner of 10th Street and Flower Street, City of Santa Ana, California, just north of Civic Center Boulevard. The undeveloped portion of the site is surrounded by an approximately 6-foot steel fence with vehicular and pedestrian access points on the east side.

The site is bounded by 10th Street to the north, Garnsey Street to the east, parking lot to the south, and Flower Street to the west. The approximate elevation of the site varies between 109 and 111 feet above mean sea level (AMSL). The coordinates for the project site are approximately 33.7533° north latitude and 117.8762° west longitude.

The site is currently comprised of six separate parcels (APN's 005-142-34, 005-142-35, 005-142-47, 005-142-48, 005-142-49, 005-142-58). The six lots are currently identified as Low-Density Residential (LR-7) land use and zoned Professional (P). They are all currently vacant areas.

Surficial material appears to be comprised of a thin layer of engineered wood fiber. There are trees at the outside perimeter of the fence and overhead utilities on the east side of Garnsey Street. *Photographs Nos. 1 through* 3 depict the present the site conditions.

Photograph No. 1: Current site conditions facing southwest.

Photograph No. 2: Current site conditions facing northeast.

Photograph No. 3: Current conditions at Garnsey Street facing southeast.

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.
- 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

Six exploratory borings (BH-01 through BH-06) were drilled on July 17, 2023, to investigate the subsurface conditions within the footprint of the proposed improvements. The borings were drilled to depths between 6.5 and 31.5 feet below ground surface (bgs). Borings details are presented in Drawings Nos. A-2 through A-7, *Logs of Borings.*

After soil samples were collected, borings BH-04 and BH-06 were set up for percolation testing and will be referred to as BH-04/PT-01 and BH-06/PT-02 hereafter. The depths of BH-04/PT-01 and BH-06/PT-02 were restricted to 5.5 and 10.4 feet bgs respectively. Details about the percolation tests are presented in Appendix C, *Percolation Testing*.

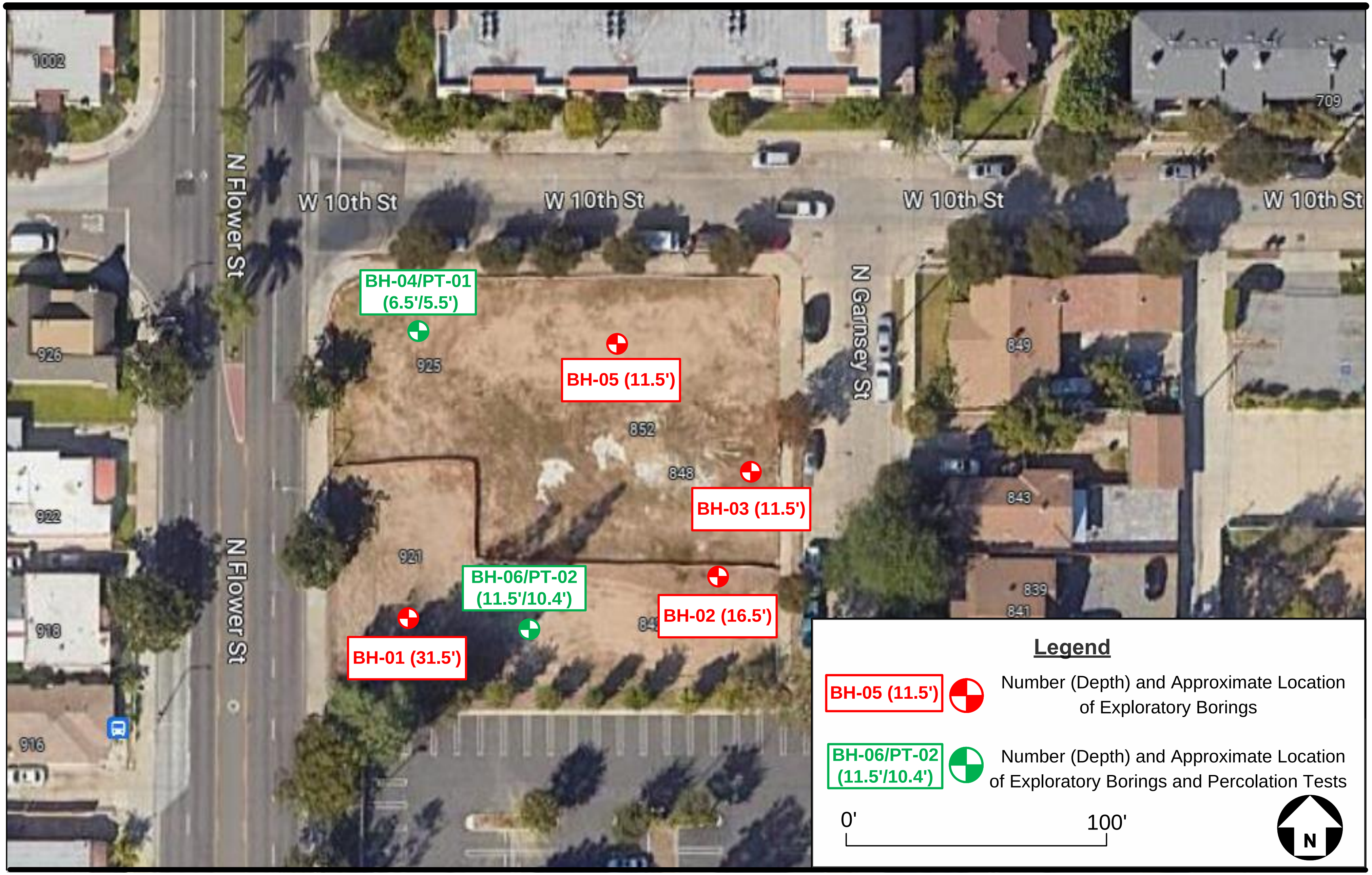
The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling.

The approximate locations of the borings are shown on Figure No. 2, *Approximate Boring and Percolation Tests Locations Map.* A detailed discussion of the subsurface exploration program 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)
- Soil corrosivity (California Tests 422, 417, and 643)
- Collapse potential (ASTM D4546)
- Grain size distribution (ASTM D6913)
- Maximum dry density and optimum moisture content (ASTM D1557)



Project: 10th and Flower Street Park
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Approximate Borings and Percolation Test Locations Map

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Figure No.
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- 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.

5.1 Subsurface Profile

Fill: Fill soils were identified up to 5 feet below ground surface (bgs.) and is likely associated with previous grading for the existing structures. The fill soils were likely derived from on-site sources and are similar to the native alluvial soils in composition and density. Based on the exploratory borings and laboratory test results, the subsurface fill soils at the site consisted primarily of a mixture of gravel, sand, silt, and clay. Few gravel up to 2.25-inch in maximum dimension was observed in the fill soils.

Alluvium: Based on the exploratory borings and laboratory test results, the subsurface alluvium soils at the site consisted primarily of a mixture of gravel, sand, silt, and clay. Gravel up to 2.0-inches in largest dimension was observed in the borings.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-7 *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 31.5 feet bgs. Converse used the generalized coordinates of 33.753290 N, 117.876200 W for the following section.

For comparison, regional groundwater data from the GeoTracker database (SWRCB, 2023) 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 the search is provided below.

- ORANGE COUNTY GSA (Site No. # T0605900574) located approximately 1,545 feet southwest of the project site reported groundwater at depths ranging from 6.8 to 52.3 feet bgs between 1993 and 2013.
- THRIFTY OIL #376 (Site No. # T0605900847) located approximately 2,601 feet southwest of the project site reported groundwater at depths ranging from 11.8 to 61.8 feet bgs between 1998 and 2017.
- ARCO #1782 (Site No. # T0605985148) located approximately 2,797 feet northwest of the project site reported groundwater at depths ranging from 41.7 to 58.5 feet bgs between 2005 and 2013.
- THRIFTY OIL #008 (Site No. # T0605900604) located approximately 2,985 feet southwest of the project site reported groundwater at depths ranging from 21.1 to 51.1 feet bgs between 1996 and 2022.
- ARCO #5038 (Site No. #T0605900517) located approximately 3,017 feet northeast of the project site reported groundwater at depths ranging from 78.9 to 85.8 feet bgs between 2001 and 2004.
- SHELL #51 FORMER (Site No. # T0605902056) located approximately 3,223 feet southwest of the project site reported groundwater at depths ranging from 20.9 to 60.4 feet bgs between 1995 and 2023.
- CHEVRON Products Service Station #9-5783 (Site No. #T10000000219), located approximately 3,571 feet northwest of the project site reported groundwater at depths ranging from 54.1 to 63.6 feet bgs between 2008 and 2011.
- FORMER APOLLO MUFFLER (Site No. # T0605970234) located approximately 3,804 feet southwest of the project site reported groundwater at depths ranging from 18.4 to 44.0 feet bgs between 2008 and 2009.
- MAIN STREET CAR WASH (Site No. # T0605901853) located approximately 4,052 feet northeast of the project site reported groundwater at depths ranging from 80.2 to 87.9 feet bgs between 2011 and 2012.
- UNLOCAL #7470 (Site No. # T0605901964) located approximately 4,055 feet southwest of the project site reported groundwater at depths ranging from 10.2 to 48.8 feet bgs between 1998 and 2023.
- LAUTERBACK FAMILY TRUST (Site No. #T0605902309) located approximately 4,076 feet northeast of the project site reported groundwater at depths ranging from 83.0 to 89.4 feet bgs between 1999 and 2012.
- NUNEZ AUTO REPAIR (Site No. # T0605999167) located approximately 4,133 feet southeast of the project site reported groundwater at depths ranging from 52.8 to 68.6 feet bgs between 2002 and 2013.

The National Water Information System (USGS, 2023) 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
334443117523401	Corner of W 1st St and N Flower St; approximately 2,954 feet south of project site	46.0	1980

The California Department of Water Resources database (DWR, 2023) 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 provided in the following list.

- Well No. 05S10W12L005S (Station 337501N1178769W001), located approximately 1,228 feet southwest of the project site, reported groundwater at a depth ranging from 139 to 314 feet bgs between 2008 and 2010.
- Well No. 05S10W13B003S (Station 337452N1178762W001), located approximately 2,967 feet southeast of the project site, reported groundwater at a depth ranging from 63.5 to 130 feet bgs between 1969 and 1980.
- Well No. 05S10W13C001S (Station 337450N1178774W001), located approximately 2,979 feet southwest of the project site, reported groundwater at a depth ranging from 61.2 to 147 feet bgs between 1969 and 1985.

Based on available data, the historical high groundwater level reported at wells within approximately one mile of the site was 6.8 bgs. Current groundwater is expected to be deeper than about 31.5 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 Collapse Potential

Soil deposits subjected to collapse/hydro-consolidation generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without an increase in external loads. Moreover, some soils may have a different degree of collapse/hydro-consolidation based on the amount of proposed fill or structure loads. Soils susceptible to collapse/hydro-consolidation include wind-blown silt, weakly cemented sand, and silt where the cementing agent is soluble (e.g., soluble gypsum, halite), alluvial or colluvial deposits within semi-arid to arid climate, and certain weathered bedrock above the groundwater table.

Granular soils may have a potential to collapse upon wetting in arid climate regions. Collapse/hydro-consolidation may occur when the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose/low density configuration from deposition.

The degree of collapse of a soil can be defined by the collapse potential value, which is expressed as a percentage of collapse of the total sample using the Collapse Potential Test (ASTM D4546). According to the ASTM guideline, the severity of collapse potential is commonly evaluated by the following Table No. 2, *Collapse Potential Values*.

Table No. 2, Collapse Potential Values

Collapse Potential Value (%)	Severity of Problem
0	None
0.1 to 2	Slight
2.1 to 6.0	Moderate
6.0 to 10.0	Moderately Severe
>10	Severe

Based on the laboratory test result (collapse potentials of +1.6 and -5.0 percent), minor swell to moderate collapse potential is anticipated at the site. Collapse potential distress is typically considered a concern when collapse potential is over 2% (LA County, 2013).

5.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade. Depending on the extent and location below finish subgrade, expansive soils can have a detrimental effect on structures. Based on the laboratory test results, the expansion indices of the upper 5 feet of site soil were 0 and 11, corresponding to very low expansion potential.

5.5 Excavatability

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

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. The selection of appropriate excavation equipment models should be done by an experienced earthwork contractor.

5.6 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 (Artificial Fill) – *In-situ* dry density and moisture content of the fill soils were determined in accordance with ASTM Standard D2216 and D2937*.* The dry densities of upper 5.0 feet ranged from 78 to 117 pcf with moisture contents ranging from 9 to 20 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- *In-situ* Moisture and Dry Density (Alluvium) – *In-situ* dry density and moisture content of the alluvial site soils were determined in accordance with ASTM Standard D2216 and D2937*.* The dry densities of the site's alluvial soils ranged from 83 to 124 pcf with moisture contents ranging from 4 to 28 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Expansion Index (EI) – Two representative soil samples were tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test results showed an EI of 0 and 11, corresponding to very low expansion potential.
- R-Value (R) – One representative bulk sample was tested in accordance with Caltrans Test Method 301. The result of the R-value test was 35.
- Collapse Potential (CL) – 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 +1.6 (swell) and -5.0, indicating minor swell to moderate collapse potential.
- Grain Size Analysis (PA) – Two representative soil samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*

- Maximum Dry Density and Optimum Moisture Content (CP) – The moisture-density relationship of a representative soil sample was tested in according to ASTM Standard D1557 and the result is presented in Drawing No. B-2, *Moisture-Density Relationship Result*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 131.5 pounds per cubic feet (pcf) with optimum moisture content of 8.6 percent. With rock correction, the dry density was 134.6 pounds per cubic feet (pcf) with optimum moisture content of 7.8 percent.
- Direct Shear (DS) – Two direct shear tests were performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The results of the direct shear tests are presented in Drawing Nos. B-3 and B-4, *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 8.8.
- The soluble sulfate content of the sample was 41 ppm (0.0041 percent by weight).
- The chloride concentration of the sample was 43 ppm.
- The minimum electrical resistivity (wet condition) of the sample when saturated was 2,216 ohm-cm.

7.0 PERCOLATION TESTING

Two percolation tests (BH-04/PT-01 and BH-06/PT-02) were performed on July 18, 2023, 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. 3, Estimated Infiltration Rates

Percolation Test	Depth of Boring* (feet)	Predominant Soil Types (USCS)	Design Percolation Rate (inches/hour)
BH-04/PT-01	5.5	Sandy Clay (CL) and Silty Sand (SM)	0.31
BH-06/PT-02	10.4	Sandy Silt (ML) and Sand with Silt (SP-SM)	2.29

(* Approximate depth)

A design infiltration rate of 0.31 and 2.29 (inches/hour) can be used for depths of 5.5 and 10.4 feet respectively. 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.

The project site is not located within a currently mapped State of California Earthquake Fault Zone for surface fault rupture. Table No. 4, *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. 4, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
San Joaquin Hills	6.84	thrust	27	0.5	7.10
Puente Hills (Coyote Hills)	13.77	thrust	17	0.7	6.90
Newport Inglewood Connected alt 2	13.85	strike slip	208	1.3	7.50
Newport-Inglewood, alt 1	13.98	strike slip	65	1.0	7.20
Newport Inglewood Connected alt 1	13.98	strike slip	208	1.3	7.50
Newport-Inglewood (Offshore)	18.35	strike slip	66	1.5	7.00
Elsinore	18.71	strike slip	241	n/a	7.85
Puente Hills (Santa Fe Springs)	23.73	thrust	11	0.7	6.70
Chino, alt 2	27.03	strike slip	29	1.0	6.80
Chino, alt 1	27.09	strike slip	24	1.0	6.70
Palos Verdes Connected	31.34	strike slip	285	3.0	7.70
San Jose	31.73	strike slip	20	0.5	6.70
Puente Hills (LA)	33.32	thrust	22	0.7	7.00
Elysian Park (Upper)	40.66	reverse	20	1.3	6.70

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Sierra Madre Connected	42.31	reverse	76	2.0	7.30
Sierra Madre	42.31	reverse	57	2.0	7.20
Cucamonga	43.43	thrust	28	5.0	6.70
Raymond	45.86	strike slip	22	1.5	6.80
Clamshell-Sawpit	48.44	reverse	16	0.5	6.70
Verdugo	49.19	reverse	29	0.5	6.90
Hollywood	52.2	strike slip	17	1.0	6.70
Coronado Bank	52.64	strike slip	186	3.0	7.40
Santa Monica Connected alt 2	54.66	strike slip	93	2.4	7.40
Santa Monica, alt 1	60.73	strike slip	14	1.0	6.60
Santa Monica Connected alt 1	60.73	strike slip	79	2.6	7.30
San Jacinto	63.87	strike slip	241	n/a	7.88
Malibu Coast, alt 1	67.42	strike slip	38	0.3	6.70
Malibu Coast, alt 2	67.42	strike slip	38	0.3	7.00
S. San Andreas	69.23	strike slip	548	n/a	8.18
Anacapa-Dume, alt 2	69.25	thrust	65	3.0	7.20
Sierra Madre (San Fernando)	69.97	thrust	18	2.0	6.70
Cleghorn	72.6	strike slip	25	3.0	6.80
San Gabriel	72.93	strike slip	71	1.0	7.30
Northridge	77.23	thrust	33	1.5	6.90
Anacapa-Dume, alt 1	80.02	thrust	51	3.0	7.20
Rose Canyon	81.59	strike slip	70	1.5	6.90
North Frontal (West)	83.94	reverse	50	1.0	7.20
Santa Susana, alt 1	85.39	reverse	27	5.0	6.90
Holser, alt 1	96.35	reverse	20	0.4	6.80
Simi-Santa Rosa	97.34	strike slip	39	1.0	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. 5, CBC Seismic Design Parameters

Parameter	Value
Site Coordinates	33.7533 N, 117.8762 W
Risk Category	II
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	1.301g
Mapped 1-second Spectral Response Acceleration, S_1	0.464g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.836
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.301g
MCE 1-second period Spectral Response Acceleration, S_{M1}	0.852g
Design Spectral Response Acceleration for short period S_{DS}	0.868g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.568g
Peak Ground Acceleration, PGA_M	0.605g

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 Earthquake Fault Zone (CGS, 2007). 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 improvement of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

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

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

Based on review of Seismic Hazard Zones map for the Orange Quadrangle (1998), the project site is not located within areas mapped for liquefaction zones by State of California (CGS, 1998).

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 medium dense, to dense and clayey soil is present. 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. The project site is not located within a currently designated State of California Landslide Zone (CGS, 2007), 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. Due to the low risk of liquefaction and dense nature of the soil materials, the risk of lateral spreading is considered low.

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

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Due to the 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 State of California designated dam inundation area (DSOD, 2023). Both dams at Villa Park, No. 1012-0 and Santiago Creek, No. 75-0, have the potential for earthquake-induced flooding. The risk of earthquake-induced flooding at the project site due to failure of offsite dams is considered extremely high.

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 new park amenities. 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. 6, Overexcavation Depths

Structure/Pavement	Minimum Excavation Depth
Footings	18 inches below footing bottoms, or 3 feet below existing ground surface, whichever is deeper
Slabs	18 inches below slab bottom or 2.5 feet below existing ground surface, whichever is deeper
Pavement	12 inches below pavement or 2.0 feet below existing ground surface, whichever is deeper

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.
- 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 10.8, *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 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 10 percent can be used for the upper 10 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.

9.6 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 for 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 improvements 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. 7, Recommended Foundation Parameters

Parameter	Value
Minimum continuous spread footing width	18 inches
Minimum isolated footing width	18 inches
Minimum continuous or isolated footing depth of embedment below lowest adjacent grade	18 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 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.

10.3.1 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. 8, 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
Seismic earth pressure (psf)	22 H

* H = height of the 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.

10.3.2 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 250 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.

10.4 Drilled Pier Foundations

The light posts 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 110 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 250 pounds per square foot per foot of depth may be used for the design.

10.5 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 31.5 feet bgs and due to the presence of sandy soil with gravel, there is some 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 no 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.

10.6 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.

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.

10.7 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.

10.8 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. 9, Correlation Between Resistivity and Corrosion

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

The measured value of the minimum electrical resistivity of the sample when saturated was 2,216 Ohm-cm. This indicates that the soil tested of the site is moderately corrosive to ferrous metals in contact with the soils (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site and site soils.

10.9 Flexible Pavement Recommendations

For pavement design, we have utilized an R-value of 35 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.

Table No. 10, 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)
35	5	4.0	3.0	4.0
	6	4.5	4.0	5.0
	7	5.0	6.0	6.5
	8	5.5	8.0	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.

10.10 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 35 and design Traffic Indices (TIs) ranging from 5 to 8. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table.

Table No. 11, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
35	5	6.5
	6	7.0
	7	7.0
	8	7.5

The preceding 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.

10.11 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 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. 12, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal: Vertical) [1]
Silty Sand (SM), Sandy Silt (ML), and Sandy Clay (CL)	C	0-4	Vertical
		4-10	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 David Evans and Associates, Inc., the City of Santa Ana, 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

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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 conducting water percolation tests. During the site reconnaissance, the surface conditions were noted, and the borings were marked at locations approved by Mr. Mark Oskorus with DEA. The boring locations were established in the field using approximate distances from existing features as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Six exploratory borings (BH-01 through BH-06) were drilled on July 17, 2023, to investigate the subsurface conditions within the footprint of the proposed improvements. The borings were drilled to depths between 6.5 and 31.5 feet below ground surface (bgs). Borings details are presented in Table No. A-1, *Summary of Borings* and Drawings Nos. A-2 through A-7, *Logs of Borings.*

Table No. A-1, Summary of Borings

Boring No.	Boring Depth (ft, bgs)		Groundwater Depth (ft, bgs)	Date Completed
	Proposed	Completed		
BH-01	30.0	31.5	Not Encountered	7/17/2023
BH-02	15.0	16.5	Not Encountered	7/17/2023
BH-03	10.0	11.5	Not Encountered	7/17/2023
BH-04/PT-01	5.0	6.5	Not Encountered	7/17/2023
BH-05	10.0	11.5	Not Encountered	7/17/2023
BH-06/PT-02	10.0	11.5	Not Encountered	7/17/2023

After logging and soil sampling, borings BH-04 and BH-06 were set up for percolation testing and will be referred to as BH-04/PT-01 and BH-06/PT-02 hereafter. The depths of BH-04/PT-01 and BH-06/PT-02 were restricted to 5.5 and 10.4 feet bgs respectively due to caving in the test holes. 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 borings BH-01, BH-02, BH-03, and BH-05 were 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 BH-04/PT-01 and BH-06/PT-02 and boreholes were backfilled with soil cuttings and compacted. If construction is delayed, the surface of the borings may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing Nos. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawing Nos. A-2 through A-7, *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

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

Dates Drilled: 7/17/2023 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): 110 Depth to Water (ft, bgs): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

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 Test Pit 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.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blows	Moisture (%)	Dry Unit Wt. (pcf)	Other
	ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 2.25" in maximum dimension, trace clay, moist, dark brown.		X				CP, EI
	SANDY SILT (ML): fine to medium-grained sand, scattered gravel up to 1.5" in maximum dimension, trace clay, rootlets, glass and brick debris, stiff, moist, dark brown to black.	X	X	16/7/9	12	117	
5		X	X	11/18/23	6	124	CL, DS
	ALLUVIUM : **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 0.5" in maximum dimension, trace pockets of clay, weakly cemented, dense, moist, brown.	X	X	10/17/20	5	118	
10	**- 7.5':** fine to medium-grained, no gravel, caliche fragments up to 0.5" in maximum dimension.	X		13/18/20	6	95	
	SILT WITH SAND (ML): fine-grained sand, non-plastic, very stiff, moist, light brown.						
15	**CLAY (CL):** trace fine-grained sand, trace silt, scattered caliche fragments, very stiff, moist, dark brown.	X		9/17/25	13	117	
20	**SANDY SILT (ML):** fine to medium-grained sand, low plasticity, weakly cemented, very stiff, moist, tannish brown.	X		8/14/22	8	120	
25	**- 25.0':** no cementation, brown.	X		6/10/14	13	116	
30	**- 30.0':** scattered gravel up to 0.25" in maximum dimension.	X		11/14/20	11	124	

End of boring at 31.5 feet below ground surface.
No groundwater was encountered.
Boring was backfilled with soil cuttings mixed with cement and compacted with augers using the drill rig weight on 7/17/23.

Converse Consultants

10th and Flower Park
Southeast corner of 10th and Flower Street
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Project No. **23-32-103-01** Drawing No. **A-2**

Log of Boring No. BH-02

Dates Drilled: 7/17/2023 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): 110 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS 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 Test Pit 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
		ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 1" in maximum dimension, few clay, roots, broken glass debris, moist, dark brown.						CR, EI
		SANDY CLAY (CL): fine to medium-grained sand, few silt, moist, dark brown to black.			5/6/9	20	105	
5		**ALLUVIUM :** **SANDY SILT (ML):** fine to medium-grained sand, trace clay, stiff, moist, brown.			5/8/13	17	111	
		SILTY SAND (SM): fine to medium-grained, scattered caliche fragments 0.5" in maximum dimension, weakly cemented, dense, moist, light brown.			10/24/25	7	108	DS
10		**SAND WITH SILT (SP-SM):** fine to medium-grained, medium dense, moist, orangish light brown.			12/13/15	4	97	
15		**CLAY WITH SAND (CL):** fine-grained sand, trace silt, very stiff, moist, dark brown.			9/18/25	14	114	

End of boring at 16.5 feet below ground surface.
No groundwater was encountered.
Boring was backfilled with soil cuttings mixed with cement and compacted with augers using the drill rig weight on 7/17/23.

Log of Boring No. BH-03

Dates Drilled: 7/17/2023 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): 111 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS 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 Test Pit 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
		ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.5" in maximum dimension, few clay, roots, moist, dark brown.		X				R
		- 2.5': fine to medium-grained, no gravel, increasing clay content, roots and rootlets, medium dense.	X	X	10/9/8	18	78	
5		**ALLUVIUM :** **SANDY SILT (ML):** fine to medium-grained sand, trace clay, weakly cemented, stiff, moist, brown.	X		6/7/10	15	87	
		SILTY SAND (SM): fine to coarse-grained, few gravel up to 2" in maximum dimension, rootlets, weakly cemented, medium dense, moist, brown.	X		7/9/12	8	101	
10		**SILTY CLAY WITH SAND (CL-ML):** fine to coarse-grained sand, scattered rounded gravel 1.5" in maximum dimension, medium stiff, moist, dark brown.	X		4/4/5	28	95	

End of boring at 11.5 feet below ground surface.
No groundwater was encountered.
Boring was backfilled with soil cuttings mixed with cement and compacted with augers using the drill rig weight on 7/17/23.

Log of Boring No. BH-04/PT-01

Dates Drilled: 7/17/2023 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): 111 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS 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 Test Pit 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
		ARTIFICIAL FILL **SANDY CLAY (CL):** fine to coarse-grained sand, scattered gravel up to 1.5" in maximum dimension, few silt, moist, dark brown.		X				PA
		- 2.5': increasing clay content, very stiff, dark brown to black.	X	X	16/22/27	9	100	
5		**ALLUVIUM :** **SILTY SAND (SM):** fine to medium-grained, scattered caliche fragments up to 0.5" in maximum dimension, weakly cemented, dense, moist, brown.	X		17/28/31	6	101	

End of boring at 6.5 feet bgs.
Groundwater was not encountered.
Borehole was used for percolation testing.
After completeion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 7/18/2023.

Log of Boring No. BH-05

Dates Drilled: 7/17/2023 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): 111 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS 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 Test Pit 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
		ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, wood chips, moist, brown to black.						
		ALLUVIUM : **SANDY CLAY (CL):** fine to medium-grained sand, few to little silt, scattered caliche fragments up to 0.25" in maximum dimension, very stiff, moist, light brown to brown.			8/16/19	9	118	CL
5					16/21/28	9	119	
		SANDY SILT (ML): fine to medium-grained sand, few caliche fragments, weakly cemented, very stiff, moist, tannish brown. **- 7.5':** no caliche fragments.			10/12/18	4	103	
10		**SAND WITH SILT (SP-SM):** fine to coarse-grained, few gravel up to 1" in maximum dimension, whitish yellow brown.			11/8/14	13	103	
		SANDY CLAY (CL): fine to medium-grained sand, few silt, stiff, moist, brown.						

End of boring at 11.5 feet below ground surface.
No groundwater was encountered.
Boring was backfilled with soil cuttings mixed with cement and compacted with augers using the drill rig weight on 7/17/23.

Converse Consultants

10th and Flower Park
Southeast corner of 10th and Flower Street
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Project No. **23-32-103-01**

Drawing No. **A-6**

Log of Boring No. BH-06/PT-02

Dates Drilled: 7/17/2023 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): 110 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS 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 Test Pit 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
		ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.25" in maximum dimension, few clay, moist, dark brown. **- 2.5':** scattered gravel up to 1" in maximum dimension, increasing clay content, medium dense, dark brown to black.	X		4/9/12	10	117	
5		**ALLUVIUM :** **SANDY SILT (ML):** fine to medium-grained sand, low plasticity, stiff, moist, tannish brown. **- 7.5':** weakly cemented, very stiff.	X	X	4/8/11	7	96	PA
			X		11/18/28	4	97	
10		**SAND WITH SILT (SP-SM):** fine to coarse-grained, scattered gravel up to 0.75" in maximum dimension, medium dense, moist, whitish yellow brown.	X		6/8/10	8	83	

End of boring at 11.5 feet bgs.
Groundwater was not encountered.
Borehole was used for percolation testing.
After completeion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 7/17/2023.

Appendix B

Laboratory Testing Program

APPENDIX B

LABORATORY TESTING PROGRAM

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

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

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

Expansion Index (EI)

Two representative bulk samples were tested to evaluate the expansion potential of material encountered at the site in accordance with ASTM D4829 Standard. The test results are presented in the following table.

Table No. B-1, Expansion Index Test Results

Boring No.	Depth (feet)	Soil Classification	Expansion Index	Expansion Potential
BH-01	0-5	Silty Sand/Sandy Silt (SM/ML)	0	Very Low
BH-02	0-5	Silty Sand/Sandy Clay (SM/CL)	11	Very Low

R-Value (R)

One representative bulk soil sample was tested by AP Engineering and Testing, Inc. (Pomona, CA) 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	Silty Sand (SM)	35

Soil Corrosivity (CR)

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

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

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	8.8	41	43	2,216

Collapse Potential (CL)

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, two collapse tests were performed on a relatively undisturbed ring samples 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-4, Collapse Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-01	5.0-6.5	Silty Sand (SM)	-5.0	Moderate
BH-05	2.5-4.0	Sandy Clay (CL)	+ 1.6	None

Grain-Size Analyses (PA)

To assist in soil classification, mechanical grain-size analyses were performed on two select samples in accordance with the ASTM Standard D6913. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results* and summarized in the following table.

Table No. B-5, Grain Size Distribution Test Results

Boring No.	Depth (feet)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-04/PT-01	0-5	Sandy Clay (CL)	1.0	38.0	61.0	
BH-06/PT-02	5-10	Sandy Silt (ML)	0.0	37.2	62.8	

Maximum Dry Density and Optimum Moisture Content (CP)

Laboratory maximum dry density-optimum moisture content relationship test was performed on one representative bulk sample. This test was conducted in accordance with the ASTM Standard D1557 test method. The test result is presented in Drawing No. B-2, *Moisture-Density Relationship Result,* and summarized in the following table.

Table No B-6, Summary of Moisture-Density Relationship Result

Boring No.	Depth (feet)	Soil Classification	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-01	0-5	Silty Sand/Sandy Silt (SM/ML), Dark Brown to Black	131.5 (134.6*)	8.6 (7.8*)

(* Rock correction: BH-01 = 10.5%)

Direct Shear (DS)

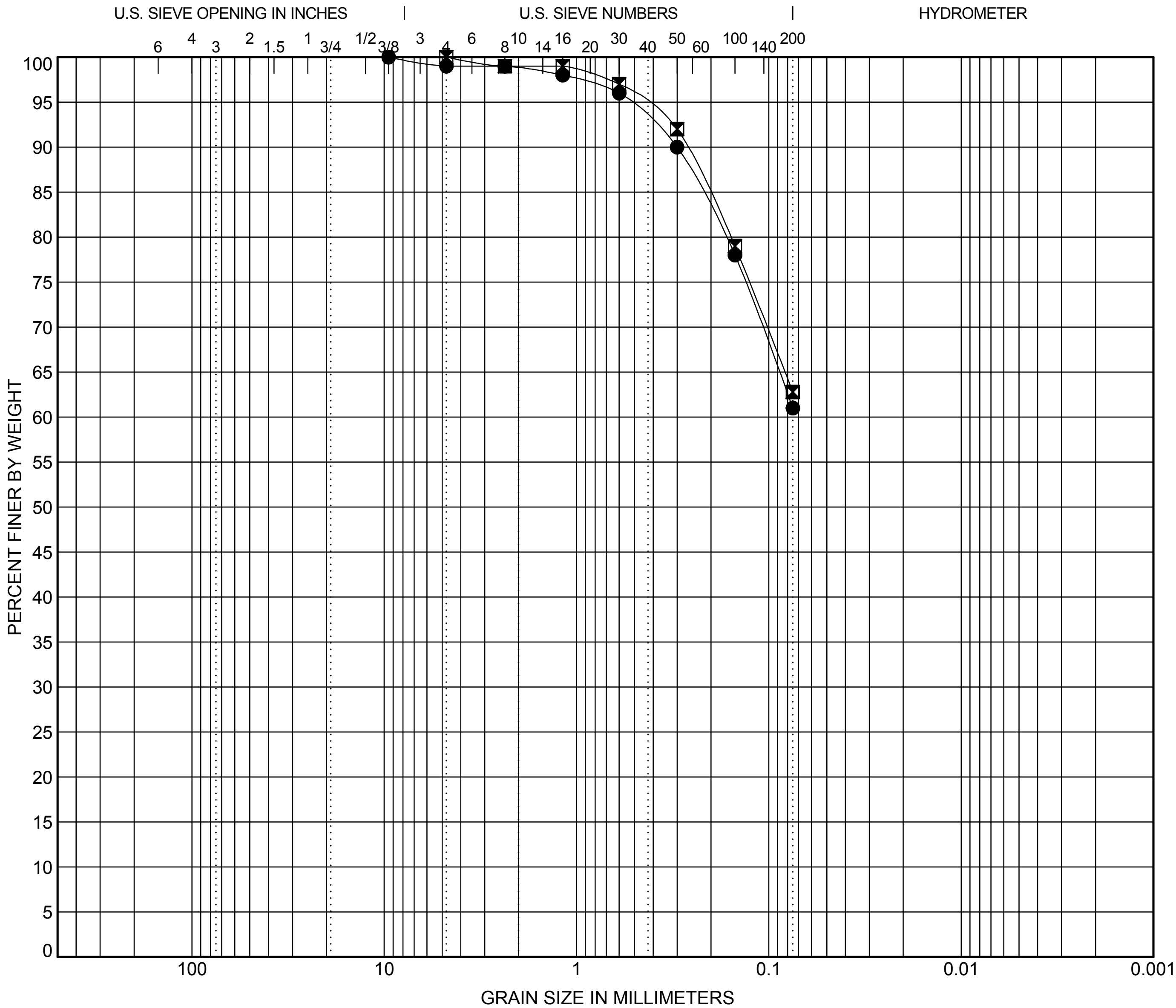
Two direct shear tests were performed on relatively undisturbed samples under soaked moisture conditions in accordance with ASTM D3080. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. Each sample was then sheared at a constant strain rate of 0.02 inches/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 Nos. B-3 and B-4*, Direct Shear Test Results,* and the following table.

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

Boring No.	Depth (feet)	Soil Description	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-01	5.0-6.5	Silty Sand (SM)	33	70
BH-02	7.5-9.0	Silty Sand (SM)	31	100

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-04/PT-01**	**0-5**	**SANDY CLAY (CL)**					
⊠	**BH-06/PT-02**	**5-10**	**SANDY SILT (ML)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-04/PT-01**	**0-5**	**9.5**				**1.0**	**38.0**	**61.0**	
⊠	**BH-06/PT-02**	**5-10**	**4.75**				**0.0**	**37.2**	**62.8**	

GRAIN SIZE DISTRIBUTION RESULTS

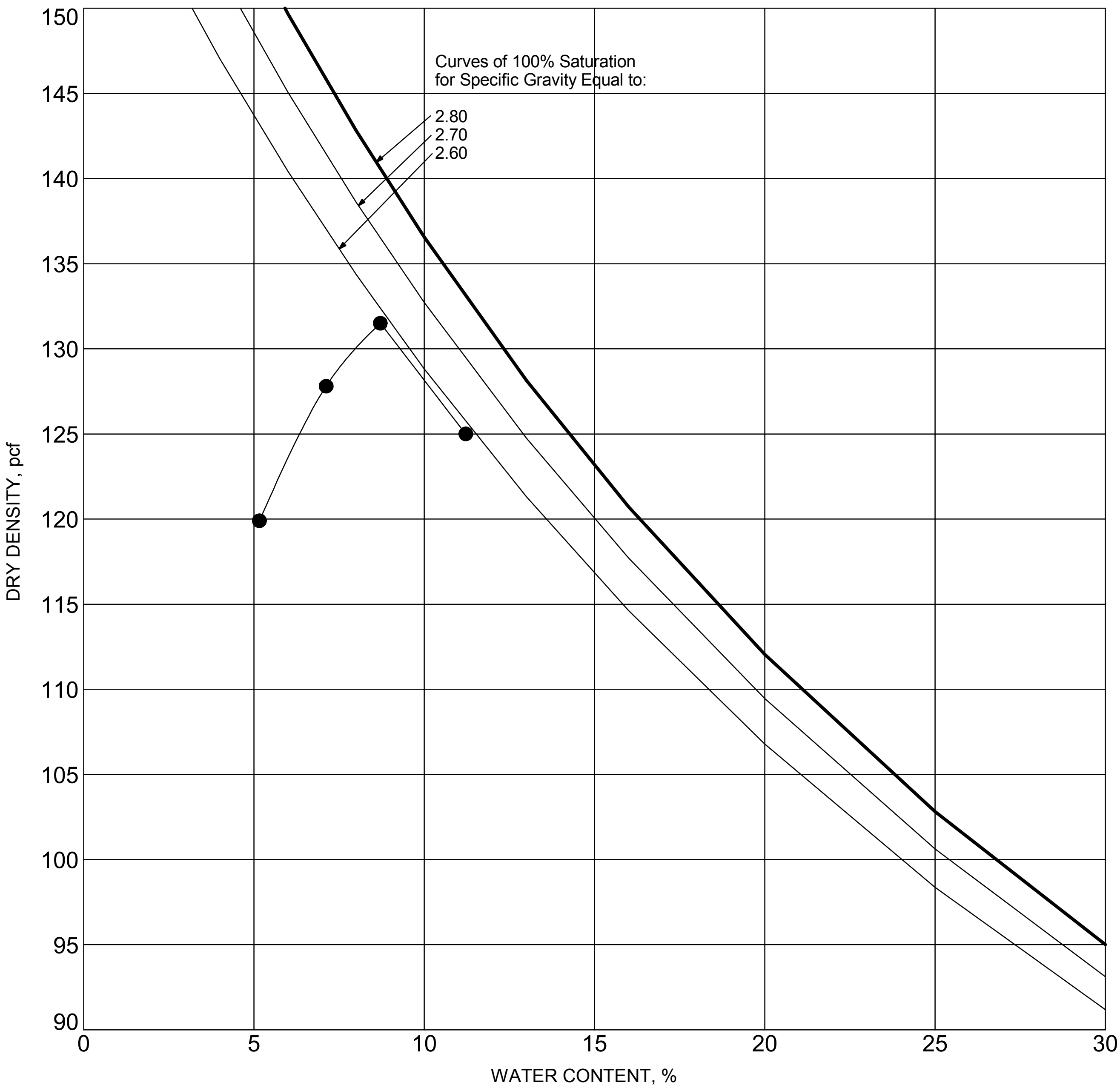
Converse Consultants

10th and Flower Park
Southeast corner of 10th and Flower Street
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Project No. **23-32-103-01**

Drawing No. **B-1**

Project ID: 23-32-103-01.GPJ; Template: GRAIN SIZE



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**0-5**	**SILTY SAND/SANDY SILT (SM/ML), DARK BROWN TO BLACK**	**D1557 Method A**	**8.6 (7.8*)**	**131.5 (134.6*)**

(* Rock correction: BH-01 = 10.5%)

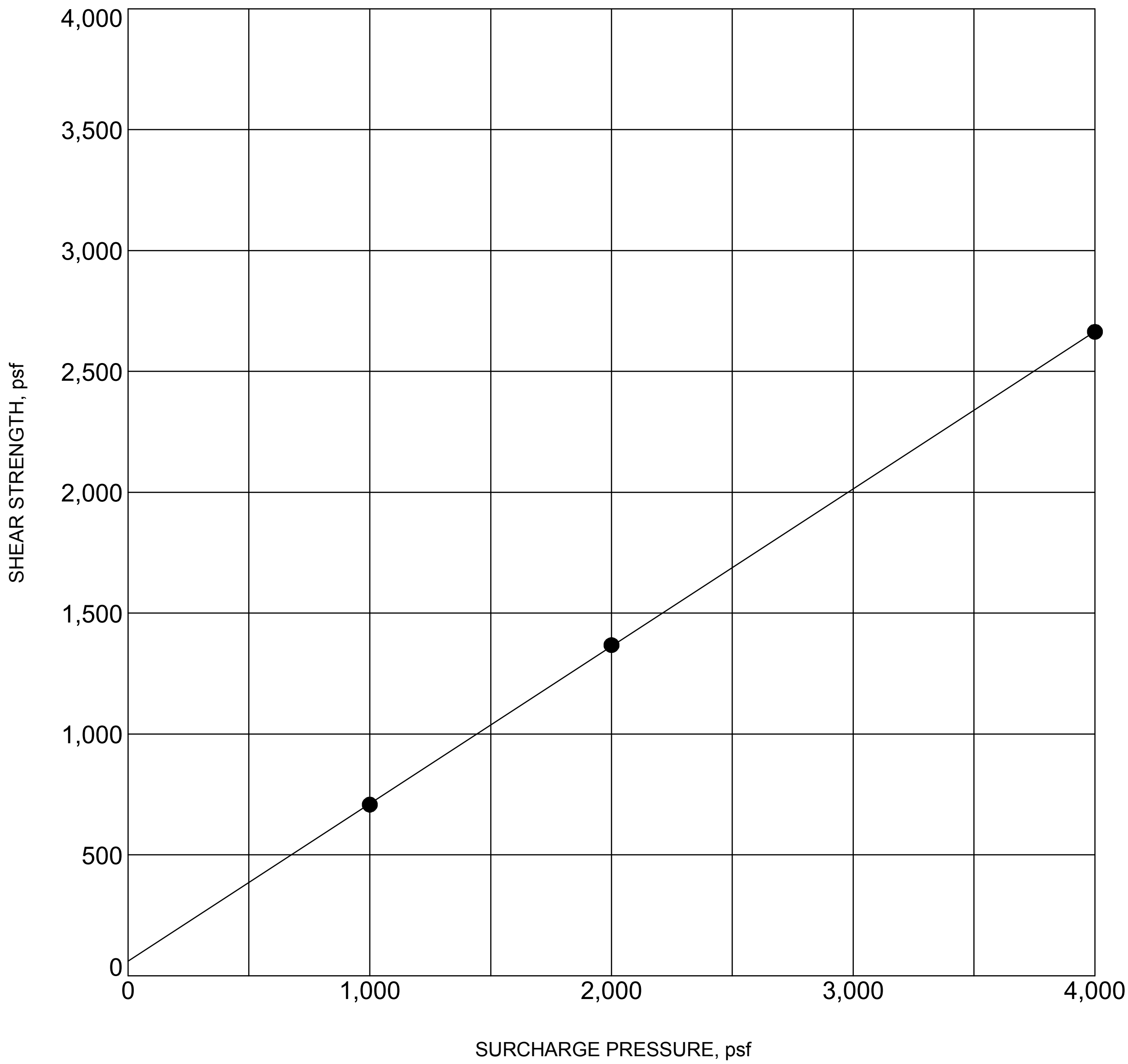
MOISTURE-DENSITY RELATIONSHIP RESULT

Converse Consultants

10th and Flower Park
Southeast corner of 10th and Flower Street
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Project No. **23-32-103-01**

Drawing No. **B-2**



BORING NO. :	**BH-01**	DEPTH (ft) :	**5.0-6.5**
DESCRIPTION :	**SILTY SAND (SM)**		
COHESION (psf) :	**70**	FRICTION ANGLE (degrees):	**33**
MOISTURE CONTENT (%) :	**6**	DRY DENSITY (pcf) :	**124**

NOTE: Ultimate Strength.

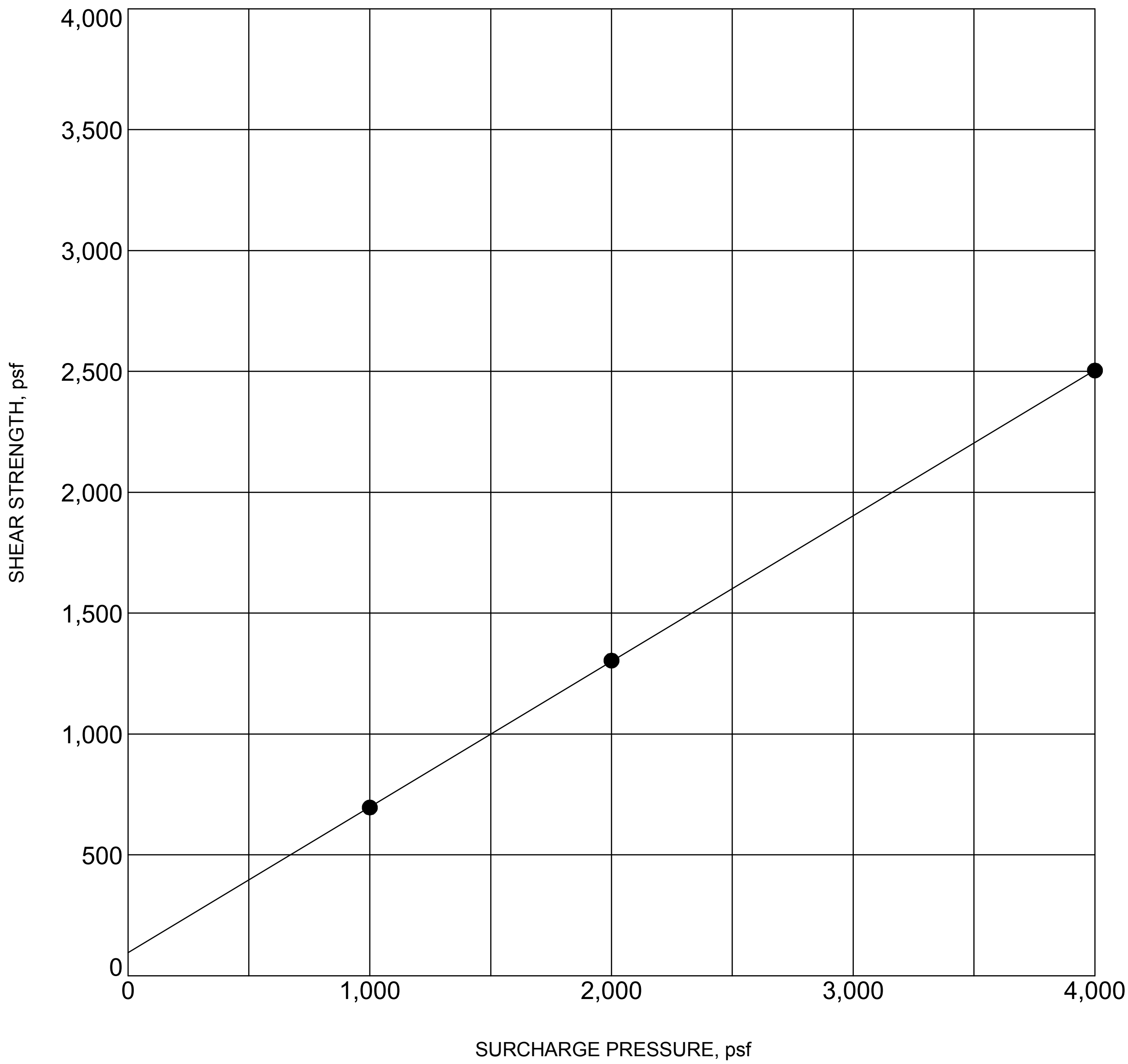
DIRECT SHEAR TEST RESULTS

Converse Consultants

10th and Flower Park
Southeast corner of 10th and Flower Street
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Project No. **23-32-103-01**

Drawing No. **B-3**



BORING NO.	:	**BH-02**	DEPTH (ft)	:	**7.5-9.0**
DESCRIPTION	:	**SILTY SAND (SM)**			
COHESION (psf)	:	**100**	FRICTION ANGLE (degrees):		**31**
MOISTURE CONTENT (%)	:	**7**	DRY DENSITY (pcf)	:	**108**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

10th and Flower Park
Southeast corner of 10th and Flower Street
City of Santa Ana, Orange County, California
For: David Evans and Associates, Inc.

Project No. **23-32-103-01**

Drawing No. **B-4**

Appendix C

Percolation Testing

APPENDIX C

PERCOLATION TESTING

Percolation testing were performed at two locations (BH-04/PT-01 and BH-06/PT-02), on July 18, 2023, 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 for BH-04/PT-01 and every 10 minutes for BH-06/PT-02 respectively. Following the completion of percolation testing, the pipes were removed from BH-04/PT-01 and BH-06/PT-02, and boreholes were backfilled with soil cuttings and compacted.

Percolation rates describe the movement of water horizontally and downward into the soil from a boring. Infiltration rates describe the downward movement of water through a horizontal surface, such as the floor of a retention basin. Percolation rates are related to infiltration rates but are generally higher and require conversion before use in 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 Nos. 1 through 4. 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)
BH-04/PT-01	5.5	Sandy Clay (CL) and Silty Sand (SM)	0.31
BH-06/PT-02	10.4	Sandy Silt (ML) and Sand with Silt (SP-SM)	2.29

A design infiltration rate of 0.31 and 2.29 (inches/hour) can be used for depths of 5.5 and 10.4 feet respectively. 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, BH-04/PT-01

Project Name	10th and Flower Park
Project Number	23-32-103-01
Test Number	BH-04/PT-01
Test Location	33.753542, -117.876277
Personnel	Aleksey Zhukov
Presoak Date	7/17/2023
Test Date	7/18/2023

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	66
Inside Diameter of Pipe, I (inches)	3.00
Outside Diameter of Pipe, O (inches)	3.12
Factor of Safety (FOS), F	2

Interval No.	Time Interval, Δt (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, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)		Infiltration Rate with FOS, I_f (inches/hr)
				0							0
1	30.00	42.24	47.52	30.00	23.76	18.48	5.28	21.12	0.91		0.46
2	30.00	42.24	47.52	60.00	23.76	18.48	5.28	21.12	0.91		0.46
3	30.00	40.80	46.44	90.00	25.20	19.56	5.64	22.38	0.93		0.46
4	30.00	40.44	46.68	120.00	25.56	19.32	6.24	22.44	1.02		0.51
5	30.00	39.84	45.96	150.00	26.16	20.04	6.12	23.10	0.98		0.49
6	30.00	39.84	45.72	180.00	26.16	20.28	5.88	23.22	0.93		0.47
7	30.00	41.28	46.56	210.00	24.72	19.44	5.28	22.08	0.88		0.44
8	30.00	40.08	45.24	240.00	25.92	20.76	5.16	23.34	0.81		0.41
9	30.00	39.72	44.40	270.00	26.28	21.60	4.68	23.94	0.72		0.36
10	30.00	39.24	43.56	300.00	26.76	22.44	4.32	24.60	0.65		0.32
11	30.00	38.52	42.84	330.00	27.48	23.16	4.32	25.32	0.63		0.32
12	30.00	37.20	41.64	360.00	28.80	24.36	4.44	26.58	0.62		0.31

Recommended Design Infiltration Rate (inches/hr)	**0.31**

Orange County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (Orange 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, BH-04/PT-01

Project Name	10th and Flower Park
Project Number	23-32-103-01
Test Number	BH-04/PT-01
Test Location	33.753542, -117.876277
Personnel	Aleksey Zhukov
Presoak Date	7/17/2023
Test Date	7/18/2023

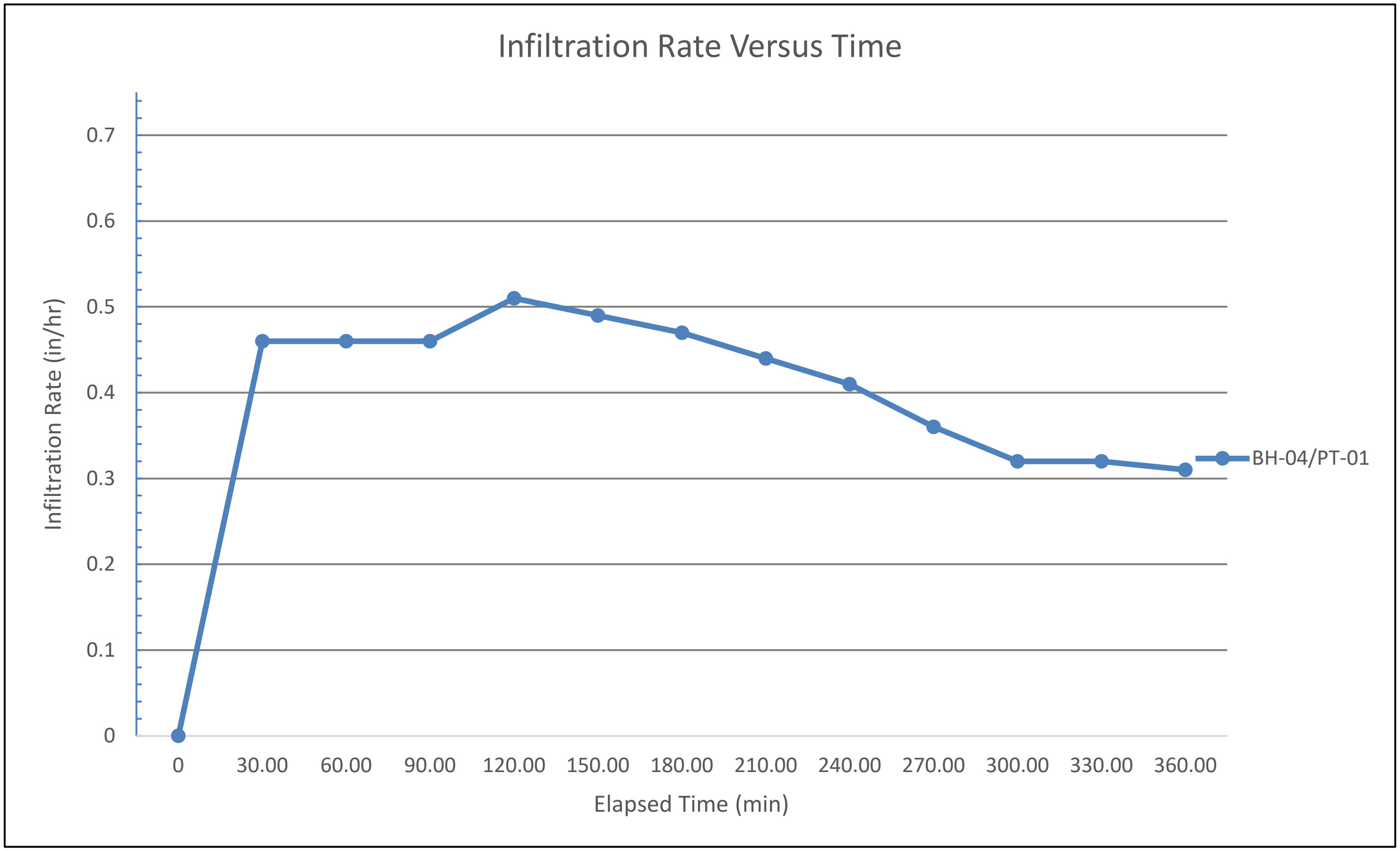


Plate No.
2

Estimated Infiltration Rate from Percolation Test Data, BH-06/PT-02

Project Name	10th and Flower Park
Project Number	23-32-103-01
Test Number	BH-06/PT-02
Test Location	33.753196, -117.876125
Personnel	Aleksey Zhukov
Presoak Date	7/17/2023
Test Date	7/17/2023

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	125
Inside Diameter of Pipe, I (inches)	3.00
Outside Diameter of Pipe, O (inches)	3.12
Factor of Safety (FOS), F	2

Interval No.	Time Interval, Δt (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, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)		Infiltration Rate with FOS, I_f (inches/hr)
				0							0
1	10.00	36.00	69.84	10.00	89.00	55.16	33.84	72.08	5.48		2.74
2	10.00	37.92	68.28	20.00	87.08	56.72	30.36	71.90	4.93		2.46
3	10.00	46.20	72.96	30.00	78.80	52.04	26.76	65.42	4.76		2.38
4	10.00	44.76	71.76	40.00	80.24	53.24	27.00	66.74	4.71		2.36
5	10.00	44.76	71.76	50.00	80.24	53.24	27.00	66.74	4.71		2.36
6	10.00	44.40	70.92	60.00	80.60	54.08	26.52	67.34	4.59		2.29

Recommended Design Infiltration Rate (inches/hr)	**2.29**

Orange County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (Orange 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, BH-06/PT-02

Project Name	10th and Flower Park
Project Number	23-32-103-01
Test Number	BH-06/PT-02
Test Location	33.753196, -117.876125
Personnel	Aleksey Zhukov
Presoak Date	7/17/2023
Test Date	7/17/2023

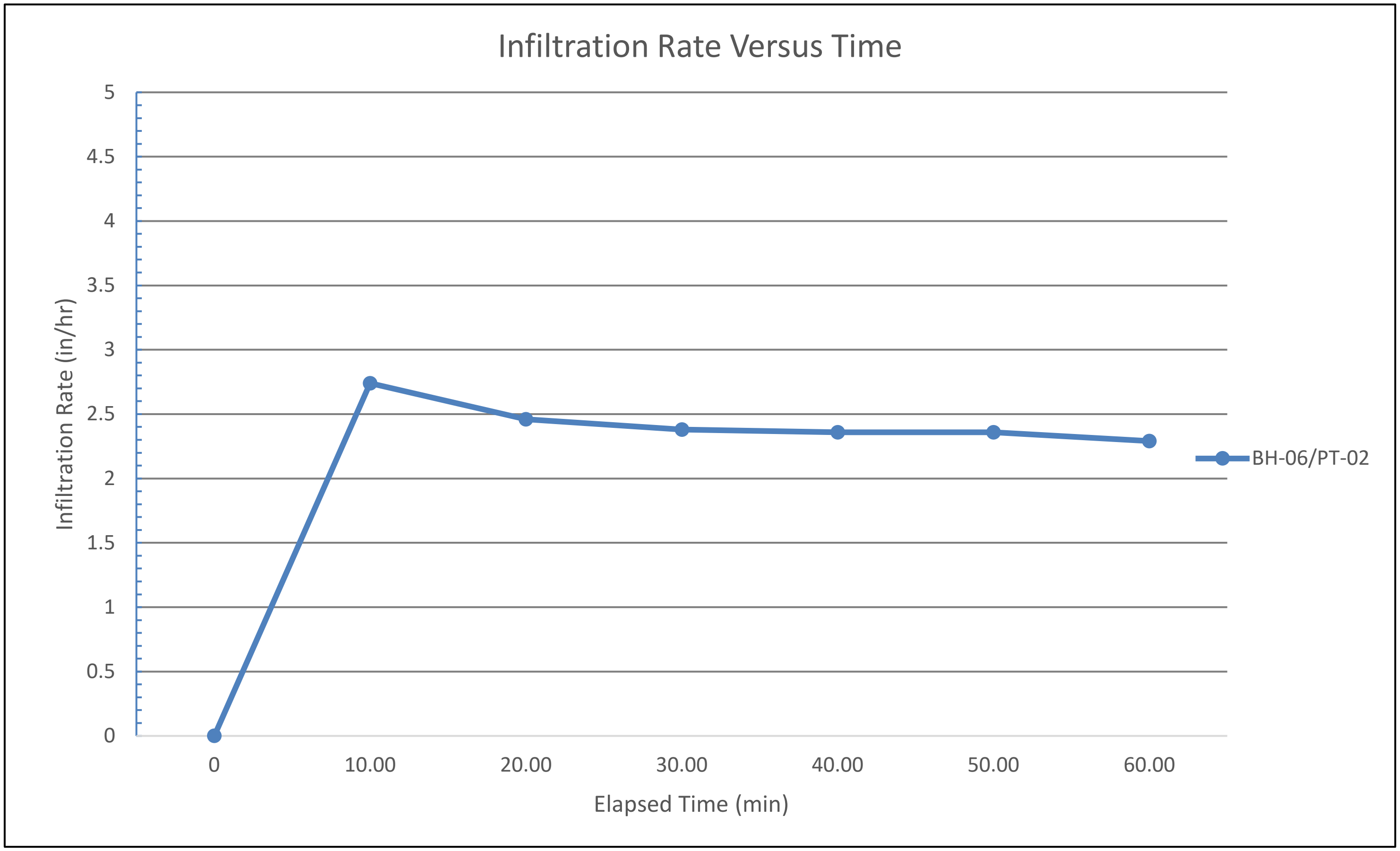


Plate No.
4