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

GEOTECHNICAL INVESTIGATION AND WATER INFILTRATION TEST REPORT

BLOOMINGTON ANIMAL SHELTER
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California

CONVERSE PROJECT NO. 22-81-206-01

Prepared For:
MILLER ARCHITECTURAL CORPORATION
1177 Idaho Street, Suite 200
Redlands, CA 92374

Presented By:
CONVERSE CONSULTANTS
2021 Rancho Drive, Suite 1
Redlands, CA 92373
909-796-0544

January 18, 2023

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

January 18, 2022

Mr. Gary Miller
President/CEO
Miller Architectural Corporation
1177 Idaho Street, Suite 200
Redlands, CA 92374

Subject: **GEOTECHNICAL INVESTIGATION AND WATER INFILTRATION TEST REPORT**
Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
Converse Project No. 22-81-206-01

Dear Mr. Miller:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation and water infiltration test report to assist with the design and construction of the Bloomington Animal Shelter project located at 18313 Valley Blvd. in the Bloomington Area, San Bernardino County, California. This report was prepared in accordance with our proposal dated June 16, 2022, your Acceptance of Agreement and Authorization to Proceed dated November 3, 2022.

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

We appreciate the opportunity to be of service to Miller Architectural Corporation and San Bernardino County Real Estate Services, Department of Project Management. Should you have any questions, please do not hesitate to contact us at 909-474-2847.

CONVERSE CONSULTANTS

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

Dist: 1-Electronic PDF/Addressee
HSQ/SR/SM/kvg

2021 Rancho Drive, Suite 1, Redlands, CA 92373
Telephone: (909) 796-0544 ♦ Facsimile: (909) 796-7675 ♦ www.converseconsultants.com

PROFESSIONAL CERTIFICATION

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

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

SK Syfur Rahman, PhD, EIT
Sr. Staff Engineer

Stephen McPherson
Staff Geologist

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

REGISTERED PROFESSIONAL ENGINEER
HASHMI S. E. QUAZI
NO. GE 2517
Exp 06/2023
GEOTECHNICAL
STATE OF CALIFORNIA

TABLE OF CONTENTS

FIGURES

TABLES

APPENDICES

1.0 INTRODUCTION

This report contains the findings of the geotechnical investigation performed by Converse to assist with the design and construction of the Bloomington Animal Shelter located at 18313 Valley Boulevard Bloomington Area of San Bernardino County, California. The approximate location of the project is shown in Figure No. 1, *Approximate Project Location Map.*

The purposes of this investigation were to evaluate the nature and engineering properties of the subsurface soils and groundwater conditions, and to provide geotechnical recommendations for the design and construction of the proposed project.

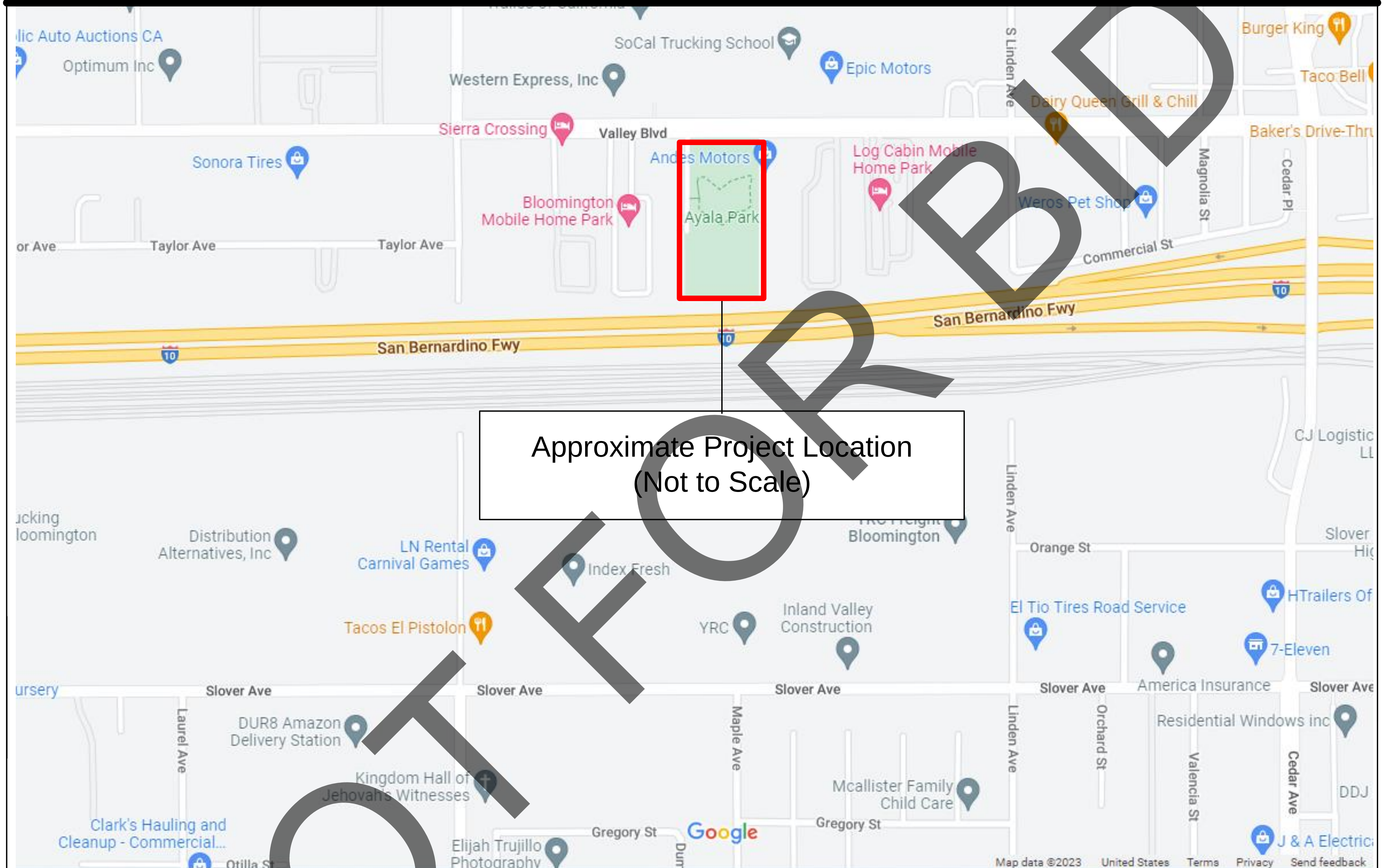
This report was prepared for the project described herein and is intended for use solely by Miller Architectural Corporation, San Bernardino County Real Estate Services-Project Management, and their authorized agents. This report may be made available to the prospective bidders for bidding purposes. However, the bidders are responsible for their own interpretation of the site conditions between and beyond the boring locations, based on factual data contained in this report. This report may not contain sufficient information for use by others and/or other purposes.

2.0 PROJECT DESCRIPTION

According to the information provided by Miller Architectural Corporation, the Bloomington Animal Shelter project will consist of the following.

- 16,000 square feet building which will include the following
  - Animal housing
  - Administration
- Veterinary care building
- Animal intake
- Quarantine and isolation building/private area
- Barn
- Storage building
- 3 stall garages
- Power generator building
- Euthanasia building
- 10-foot-high x 8” thick CMU wall along the Interstate freeway 10 (I-10).
- 8-foot-high x 8” thick CMU wall along the east and west property lines.
- Outdoor community events for school group, tours, and presentations
- Trash disposal
- Segregated and covered parking

We have assumed that there will also be one water infiltration device installed within the project area. Also, associated with the above-mentioned development, there will be



Project: Bloomington Animal Shelter
Location: 18313 Valley Boulevard
Bloomington area of San Bernardino County, California

For: Miller Architectural Corporation

Approximate Project Location Map

Project No.
22-81-206-01

Converse Consultants

Figure No.
1

interior streets, concrete walkways, underground utilities, and landscaping. Based on the shallow relief on the site, it is anticipated that grading will consist of cuts and fills of up to about 5 feet or less.

3.0 SITE DESCRIPTION

The approximately 6-acre, 330' x 800' site is located in the unincorporated community of Bloomington in the San Bernardino Valley, surrounded by the cities of Rialto and Fontana in San Bernardino County, and Jurupa Valley in Riverside County. The site is bounded to the north by Valley Boulevard, to the west by residential properties, to the east by a used car lot and vacant lot and to the south by Interstate Freeway 10 (I-10).

A review of Google Maps indicates that Ayala Park was previously situated within the footprint of the proposed animal shelter location. Ayala Park had three to four enclosed structures, two gazebos, parking areas with associated access roads, a basketball court, children's play area, paved walkways, approximately fifty trees and grass covered parkland. At the time of the field investigation, all of the structures, paved areas, trees, and grassland had been removed with the exception of a utility box and the soil had been disced in preparation for the construction of the proposed Bloomington Animal shelter.

The subject site terrain is almost flat, gently slopes southward toward concrete storm drain channel along I-10. The site is presently fenced off and vacant. Photograph Nos. 1 and 2 depict the present site conditions.

Photograph No. 1, Present site conditions facing northeast from the eastern edge of the infiltration basin.

Photograph No. 2, Present site conditions facing north from the proposed cats building

4.0 SCOPE OF WORK

The scope of Converse's investigation is described in the following sections.

4.1 Project Set-up

We reviewed the following documents.

- Plans and documents for construction.
- Previous geologic/geotechnical publications of the site and surrounding area.
- Faulting and seismic hazard maps.
- Groundwater data.
- Aerial photographs.

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

- Prepared a geotechnical exploration plan and submitted it to Mr. Brent Adams with Miller Architectural Corporation for approval.
- Coordinated with Mr. Brent Adams for site access.
- Conducted a site reconnaissance and staked/marked the field exploration locations such that is available.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring locations of any conflict with existing underground utilities.
- Engaged a California-licensed driller to drill exploratory borings.

4.2 Subsurface Exploration

Eight borings (BH-01 through BH-08) were drilled on December 8, 2022, to investigate the subsurface conditions using a truck mounted drill rig equipped with an 8-inch diameter hollow stem auger for soil sampling. The borings were drilled to depths ranging between 5.0 and 50.0 feet below ground surface (bgs). Two test holes (PT-01 and PT-02) were drilled on December 8, 2023, to depths of 5.3 and 10.2 bgs, respectively to perform percolation testing. The boreholes were fit with perforated pipe for percolation testing that was performed on December 9, 2022.

The purpose of the borings was to:

- Estimate the extent and depths of remedial grading.
- Classify the soils within the borings.
- Collect soils samples for laboratory testing.
- Determine the excavatability of the soil.
- Preform percolation testing in two of the borings at depths of 5.3 and 10.2 feet bgs.

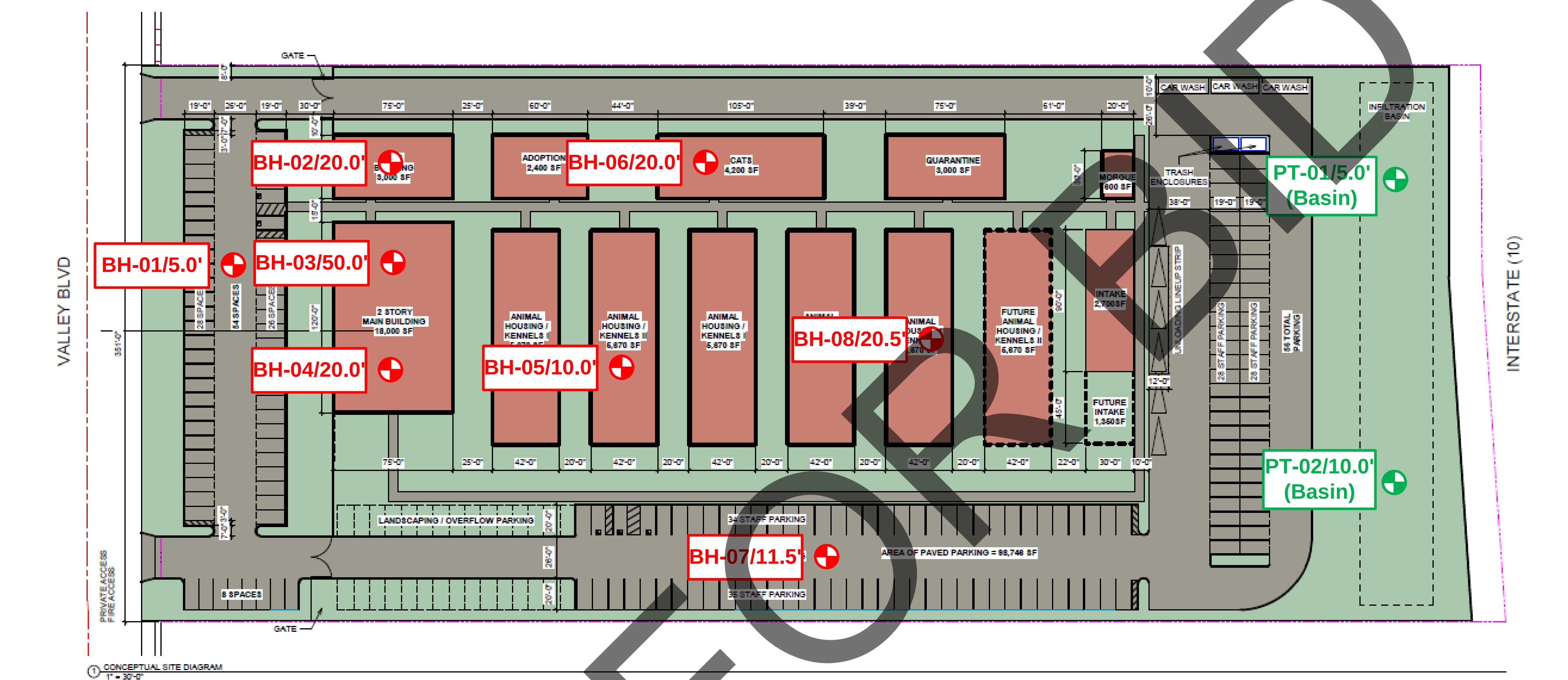
Details of these borings are presented in Table No. 1, *Summary of Borings*.

Table No. 1, Summary of Borings

Boring No.	Boring Depth (ft, bgs)		Groundwater Depth (ft, bgs)	Date Completed
	Proposed	Completed		
BH-01	5.0	5.0	N/E	12/8/2022
BH-02	20.0	20.0	N/E	12/8/2022
BH-03	50.0	50.0	N/E	12/8/2022
BH-04	20.0	20.0	N/E	12/8/2022
BH-05	10.0	10.0	N/E	12/8/2022
BH-06	20.0	20.0	N/E	12/8/2022
BH-07	10.0	11.5	N/E	12/8/2022
BH-08	20.0	20.5	N/E	12/8/2022
PT-01	5.0	5.3	N/E	12/8/2022
PT-02	10.0	10.2	N/E	12/8/2022

Note:
N/E = Not Encountered
For location of the borings, see Figure No. 2, *Approximate Boring Locations Map*.

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



BUILDING SQUARE FOOTAGES	
SQUARE FOOTAGE	
NAME	SQUARE FOOTAGE
MAIN BUILDING	18,000 SF
ANIMAL HOUSING / KENNELS I	5,670 SF
ANIMAL HOUSING / KENNELS II	5,670 SF
ANIMAL HOUSING / KENNELS III	5,670 SF
ANIMAL HOUSING / KENNELS IV	5,670 SF
ANIMAL HOUSING / KENNELS V	5,670 SF
INTAKE	2,700 SF
VET BUILDING	3,000 SF
ADOPTION	2,400 SF
CATS	4,200 SF
QUARANTINE	3,000 SF
MORGUE	800 SF
TOTAL:	62,250 SF
FUTURE SQUARE FOOTAGE	
FUTURE ANIMAL HOUSING / KENNELS I	5,670 SF
FUTURE INTAKE	1,350 SF
TOTAL:	7,020 SF
GRAND TOTAL:	69,270 SF

GRAPHIC SCALE
0 30 60 90 120

Legend

BH-04/20.5' Number/Depth and Approximate Location of Exploratory Borings

PT-06/10.0' Number/Depth and Approximate Location of Percolation Tests

N

Project: Bloomington Animal Shelter
Location: 18313 Valley Boulevard
Bloomington area of San Bernardino County, California

For: Miller Architectural Corporation

Approximate Boring and Percolation Test Locations Map

Project No.
22-81-206-01

Converse Consultants

Figure No.
2

4.3 Laboratory Testing

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

- *In-situ* moisture contents and dry densities (ASTM D2216 and D2937)
- R-value (California Test 301)
- Soil corrosivity (California Test Methods 643, 422, and 417)
- Collapse potential (ASTM D4546)
- Grain size analysis (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)
- Consolidation (ASTM D2435)

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

4.4 Analysis and Report Preparation

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

5.0 SUBSURFACE CONDITIONS

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

5.1 Subsurface Profile

Based on the exploratory borings and laboratory test results, the subsurface materials at the site primarily consist of a mixture of sand, silt, gravel and cobbles. Few to some gravels up to 3 inches in maximum dimension and cobbles up to 6 inches in maximum dimension were observed in the borings.

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

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-11, *Logs of Borings,* in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was not encountered during the field investigation up to a depth of 50.0 feet bgs.

The GeoTracker database (SWRCB, 2022) was reviewed for groundwater data from sites within an approximately 1.0-mile radius of the proposed development. Results of that search are as follows.

- Merit Oil (Site No. # T0607100201), located approximately 5,200 feet northeast of the project site reported groundwater at a depth of 350 feet bgs in 2001.
- SBCFD Central Valley #76 (Site No. # T0607100439), located approximately 2,300 feet east of the project site reported groundwater depths ranging from 200 to 300 feet bgs in 1997.

The National Water Information System (USGS, 2022) was reviewed for current and historical groundwater data from sites within an approximately 1.0-mile radius of the proposed development and the results of that search are included below.

Table No. 2, Summary of USGS Groundwater Depth Data

Site Number	Location	Groundwater Depth Range (ft. bgs)	Date Range
340402117234601	Cedar Place south of railroad tracks; approximately 2,700 feet east of project site	240.0-288.0	1956-2001
340402117234501	Cedar Place south of railroad tracks; approximately 2,800 feet east of project site	250.0-260.81	2001-2008

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

- Well Name Chino 1006993 (Station 340672N1173970W001), located approximately 2,800 feet east of the project site, reported groundwater at a depth ranging from 101.00 to 335.00 feet bgs in 1993.
- Well Number 01S05W22M003S (Station 340672N1173967W001), located approximately 2,800 feet east of the project site, reported groundwater at a depth ranging from 127.21 to 260.81 feet bgs between 2005 and 2008.

Based on available data, the historical high groundwater level reported at wells within approximately one mile of the site was approximately 101.00 feet bgs. Current groundwater is expected to be deeper than 101.00 feet bgs. Groundwater is not expected to be encountered during excavation or construction. 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 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 soils were 0, corresponding to very low expansion potentials.

5.4 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 an 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 percent 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. 3, *Collapse Potential Values*.

Table No. 3, 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 results (collapse potential of 0.6 and 1.5 percent), slight collapse potential is anticipated at the site. Collapse potential distress is typically considered a concern when collapse potential is over 2% (LA County, 2013).

5.5 Excavatability

The subsurface materials at the project are expected to be excavatable by conventional heavy-duty earth moving equipment. However, Excavation will be difficult if high concentration of gravel or cobbles are encountered within the excavation depth.

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

5.6 Subsurface Variations

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

6.0 ENGINEERING GEOLOGY

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

6.1 Regional Geology

The project site lies within the northernmost portion of the Peninsular Ranges Geomorphic Province of California, near the boundary with the Transverse Ranges Province. The Peninsular Ranges Province is characterized by northwest trending valleys and mountain ranges, which have formed in response to the regional tectonic forces along the boundary between the Pacific and North American tectonic plates. The geologic structure is dominated by northwest trending right-lateral faults, most notably, the San Andreas Fault System. The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the southwest by the Pacific Ocean and extends southward from the Transverse Ranges into the Baja California Peninsula.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto and Elsinore faults, as well as the San Gorgonio and San Andreas fault zones (CGS, 2007), all of which have been known to be active during Quaternary time.

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

The project site is located at the extreme northeast margin of a structural block within the Peninsular Ranges known as the Perris Block. The Perris Block is a relatively stable structural block bounded by the San Jacinto fault and Ellsinore fault. The northern boundary is formed by the east-west compressional faults associated with the Transverse Ranges Physiographic Province. The southern boundary is less clearly defined.

The project site is located in an active seismic area. The active Cucamonga, San Jacinto, and San Andreas faults are located nearby. A detailed discussion on site-specific faulting and seismicity is presented in Section 7.0, Faulting and Seismicity.

6.2 Local Geology

The project site is underlain by late Holocene aged young alluvial-fan deposits (Qyf_5), consisting of unconsolidated to slightly consolidated coarse-grained sand having slightly dissected to undissected surfaces to alluvial deposited boulders (Morton and Miller, 2006).

6.3 Flooding

Review of National Flood Insurance Rate Maps indicates that the project site is within a Flood Hazard Zone "X". The Zone “X” is designated as an “Area of Minimal Flood Hazard” (FEMA, 2008).

7.0 FAULTING AND SEISMICITY

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

7.1 Faulting

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

The project site is not located within a currently mapped State of California Earthquake Fault Zone for surface fault rupture (CGS, 2007; Riverside County, 2022). Table No. 4, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 100 kilometers of the site based on the generalized coordinates (34.0694N, 117.4053W). 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 Jacinto	8.13	strike slip	241	n/a	7.88
Cucamonga	12.42	thrust	28	5.0	6.70
S. San Andreas	16.15	strike slip	548	n/a	8.18
Cleghorn	24.06	strike slip	25	3.0	6.80
San Jose	26.81	strike slip	20	0.5	6.70
Chino, alt 1	28.8	strike slip	24	1.0	6.70
Chino, alt 2	28.87	strike slip	29	1.0	6.80
North Frontal (West)	30.18	reverse	50	1.0	7.20
Elsinore	31.39	strike slip	241	n/a	7.85
Sierra Madre	31.53	reverse	57	2.0	7.20
Sierra Madre Connected	31.53	reverse	76	2.0	7.30
Clamshell-Sawpit	44.88	reverse	16	0.5	6.70
Puente Hills (Coyote Hills)	46.81	thrust	17	0.7	6.90
Raymond	55.01	strike slip	22	1.5	6.80
San Joaquin Hills	55.99	thrust	27	0.5	7.10
Puente Hills (Santa Fe Springs)	58.7	thrust	11	0.7	6.70

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Helendale-So Lockhart	59.08	strike slip	114	0.6	7.40
North Frontal (East)	63.14	thrust	27	0.5	7.00
Pinto Mtn	63.18	strike slip	74	2.5	7.30
Elysian Park (Upper)	64.18	reverse	20	1.3	6.70
Puente Hills (LA)	67.57	thrust	22	0.7	7.00
Verdugo	69.46	reverse	29	0.5	6.90
Newport Inglewood Connected alt 2	69.76	strike slip	208	1.3	7.50
Newport-Inglewood, alt 1	69.88	strike slip	65	1.0	7.20
Newport Inglewood Connected alt 1	69.88	strike slip	208	1.3	7.50
Newport-Inglewood (Offshore)	71.01	strike slip	66	1.5	7.00
Hollywood	76.39	strike slip	17	1.0	6.70
Lenwood-Lockhart-Old Woman Springs	76.77	strike slip	145	0.9	7.50
Santa Monica Connected alt 2	81.29	strike slip	93	2.4	7.40
Johnson Valley (No)	83.52	strike slip	35	0.6	6.90
San Gabriel	85.28	strike slip	71	1.0	7.30
Sierra Madre (San Fernando)	85.32	thrust	18	2.0	6.70
Palos Verdes Connected	86.31	strike slip	285	3.0	7.70
Palos Verdes	86.31	strike slip	99	3.0	7.30
Landers	90.36	strike slip	95	0.6	7.40
Burnt Mtn	91.56	strike slip	21	0.6	6.80
Santa Monica, alt 1	92.97	strike slip	14	1.0	6.60
Santa Monica Connected alt 1	92.97	strike slip	79	2.6	7.30
Eureka Peak	93.39	strike slip	19	0.6	6.70
Northridge	93.61	thrust	33	1.5	6.90
So Emerson-Copper Mtn	94.56	strike slip	54	0.6	7.10
Gravel Hills-Harper Lk	99.57	strike slip	65	0.7	7.10
Coronado Bank	99.63	strike slip	186	3.0	7.40

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

7.2 CBC Seismic Design Parameters

Seismic parameters based on the 2022 California Building Code (CBC, 2022) and ASCE 7-16 are provided in the following table. These parameters were determined using the generalized coordinates (34.0694N, 117.4053W) and the Seismic Design Maps ATC online tool.

Table No. 5, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.0694N, 117.4053W
Site Class	D
Risk Category	II
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.560g
Mapped 1-second Spectral Response Acceleration, S_1	0.604g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.560g
MCE 1-second period Spectral Response Acceleration, SM_1	1.027g
Design Spectral Response Acceleration for short period S_{DS}	1.040g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.685g
Maximum Peak Ground Acceleration, PGA_M	0.727g

7.3 Secondary Effects of Seismic Activity

In addition to ground shaking, effects of seismic activity on a project site may include surface fault rupture, soil liquefaction, landslides, lateral spreading, seismic settlement, tsunamis, seiches and earthquake-induced flooding. Results of a site-specific evaluation of each of the above secondary effects are explained below.

Surface Fault Rupture: The project site is not located within a currently designated State of California or San Bernardino County Hazard Map fault zone (CGS, 2007; San Bernardino County, 2019b). Based on review of existing geologic information, no major surface fault crosses through or extends toward the site. The potential for surface rupture resulting from the movement of active faults near the site is not known with certainty but is considered very low.

Liquefaction: Liquefaction is defined as the phenomenon in a soil mass, because of the development of excess pore pressures, soil mass suffers a substantial reduction in its shear strength. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction. Soil liquefaction occurs in submerged granular soils during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows.

- Soils must be submerged.
- Soils must be primarily granular.
- Soils must be contractive, that is, loose to medium-dense.
- Ground motion must be intense.
- Duration of shaking must be sufficient for the soils to lose shear resistance.

The project site is not located within a currently designated area susceptible to liquefaction (San Bernardino County, 2019b). The potential for liquefaction of the site is expected to be very low. Based on a site-specific settlement analysis presented in Appendix C, *Liquefaction and Settlement Analysis*, liquefaction settlement is negligible for the site.

Seismic Settlement*:* Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. Based on a site-specific settlement analysis presented in Appendix C, *Liquefaction and Settlement Analysis*, we estimate that the site will have the potential for up to approximately 1.4 inches of total dry seismic settlement.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over underlying materials which are liquefied due to ground shaking. It differs from slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography at the project site and in the immediate vicinity is very flat. Under these circumstances, the potential for lateral spreading at the subject site is considered low to moderate.

Tsunamis: Tsunamis are tidal waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location of the site, tsunamis do not pose a hazard to this site.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Review of the California Department Of Water Resources Dam Inundation Map and the San Bernardino County Hazard Map (DWR, San Bernardino County, 2019a) indicates the site is not located in any potential inundation path of any reservoir. The potential for flooding of the site due to dam failure is considered very low.

8.0 LABORATORY TEST RESULTS

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the subsurface soils. Tests results are included in Appendix A, *Field Exploration* and Appendix B, *Laboratory Testing Program*. Discussions of the various test results are presented below.

8.1 Physical Testing

- In-situ Moisture and Dry Density – *In-situ* dry density and moisture content of the subsurface alluvium soils were determined in accordance with ASTM Standard D2216 and D2937. The Dry densities of the alluvial soils at the site ranged from 83.0 to 118.0 pcf with moisture contents ranging from 1 to 17 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Expansion Index –Four representative bulk soil samples from the upper 5 feet of the site materials were tested in accordance with ASTM Standard D4829 to evaluate the expansion potential. The test results indicated an expansion index of 0, corresponding to very low expansion potential.
- R-Value – Two representative bulk samples were tested in accordance with Caltrans Test Method 301. The results of the R-value tests were 74 and 81.
- Collapse Potential – The collapse potential of three relatively undisturbed samples were tested in accordance with ASTM Standard D4546 under a vertical stress of up to 2.0 kips per square foot (ksf). The test results showed collapse potential of 0.6 to 1.5 percent, indicating none to slight collapse potential.
- Grain Size Analysis –Four representative samples were tested in accordance with ASTM Standard D6913 to determine the relative grain size distribution. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationships of two representative soil samples were performed in accordance with ASTM Standard D1557. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 118.2 and 121.0 pounds per cubic feet (pcf), with optimum moisture contents of 10.5 and 8.3 percent, respetively.
- Direct Shear –Two direct shear tests were performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The direct shear test results are presented in Drawings No. B-3 and B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.
- Consolidation Test – Two consolidation tests were conducted in accordance with ASTM Standard D2435 method. For test results, including sample density and moisture content, see Drawing Nos. B-5 and B-6, *Consolidation Test Results* in Appendix B, *Laboratory Testing Program*.

8.2 Chemical Testing - Corrosivity Evaluation

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

Table No. 6, Summary of Corrosivity Test Results

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-03	3.0-8.0	8.0	187	18	3,989
BH-07	0.0-2.0	8.1	16	17	33,110

9.0 PERCOLATION TESTING

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

Table No. 7, Estimated Infiltration Rates

Percolation Test	Approx. Depth of Boring (feet)	Predominant Soil Types (USCS)	Average Percolation Rate (inches/hour)
PT-01	5.3	Silty Sand (SM)	1.82
PT-02	10.2	Silty Sand (SM)	6.30

Based on the calculated infiltration rate during the final respective intervals in each test, a design infiltration rate of 1.82 and 6.30 (inches/hour) can be used for depth of 5 feet and 10 feet respectfully for selected percolation testing locations. Please note that infiltration rates may change if the soil type and location of the proposed system changes. If that is the case, then additional percolation testing should be performed in the required location.

10.0 EARTHWORK RECOMMENDATIONS

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

10.1 General

This section contains our general recommendations regarding earthwork and grading for the project. These recommendations are based on the results of our field exploration, laboratory tests, our experience with similar projects, and data evaluation as presented in the preceding sections. These recommendations may require modification by the geotechnical consultant based on observation of the actual field conditions during grading. Prior to the start of construction, all existing underground utilities and appurtenances should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing utilities and structure (if any).

All debris, deleterious material, artificial fill and demolished materials should be removed from the site.

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

10.2 Remedial Grading

Structures and building footings 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. 8, Overexcavation Depths

Structure	Minimum Overexcavation Depth
Building Footings	18 inches below footings bottom or 3 feet below ground surface, whichever is deeper
Slab-on-Grade	15 inches below slab bottom
Pavement	12 inches below finish grade

The overexcavation should extend to at least 2 feet beyond the footprint of the footings, slabs or building foundations and at least 1 foot beyond the edge of pavement. The overexcavation bottom should be scarified and compacted as described in Section 10.4, *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 (if any).

10.3 Engineered Fill

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

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

Based on field investigation and laboratory testing results, on-site soils may be suitable as fill materials provided proper screenings will be performed to remove large sized particles to meet above mentioned criteria.

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

10.4 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 mixed thoroughly, 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.

Fill materials should not be placed, spread or compacted during unfavorable weather conditions. When sites 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.

10.5 Shrinkage and Subsidence

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

- The shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) for the alluvial soils is estimated. An average value of 10 percent may be used 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. Ground subsidence is estimated to be approximately 0.1 foot to 0.15 foot.

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

10.6 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. A desirable drainage gradient is 1 percent for paved areas and 2 percent in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

11.0 UTILITY TRENCH BACKFILL

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

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

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

11.1 Pipe Sub-grade Preparation

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

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

11.2 Pipe Bedding

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

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

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

Bedding materials should be vibrated in-place to achieve compaction. Care should be taken to densify the bedding material below the springline of the pipe. Prior to placing the pipe bedding material, the pipe subgrade should be uniform and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material.

Migration of fines from the surrounding native and/or fill soils must be considered in selecting the gradation of any imported bedding material. We recommend that the pipe bedding material should satisfy the following criteria to protect migration of fine materials.

i. $\dfrac{D15(F)}{D85(B)} \leq 5$

ii. $\dfrac{D50(F)}{D50(B)} < 25$

iii. Bedding Materials must have less than 5 percent passing No. 200 sieve (0.0074 mm) to avoid internal movement of fines.

Where,

F	=	Bedding Material
B	=	Surrounding Native and/or Fill Soils
*D*15(F)	=	Particle size through which 15% of bedding material will pass
*D*85(B)	=	Particle size through which 85% of surrounding soil will pass
*D*50(F)	=	Particle size through which 50% of bedding material will pass
*D*50(B)	=	Particle size through which 50% of surrounding soil will pass

If the above criteria do not satisfy, commercially available geofabric used for filtration purposes (such as Mirafi 140N or equivalent) may be wrapped around the bedding material encasing the pipe to separate the bedding material from the surrounding native or fill soils.

11.3 Trench Zone Backfill

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

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.
- Trench zone backfill should be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method. At least the upper 1 foot

of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.

- Particles larger than 1 inch should not be placed within 12 inches of the pavement subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than 3 inches in the largest dimension should not be placed as trench backfill.
- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse-grained soil, and between optimum and 2 percent above optimum for fine-grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- The field density of the compacted soil should be measured by the ASTM D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) or equivalent.
- Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.
- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.
- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

12.0 DESIGN RECOMMENDATIONS

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

12.1 Shallow Foundation Design Parameters

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

Table No. 9, Recommended Foundation Parameters

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

12.2 Lateral Earth Pressures and Resistance to Lateral Loads

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

12.2.1 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 lateral earth pressures for the project site are presented in the following tables.

Table No. 10, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure[1] (psf)
Active earth conditions (wall is free to deflect at least 0.001 radian)	45
At-rest (wall is restrained)	65

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.

12.2.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 220 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.

12.2.3 Seismic Earth Pressure

The seismic force applied to structural wall is based on a horizontal seismic acceleration coefficient equal to one-third of the peak ground. An equivalent fluid seismic pressure of 24H pcf may be assumed under active loading conditions (regular triangular pressure distribution) where H is the height of the backfill behind the wall.

12.3 Slabs-on-Grade

Slabs-on-grade should be supported on properly compacted fill. Compacted fill used to support slabs-on-grade should be placed and compacted in accordance with Section 10.4 *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 (200 kcf) of the supporting materials and should be designed by a structural engineer.

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

Subgrade for slabs-on-grade should be firm and uniform. All loose or disturbed soils including under-slab utility trench backfill should be recompacted.

In hot weather, the contractor should take appropriate curing precautions after placement of concrete to minimize cracking or curling of the slabs. The potential for slab cracking may be lessened by the addition of fiber mesh to the concrete and/or control of the water/cement ratio.

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

12.4 Soil Parameters for Pipe Design

Structural design requires proper evaluation of all possible loads acting on pipe. The stresses and strains induced on buried pipe depend on many factors, including the type of soil, density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils. The recommended values of the various soil parameters for design are provided in the following table.

Table No. 11, Soil Parameters for Pipe Design

Soil Parameters	Value
Average compacted fill total unit weight (assuming 92% relative compaction), γ (pcf)	124
Angle of internal friction of soils, ϕ	28
Soil cohesion, c (psf)	35
Coefficient of friction between concrete and native soils, fs	0.35
Coefficient of friction between PVC pipe and native soils, fs	0.25
Bearing pressure against native soils (psf)	2,500
Coefficient of passive earth pressure, Kp	2.77
Coefficient of active earth pressure, Ka	0.36
Modulus of Soil Reaction E' (psi)	1,500

12.5 Settlement

The total settlement of shallow footings designed as recommended above, from static structural loads and short-term settlement of properly compacted fill is anticipated to be

0.5 inch or less. The static differential settlement can be taken as equal to one-half of the static total settlement over a lateral distance of 40 feet.

Our analysis of the potential dynamic settlement is presented in Appendix C, *Liquefaction and Settlement Analysis*. We estimate that the site has negligible potential for liquefaction induced settlement with up to 1.44 inches of dry seismic settlement. The soil profile across the site is relatively similar. So, we anticipate that the total settlement will be uniform. We recommend that the planned structure be designed in anticipation of dynamic differential settlement of 0.72 inches in 40 horizontal feet.

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

12.6 Soil Corrosivity

The results of chemical testing of a representative sample of site soils were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program*, Summary of Corrosivity Test Results, and are discussed below.

The sulfate contents of the soils tested correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentration (ACI 318-14, Table 19.3.1.1) ACI recommends a minimum compressive strength of 2,500 psi for exposure category S0 in ACI 318-14, Table 19.3.2.1.

We anticipate that concrete structures such as footings, slabs, and flatwork will be exposed to moisture from precipitation and irrigation. Based on the project location 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 minimum compressive strength of 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. 12, 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 values of the minimum electrical resistivities when saturated were 3,989 and 33,110 Ohm-cm. This indicates that the soils tested are mild to moderately corrosive for ferrous metals in contact with the soils. <u>Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for ferrous metals in contact with the site soils</u>.

12.7 Flexible Pavement Recommendations

R-values of the subgrade soils were 74 and 81. For pavement design, we have utilized an R-value of 50 and design Traffic Indices (TIs) ranging from 5 to 8.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2020), Chapter 630 with a safety factor of 0.2 for asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table below.

Table No. 13, Recommended Preliminary Flexible Pavement Sections

R-value	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
50	5	3.0	3.0	4.5
	6	3.5	3.5	5.5
	7	4.0	4.5	7.0
	8	5.0	5.0	8.5

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

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

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

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

12.8 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 50 and design Traffic Indices (TIs) ranging from 5 to 8. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 14, Recommended Preliminary Rigid Pavement Sections

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

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

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

Prior to placement of concrete, at least the upper 12.0 inches of subgrade soils below rigid pavement sections should be compacted to at least 95% relative compaction as defined by the ASTM D 1557 standard test method.

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

12.9 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 within 3 percent of optimum moisture content for coarse-grained soils and 0 to 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.

13.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

13.1 General

Prior to the start of construction, all existing underground utilities (if any) 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, pavement, or structure (if any). Recommendations pertaining to temporary excavations are presented in this section.

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

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

13.2 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. 15, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
Silty Sand (SM), Sand with Silt and Gravel (SP-SM), Sand (SP)	C	0-10	1.5:1

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

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

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.

14.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

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

15.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Miller Architectural Corporation, San Bernardino County Real Estate Services-Project Management, and their authorized agents, to assist in the development 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 are reviewed, and the recommendations of this report are modified or verified in writing. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

As the project evolves, a 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.

16.0 REFERENCES

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

AMERICAN SOCIETY OF CIVIL ENGINEERS (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, SEI/ASCE Standard No. 7-16, dated 2017.

Appendix VII, Infiltration Rate, Evaluation Protocol and Factor of Safety Recommendations, San Bernardino County, 2013.

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

CALIFORNIA DEPARTMENT OF TRANSPORTATION (Caltrans), 2020, Highway Design Manual, dated January 2020.

CALIFORNIA GEOLOGICAL SURVEY (CGS), 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Faulting Zoning Act with Index to Earthquake Fault Zone Maps, Special Publication 42, revised 2007.

CALIFORNIA STATE WATER RESOURCES CONTROL BOARD (SWRCB), 2023, GeoTracker database (http://geotracker.waterboards.ca.gov/), accessed in January 2023.

CALIFORNIA DEPARTMENT OF WATER RESOURCES (DWR), 2023, Water Data Library (http://wdl.water.ca.gov/waterdatalibrary/), accessed January 2023.

COUNTY OF LOS ANGELES, 2013, Manual for the Preparation of Geotechnical Reports, July 1, 2013.

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

DEPARTMENT OF WATER RESOURCES - DIVISION OF SAFETY OF DAMS (DSOD), 2023, California Dam Breach Inundation Maps, (https://fmds.water.ca.gov/webgis/?appid=dam_prototype_v2), accessed in January 2023.

FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA), 2008, Flood Insurance Rate Map, Riverside County, California and Incorporated Areas, Map No. 06065C0684G, effective date August 28, 2008.

MORTON, D.M. and MILLER, F.K., 2006, Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangles, California, U.S. Geological Survey Open-File Report 2006-1217, scale 1:100,000.

MOSER A. P. Buried Pipe Design, Second Edition, published by McGraw-Hill, 2001.

PUBLIC WORKS STANDARDS, INC., 2021, Standard Specifications for Public Works Construction ("Greenbook"), 2021.

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

SAN BERNARDINO COUNTY, 2013, Technical Guidance Document for Water Quality Management Plans, dated June 7, 2013.

SAN BERNARDINO COUNTY, 2019a, (San Bernardino County, 2019a) San Bernardino County General Plan Hazard Overlays, Map Sheet FH29B, scale 1:14,400, dated March 9, 2010.

SAN BERNARDINO COUNTY, 2019b, (San Bernardino County, 2019b) San Bernardino County General Plan Geologic Hazard Overlays, Map Sheet FH29C, scale 1:115,200, dated March 9, 2010.

U.S. GEOLOGICAL SURVEY (USGS), 2023, National Water Information System: Web Interface (https://maps.waterdata.usgs.gov/mapper/index.html), accessed January 2023.

U.S. GEOLOGICAL SURVEY (USGS), 2008, 2008 National Seismic Hazard Maps (https://earthquake.usgs.gov/cfusion/hazfaults_2008_search), accessed January 2023

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 percolation testing. During the site reconnaissance, the surface conditions were noted, and the borings were marked at locations approved by Mr. Brent Adams with the Miller Architectural Corporation. The approximate boring locations were established in the field using approximate distances from local streets as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Eight soil borings (BH-01 through BH-08) were drilled on December 8, 2022, to investigate the subsurface conditions. The borings were drilled to depths ranging between 5.0 and 50.0 feet below ground surface (bgs).

Two test holes (PT-01 and PT-02) were drilled on December 8, 2022, within the project site to perform water percolation testing. The borings were drilled to depths of 5.3 feet and 10.2 feet below ground surface (bgs) respectively. Details about the percolation tests are presented in Appendix D, *Percolation Testing*. Details of the exploratory borings are presented in the table (No. A-1) below.

Table No. A-1, Summary of Borings

Boring No.	Boring Depth (ft, bgs)		Groundwater Depth (ft, bgs)	Date Completed
	Proposed	Completed		
BH-01	5.0	5.0	N/E	12/8/2022
BH-02	20.0	20.0	N/E	12/8/2022
BH-03	50.0	50.0	N/E	12/8/2022
BH-04	20.0	20.0	N/E	12/8/2022
BH-05	10.0	10.0	N/E	12/8/2022
BH-06	20.0	20.0	N/E	12/8/2022
BH-07	10.0	11.5	N/E	12/8/2022
BH-08	20.0	20.5	N/E	12/8/2022
PT-01	5.0	5.3	N/E	12/8/2022
PT-02	10.0	10.2	N/E	12/8/2022

Note:
N/E = Not Encountered
For location of the borings, see Figure No. 2, *Approximate Boring and Percolation Test Locations Map*.

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

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

Standard Penetration Testing (SPT) was also performed in accordance with the ASTM Standard D1586 test using 1.4 inches inside diameter and 2.0 inches outside diameter split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

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 through BH-08) were backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight. After completion of the percolation testing, pipes were removed from PT-01 and PT-02 and the borings 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 No. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 through A-11, *Logs of Borings*.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

FIELD AND LABORATORY TESTS

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

BORING LOG SYMBOLS

SAMPLE TYPE

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

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

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

DRILLING METHOD SYMBOLS

Auger Drilling | Mud Rotary Drilling | Dynamic Cone or Hand Driven | Diamond Core

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

CONSISTENCY OF COHESIVE SOILS

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

APPARENT DENSITY OF COHESIONLESS SOILS

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

MOISTURE

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

PERCENT OF PROPORTION OF SOILS

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

SOIL PARTICLE SIZE

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

PLASTICITY OF FINE-GRAINED SOILS

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

CEMENTATION/ Induration

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

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

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Log of Boring No. BH-01

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1115 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 1.0 inches maximum dimension, trace clay, medium dense, moist, brown.	X	X	10/12/15	6	117	EI,R
		-@3.5': scattered gravel up to 3 inches maximum dimension.	X	X	10/15/18	4	95	C
5		End of boring at 5.0 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

Log of Boring No. BH-02

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1110 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 0.75 inches maximum dimension, trace clay, medium dense, moist, brown.			12/13/20			*No Recovery
5		-@4.0': scattered to few gravel up to 3 inches maximum dimension, scattered cobble up to 6 inches maximum dimension						
		-@7.0': very dense			29/35/36	3	84	CL
10								
		-@12.0': dense			19/19/22	3	112	
15								
		-@17.0': medium dense.			9/11/17	11	100	
20		End of boring at 20.0 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

Log of Boring No. BH-03

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1113 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
	ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 0.5 inches maximum dimension, trace clay, roots and rootlets, moist, brown.				EI, R, CP
5	**SAND WITH SILT AND GRAVEL (SP-SM):** fine to coarse-grained, mostly gravel up to 3 inches maximum dimension, medium dense, moist, brown.	13/12/12	2	116	CR, PA
10	**GRAVEL WITH SILT AND SAND (GP-GM):** fine to coarse-grained, gravel up to 3" maximum dimension, scattered cobble up to 5" maximum dimension, dense., brown.	17/26/25			*No Recovery
15		13/40/38	5	104	
20		6/9/16	11	105	
25	**SILTY SAND-SANDY SLIT (SM-ML):** fine to medium-grained, medium dense, moist, brown.	3/5/7	14		
30		7/11/17	17	113	
		4/7/9	14		

Log of Boring No. BH-03

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1113 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND-SANDY SLIT (SM-ML):** fine to medium-grained, medium dense, moist, brown.						
		-@38.0': dense.			9/17/27	7	117	
40								
					9/14/20	6		
45								
		-@48.0': very dense.			12/35/48	5	116	
50		End of boring at 50.0 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

Log of Boring No. BH-04

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1112 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 1 inches maximum dimension, trace clay, medium dense, moist, brown.						
5		-@4.0': few to little gravel up to 3 inches maximum dimension, scattered cobble up to 5 inches maximum dimension	■		11/13/14	4	94	C
10		-@9.0': dense.	■		22/21/18	2	118	
15		-@14.0': medium dense.	■		7/10/15	7	106	
		SAND (SP): fine to medium-grained, trace clay, medium dense, moist, brown.						
20		-@19.0': very dense.	■		42/50-6"	4		*disturbed
		End of boring at 20.0 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

NOT FOR BID

Log of Boring No. BH-05

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1115 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, trace clay, roots and rootlets, medium dense, moist, brown.	■		4/8/13	2	83	
		-@3.0': scattered to few gralel up to 3 inches maximum dimension, dense.	■		21/31/28	2	98	
5								
		-@6.0': mostly gravel up 2 inches maximum dimension.						
		-@8.0': scattered gravel up to 0.75 inches maximum dimension, medium dense.	■		8/8/9	5	103	DS
10		End of boring at 10.0 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. 22-81-206-01

Drawing No. A-6

Log of Boring No. BH-06

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1111 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, trace clay, roots and rootlets, dense, moist, brown.						
2		-@2.0': scattered gravel up to 3 inches maximum dimension	X	X	9/24/33	1	107	CL
4				X				EI, PA
5								
7		-@7.0': some gravel up to 3 inches maximum dimension, very dense.	X		21/36/28	2	115	
10								
12		-@12.0': dense.	X		21/27/31	1	117	
15								
17		-@17.0': medium dense.	X		11/8/16	6	112	
20		End of boring at 20.0 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. 22-81-206-01

Drawing No. A-7

Log of Boring No. BH-07

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1112 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, trace clay, dense, moist, dark brown.		X				EI, CR, CP
		little gravel up to 2.5 inches maximum dimension, roots and rootlets,.	X		8/26/28	2	98	
5								
		-@8.0': medium dense.	X		7/12/13	4	115	
10		-@10.0': dense.	X		11/23/34	4	135	
		End of boring at 11.5 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

NOT FOR BID

Log of Boring No. BH-08

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1108 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, dense, moist, brown.						
5		-@4.0': trace clay,, roots and rootlets	■	▨	14/18/20	4	117	CL, DS
								PA
10		-@9.0': medium dense.	■		4/6/9	6	91	
15		-@14.0': caliche.	■		5/8/12	9	83	
20			■		8/9/13	6	107	
		End of boring at 20.5 feet bgs. Groundwater not encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/8/2022.						

Log of Boring No. PT-01

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 1101 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 3 inches maximum dimension, trace clay, moist, dark brown.						PA
5		End of boring at 5.0 feet bgs. Groundwater not encountered. Borehole fitted with perforated pipe, filter and gravel for percolation testing on 12/8/2022. Upon completion of percolation testing, pipe was removed and borehole was backfilled with soil cuttings and compacted on 12/9/2022.						

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. 22-81-206-01

Drawing No. A-10

Log of Boring No. PT-02

Date Drilled: 12/8/2022 Logged by: Stephen McPherson Checked By: Hashmi Quazi

Equipment: 8" DIAMETER HOLLOW STEM AUGER Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 1103 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 Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0–10		**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" maximum dimension, trace clay, moist, dark brown. -@9.0': scattered to few gravel up to 0.75" maximum dimension.						
10		End of boring at 10.0 feet bgs. Groundwater not encountered. Borehole fitted with perforated pipe, filter and gravel for percolation testing on 12/8/2022. Upon completion of percolation testing, pipe was removed and borehole was backfilled with soil cuttings and compacted on 12/9/2022.						

Appendix B

Laboratory Testing Program

APPENDIX B

LABORATORY TESTING PROGRAM

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

In-Situ Moisture Content and Dry Density

In-situ dry density and moisture content tests were performed in accordance with ASTM Standard D2216 and D2937 on relatively undisturbed ring samples 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

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

Table No. B-1, Expansion Index Test Results

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-01	0.0-5.0	Silty Sand (SM)	0	Very Low
BH-03	0.0-3.0	Silty Sand (SM)	0	Very Low
BH-06	2.0-7.0	Silty Sand (SM)	0	Very Low
BH-07	0.0-2.0	Silty Sand (SM)	0	Very Low

R-value

Two representative bulk soil samples were tested in accordance with California Test Method CT301 for resistance value (R-value). The test provides a relative measure of soil strength for use in pavement design. The test results are presented in the following table.

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

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-01*	0.0-5.0	Silty Sand (SM)	81
BH-03*	0.0-3.0	Silty Sand (SM)	74

* Since the R-Values were slightly higher than usual range of R-Value for similar soil type, a design R-Value of 50 was used.

Soil Corrosivity

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

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

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-03	3.0-8.0	8.0	187	18	3,989
BH-07	0.0-2.0	8.1	16	17	33,110

Collapse

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

Table No. B-4, Collapse Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-02	7.0-8.5	Silty Sand (SM)	-0.6	Slight
BH-06	2.0-3.5	Silty Sand (SM)	-0.6	Slight
BH-08	4.0-5.5	Silty Sand (SM)	-1.5	Slight

Grain-Size Analyses

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

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

Boring No./Report	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-03	3.0-8.0	Sand with Silt and Gravel (SP-SM)	39.0	49.7	11.3	
BH-06	2.0-7.0	Silty Sand (SM)	13.0	54.1	32.9	
BH-08	4.0-9.0	Silty Sand (SM)	6.0	57.6	36.4	
PT-01	0.0-5.0	Silty Sand (SM)	8.0	67.9	24.1	

Maximum Dry Density and Optimum Moisture Content

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

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

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-03	0.0-3.0	Silty Sand (SM), Brown	10.5	118.2
BH-07	0.0-2.0	Silty Sand (SM), Brown	8.3	121.0

Direct Shear

One direct shear test was performed in accordance with ASTM Standard D3080 on relatively undisturbed samples in soaked moisture condition. One direct shear test was performed in accordance with ASTM Standard D3080 on remolded samples in soaked moisture condition. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawings No. B-3 and B-4, *Summary of Direct Shear Test Results,* and the following table.

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

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-05	8.0-9.5	Silty Sand (SM)	28	70
*BH-08	4.0-5.5	Silty Sand (SM)	30	160

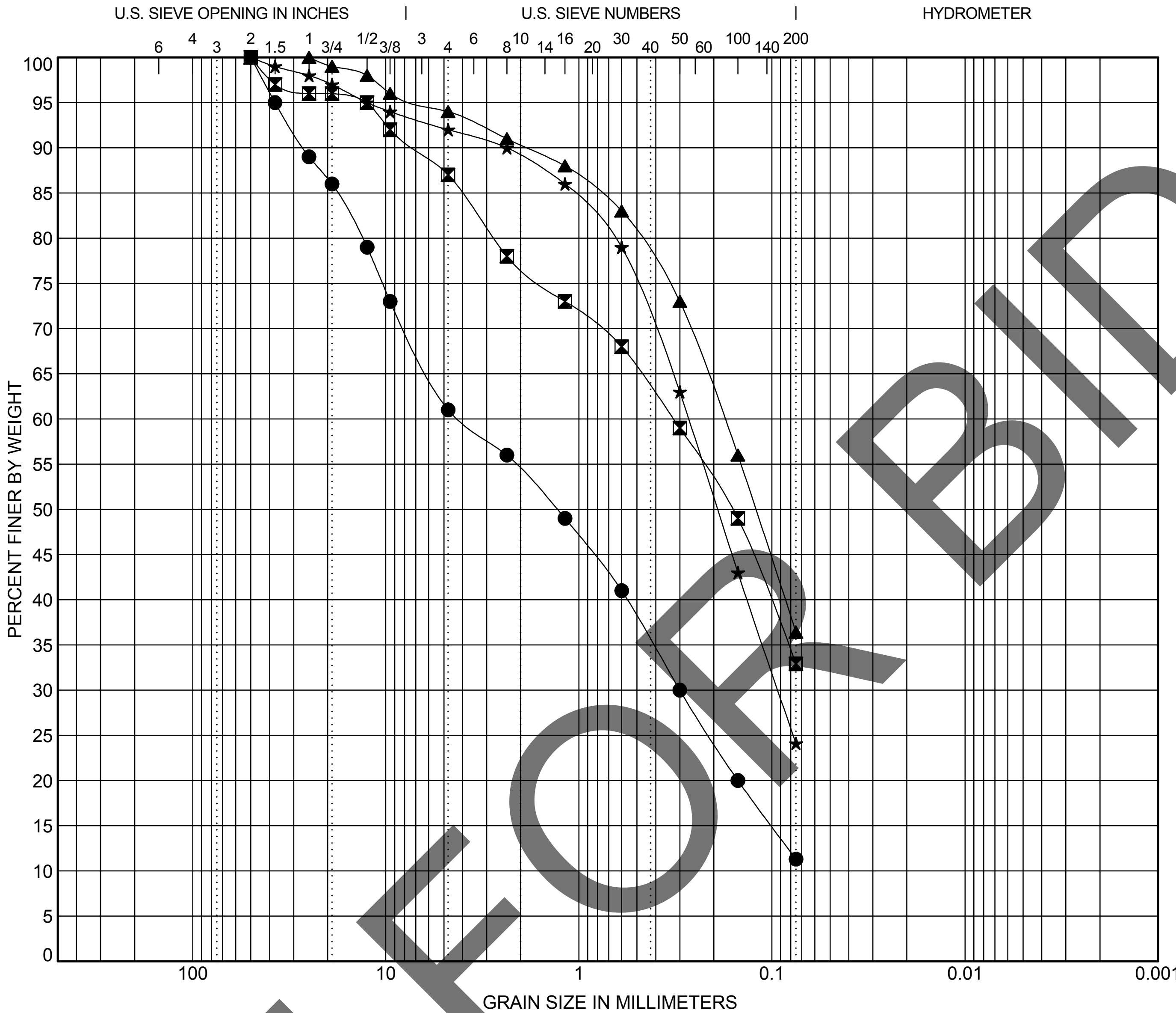
(*Remolded to 90% of laboratory maximum dry density.)

Consolidation

Two consolidation tests were conducted in accordance with ASTM Standard D2435 method. Data obtained from the test performed on one relatively undisturbed ring sample was used to evaluate the settlement characteristics of the on-site soils under load. Preparation for the test involved trimming the sample, placing it in a 1-inch-high brass ring, and loading it into the test apparatus, which contained porous stones to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. For test results, including sample density and moisture content, see Drawing Nos. B-5 and B-6, Consolidation Test Results.

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-03**	**3.0-8.0**	**SAND WITH SILT AND GRAVEL (SP-SM)**				**0.32**	**61.07**
⊠	**BH-06**	**2.0-7.0**	**SILTY SAND (SM)**					
▲	**BH-08**	**4.0-9.0**	**SILTY SAND (SM)**					
★	**PT-01**	**0.0-5.0**	**SILTY SAND (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-03**	**3.0-8.0**	**50**	**4.13**	**0.3**		**39.0**	**49.7**	**11.3**	
⊠	**BH-06**	**2.0-7.0**	**50**	**0.324**			**13.0**	**54.1**	**32.9**	
▲	**BH-08**	**4.0-9.0**	**25**	**0.177**			**6.0**	**57.6**	**36.4**	
★	**PT-01**	**0.0-5.0**	**50**	**0.27**	**0.093**		**8.0**	**67.9**	**24.1**	

GRAIN SIZE DISTRIBUTION RESULTS

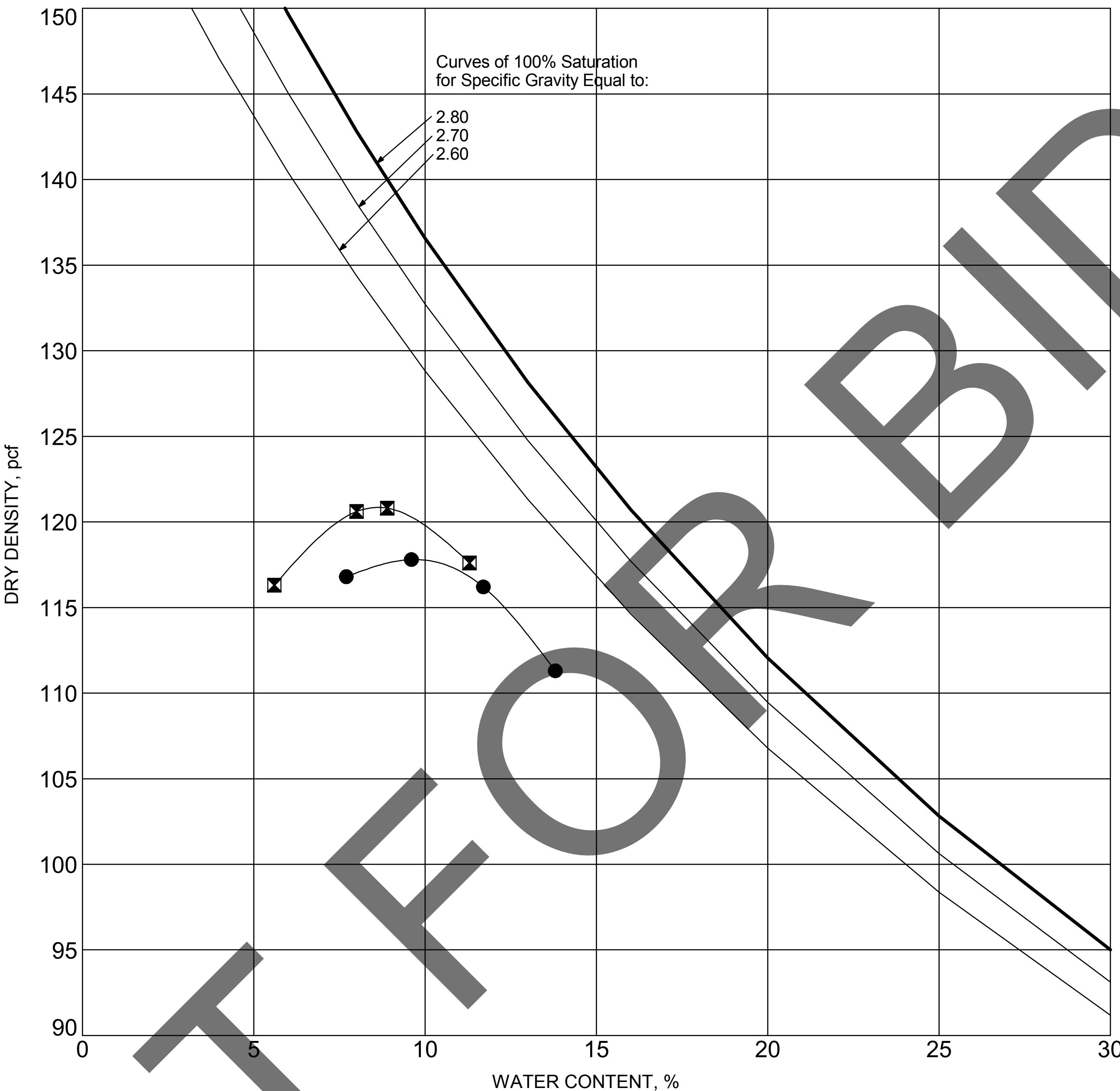
Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. **22-81-206-01**

Drawing No. **B-1**

Project ID: 22-81-206-01.GPJ; Template: GRAIN SIZE



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-03**	**0.0-3.0**	**SILTY SAND (SM), BROWN**	**D1557 Method D**	**10.5**	**118.2**
⊠	**BH-07**	**0.0-2.0**	**SILTY SAND (SM), DARK BROWN**	**D1557 Method D**	**8.3**	**121**

MOISTURE-DENSITY RELATIONSHIP RESULTS

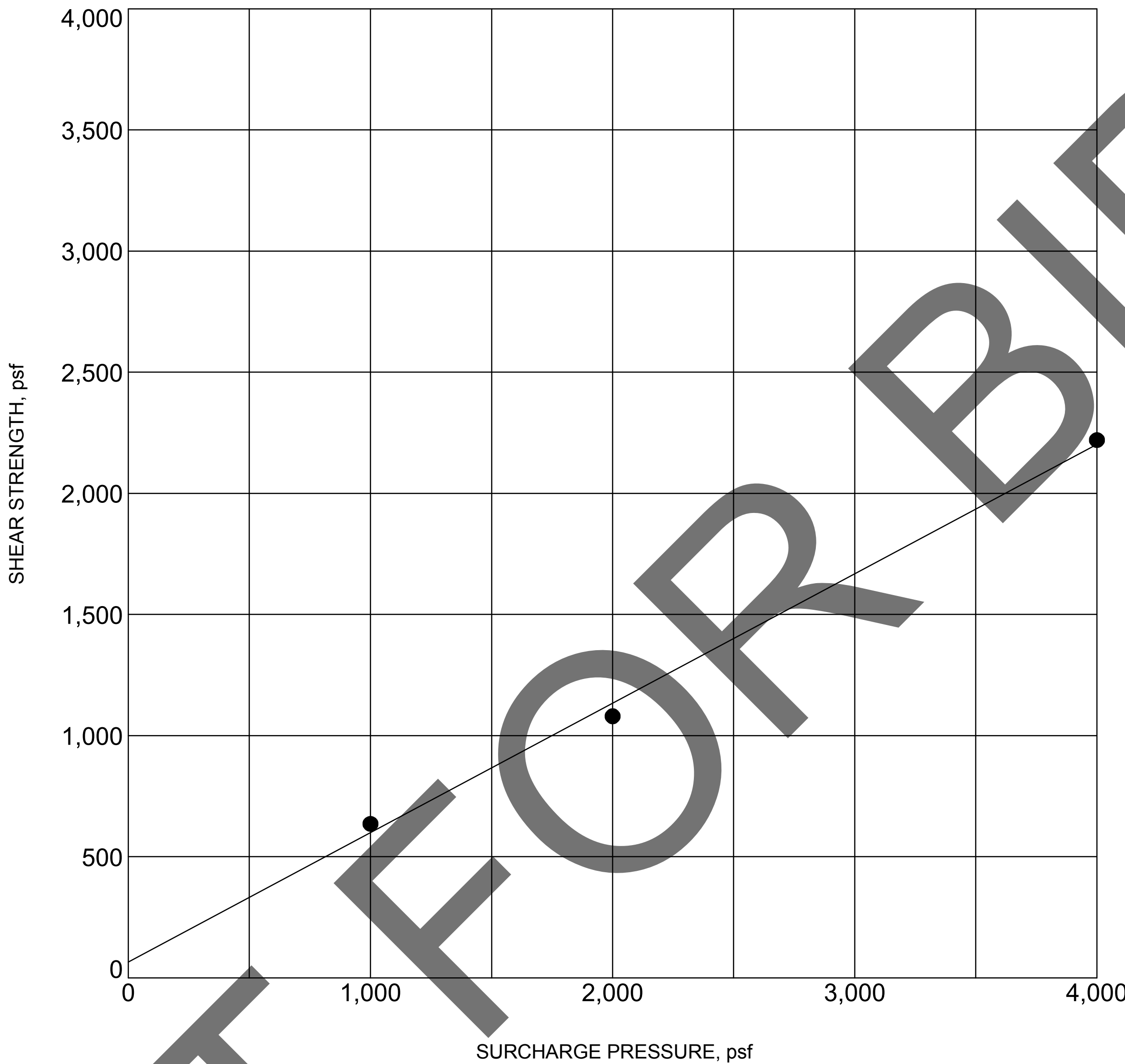
Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. **22-81-206-01**

Drawing No. **B-2**

Project ID: 22-81-206-01.GPJ; Template: COMPACTION



BORING NO. :	**BH-05**	DEPTH (ft) :	**8.0-9.5**
DESCRIPTION :	**SILTY SAND (SM)**		
COHESION (psf) :	**70**	FRICTION ANGLE (degrees):	**28**
MOISTURE CONTENT (%) :	**5.0**	DRY DENSITY (pcf) :	**103.0**

NOTE: Ultimate Strength.

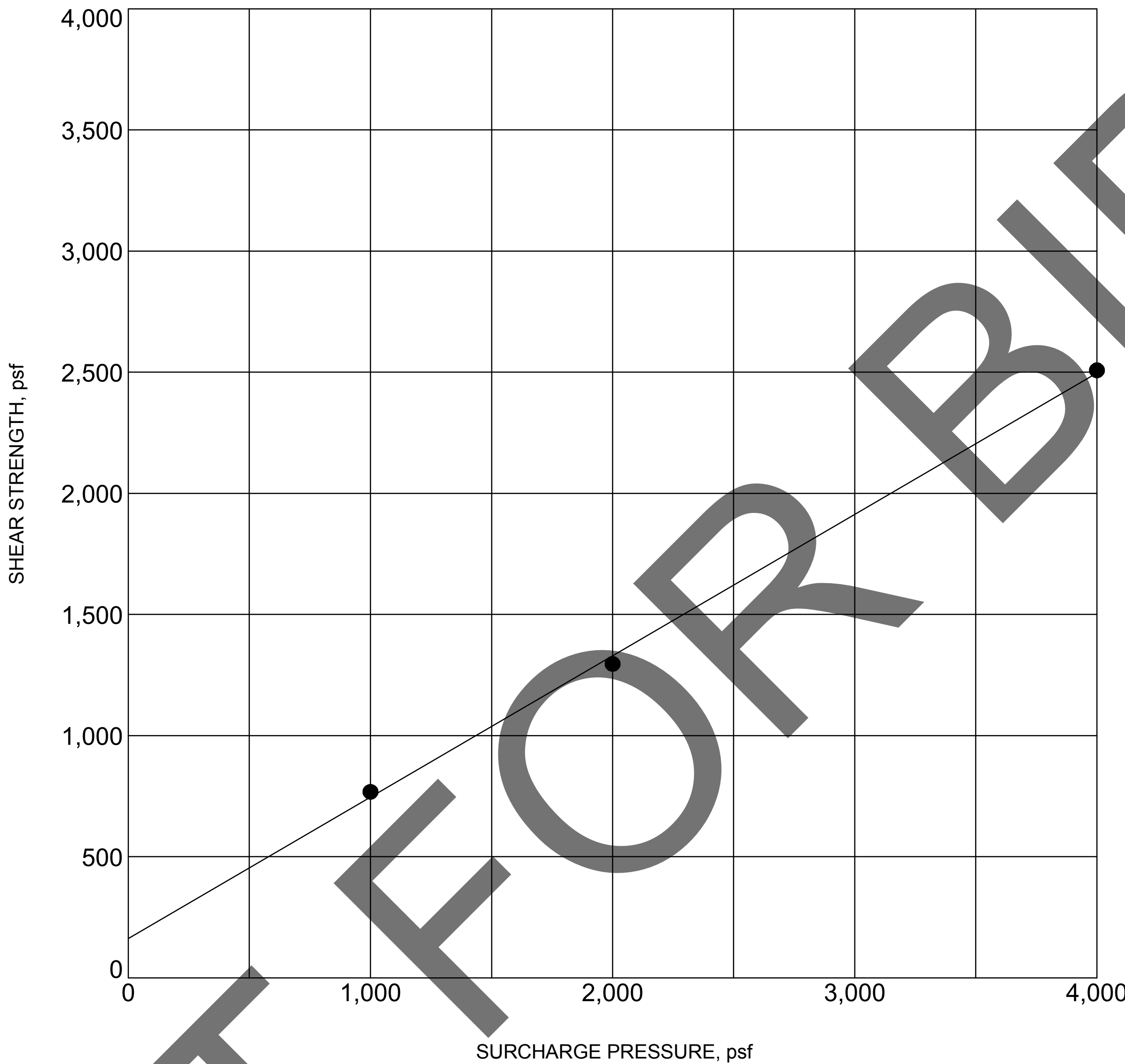
DIRECT SHEAR TEST RESULTS

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. **22-81-206-01**

Drawing No. **B-3**



BORING NO.	:	BH-08	DEPTH (ft)	:	3.0-4.5
DESCRIPTION	:	SILTY SAND (SM)*			
COHESION (psf)	:	160	FRICTION ANGLE (degrees):		30
MOISTURE CONTENT (%)	:	4.0	DRY DENSITY (pcf)	:	117.0

*Remolded to 90% of laboratory maximum dry density.

NOTE: Ultimate Strength.

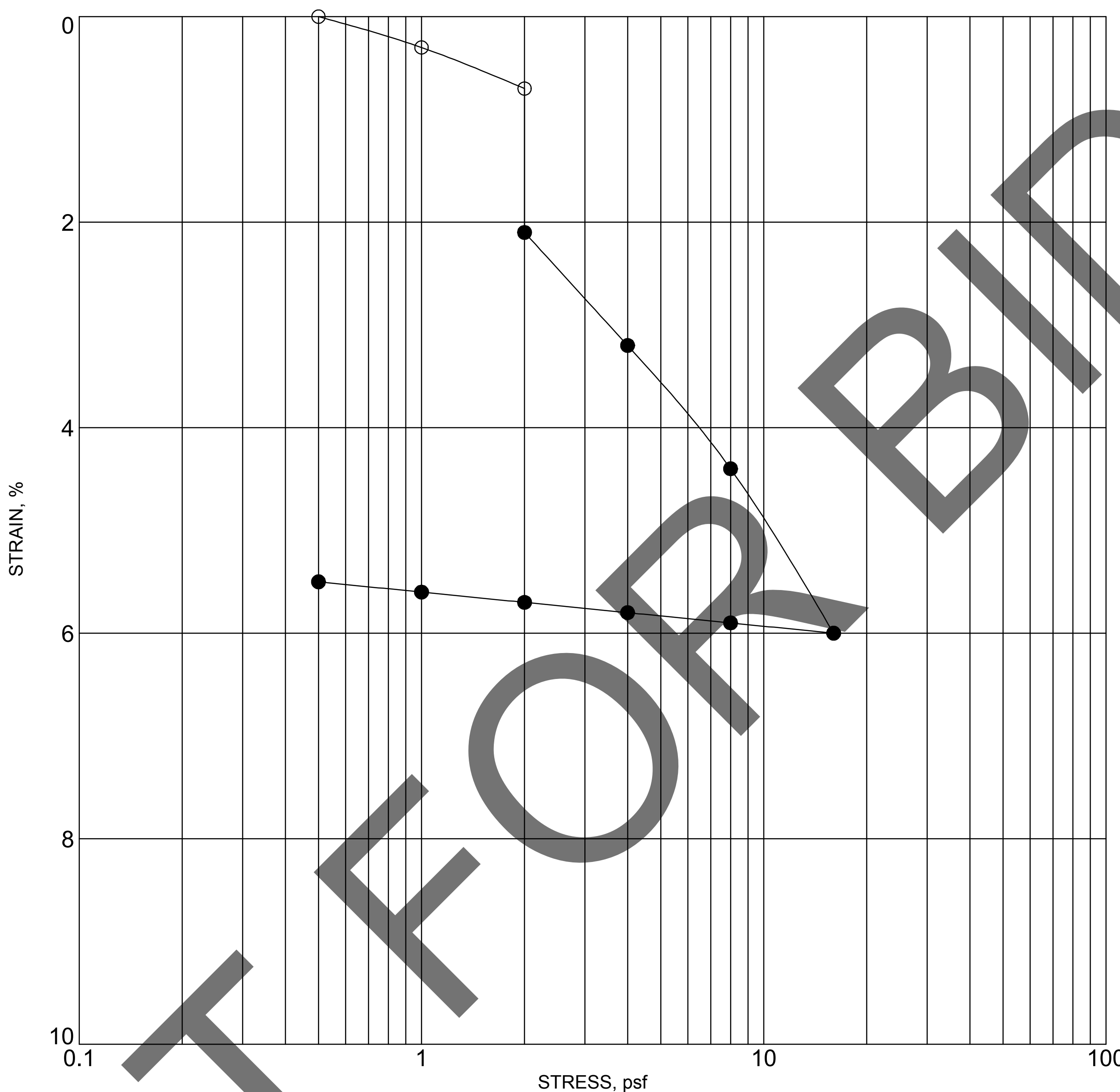
DIRECT SHEAR TEST RESULTS

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. **22-81-206-01**

Drawing No. **B-4**



BORING NO. :	BH-01	DEPTH (ft) :	3.5-5.0
DESCRIPTION :	SILTY SAND (SM)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	4	95.1	14	0.734
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

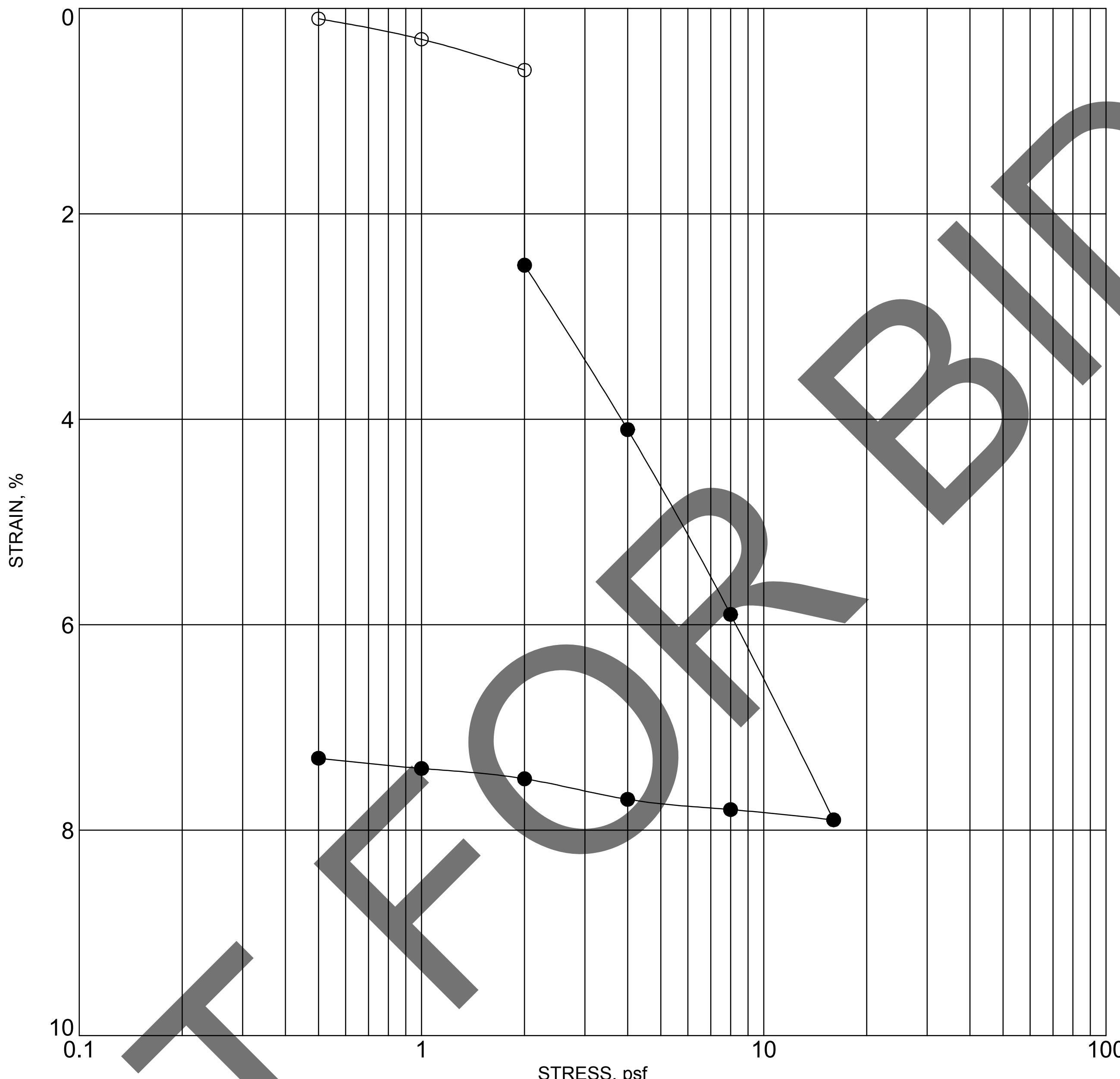
CONSOLIDATION TEST RESULTS

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. **22-81-206-01**

Drawing No. **B-5**



BORING NO. :	BH-04	DEPTH (ft) :	4.0-5.5
DESCRIPTION :	SILTY SAND (SM)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	4	94.0	7	1.459
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Bloomington Animal Shelter
18313 Valley Boulevard
Bloomington Area of San Bernardino County, California
For: Miller Architectural Corporation

Project No. **22-81-206-01**

Drawing No. **B-6**

Appendix C

Liquefaction and Settlement Analysis

APPENDIX C

LIQUEFACTION AND SETTLEMENT ANALYSIS

The subsurface data obtained from the boring BH-03 was used to evaluate the liquefaction potential and associated dry seismic settlement when subjected to ground shaking during earthquakes.

A simplified liquefaction hazard analysis was performed using the program SPTLIQ (InfraGEO Software, 2021) using the liquefaction triggering analysis method by Boulanger and Idriss (2014). A modal earthquake magnitude of M 8.1 was selected for the site based on the results of seismic disaggregation analysis using the USGS interactive online tool (https://earthquake.usgs.gov/hazards/interactive/).

A peak ground acceleration (PGA_M) of 0.727g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA was based on the 2022 CBC seismic design parameters presented in Section 7.2, *CBC Seismic Design Parameters.*

The results of our analyses are presented on Plates of Appendix C and summarized in the following table.

Table No. C-1, Estimated Dynamic Settlements

Location	Groundwater Current Depth (feet bgs)	Groundwater Historical Depth (feet bgs)	Dry Seismic Settlement (inches)	Liquefaction Induced Settlement (inches)
BH-03	> 50.0	>50.0	1.44	Negligible

Based on our analysis, we anticipate the site has the potential for up to 1.44 inches of dry seismic settlement. The differential settlement resulting from dynamic loads is anticipated to be 0.72 inches over a horizontal distance of 40 feet. The structural engineer should consider this in the design.

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

PROJECT INFORMATION	
Project Name	Bloomington Animal Shelter
Project No.	22-81-206-01
Project Location	18313 Valley Boulevard, Bloomington Area of San Bernardino County, California
Analyzed By	Sk Syfur Rahman
Reviewed By	Hashmi S. Quazi

SELECTED METHODS OF ANALYSIS	
Analysis Description	
Triggering of Liquefaction	Boulanger-Idriss (2014)
Severity of Liquefaction	LPI: Liquefaction Potential Index based on Iwasaki et al. (1978)
Seismic Compression Settlement (Dry/Unsaturated Soil)	Pradel (1998)
Liquefaction-Induced Settlement (Saturated Soil)	Ishihara and Yoshimine (1992)
Liquefaction-Induced Lateral Spreading	Zhang et al. (2004)
Residual Shear Strength of Liquefied Soil	Idriss and Boulanger (2008)

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	8.10
Peak Ground Acceleration, A_{max}	0.73 g
Factor of Safety Against Liquefaction, FS	1.20

BORING DATA AND SITE CONDITIONS			
Boring No.	BH-03		
Ground Surface Elevation	1,113.00 feet		
Proposed Grade Elevation	1,113.00 feet		
GWL Depth Measured During Test	50.00 feet		
GWL Depth Used in Design	50.00 feet		
Borehole Diameter	8.00 inches		
Hammer Weight	140.00 pounds		
Hammer Drop	30.00 inches		
Hammer Energy Efficiency Ratio, ER (%)	80.00 %		
Hammer Distance to Ground Surface	5.00 feet		
Topographic Site Condition:	TSC3 (Level Ground with Nearby Free Face)		
- Ground Slope, S (%)	<<= Leave this blank		
- Free Face Distance to Slope Height Ratio, (L/H)	5.00 <<= Enter (L/H)	Enter H =>>	15.00 feet

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Screening *Susceptible Soil? (Y, N)*	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	2.50	SM	Y	118.0	MCal	24.00	11.00
2.50	5.00	SP-SM	Y	118.0	MCal	24.00	11.00
5.00	10.00	SP-SM	Y	118.0	MCal	51.00	11.00
10.00	15.00	SP-SM	Y	109.0	MCal	78.00	10.00
15.00	20.00	SP-SM	Y	117.0	MCal	25.00	10.00
20.00	25.00	SP-SM	Y	117.0	SPT1	12.00	10.00
25.00	30.00	SM	Y	132.0	MCal	28.00	10.00
30.00	35.00	SM	N	132.0	SPT1	16.00	10.00
35.00	40.00	SM	N	125.0	MCal	44.00	10.00
40.00	45.00	SM	N	125.0	SPT1	34.00	10.00
45.00	50.00	SM	N	122.0	MCal	83.00	10.00

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

PROJECT INFORMATION

Project Name	Bloomington Animal Shelter
Project No.	22-81-206-01
Project Location	18313 Valley Boulevard, Bloomington Area of San Bernardino County, C
Analyzed By	Sk Syfur Rahman
Reviewed By	Hashmi S. Quazi

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	8.10
Peak Ground Acceleration, A_{max}	0.73 g
Factor of Safety Against Liquefaction, FS	1.20

BORING DATA AND SITE CONDITIONS

Boring No.	BH-03
Ground Surface Elevation	1,113.00 feet
Proposed Grade Elevation	1,113.00 feet
GWL Depth Measured During Test	50.00 feet
GWL Depth Used in Design	50.00 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet
Topographic Site Condition:	TSC3 (Level Ground with Nearby Free Face)
- Ground Slope, S	N/A
- Free Face (L/H) Ratio	5.00 H = 15 feet

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils: 0.00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 0.00 *** (Very low risk, with no surface manifestation of liquefaction)

Seismic Ground Settlements:	Analysis Method	Upper 30 feet	Upper 50 feet	Upper 65 feet	
Seismic Compression Settlement:	Pradel (1998)	1.44 inches	1.44 inches	1.44 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0.00 inches	0.00 inches	0.00 inches	(Saturated Soils)
Total Seismic Settlement:		1.44 inches	1.44 inches	1.44 inches	

Seismic Lateral Displacements:	Analysis Method	Upper 30 feet	Upper 50 feet	Upper 65 feet	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0.63 inches	0.63 inches	0.63 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al. (2004)	0.00 inches	0.00 inches	0.00 inches	(After Ground Shaking)

NOTES AND REFERENCES

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

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

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

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

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

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

INPUT SOIL PROFILE DATA

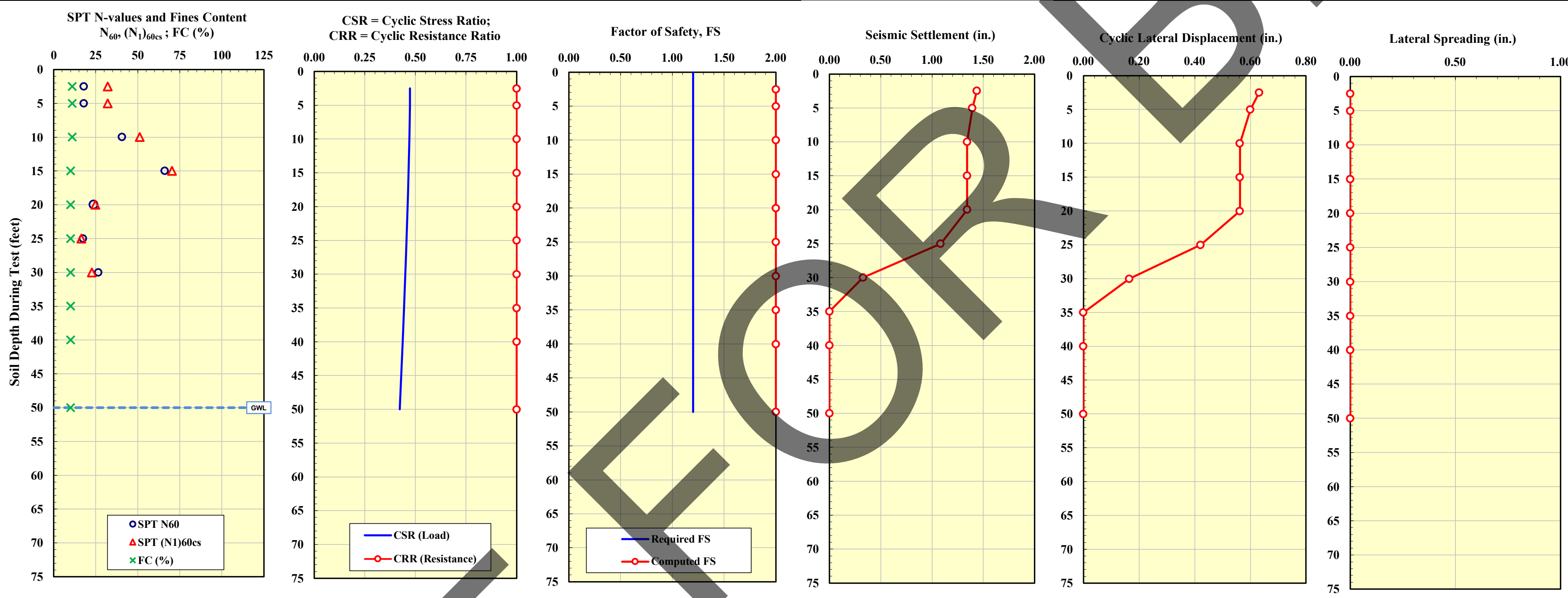
Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field SPT Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	2.50	SM	Y	118.00	MCal	24.00	11.00
2.50	5.00	SP-SM	Y	118.00	MCal	24.00	11.00
5.00	10.00	SP-SM	Y	118.00	MCal	51.00	11.00
10.00	15.00	SP-SM	Y	109.00	MCal	78.00	10.00
15.00	20.00	SP-SM	Y	117.00	MCal	25.00	10.00
20.00	25.00	SP-SM	Y	117.00	SPT1	12.00	10.00
25.00	30.00	SM	Y	132.00	MCal	28.00	10.00
30.00	35.00	SM	N	132.00	SPT1	16.00	10.00
35.00	40.00	SM	N	125.00	MCal	44.00	10.00
40.00	45.00	SM	N	125.00	SPT1	34.00	10.00
45.00	50.00	SM	N	122.00	MCal	83.00	10.00

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

Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. for Vert. Stress C_N	SPT Corr. for Hammer Energy C_E	SPT Corr. for Borehole Size C_B	SPT Corr. for Rod Length C_R	SPT Corr. for Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
147.50	147.50	1.700	1.333	1.150	0.750	0.650	17.9	30.5	32.1	1.000	1.100	0.473						1.44	0.63	0.00
442.50	442.50	1.700	1.333	1.150	0.750	0.650	17.9	30.5	32.1	1.000	1.100	0.473						1.39	0.60	0.00
885.00	885.00	1.219	1.333	1.150	0.800	0.650	40.7	49.6	51.2	0.995	1.100	0.470						1.34	0.56	0.00
1,452.50	1,452.50	1.047	1.333	1.150	0.850	0.650	66.1	69.2	70.4	0.986	1.096	0.466						1.34	0.56	0.00
2,017.50	2,017.50	0.996	1.333	1.150	0.950	0.650	23.7	23.6	24.7	0.976	0.999	0.461						1.34	0.56	0.00
2,602.50	2,602.50	0.881	1.333	1.150	0.950	1.000	17.5	15.4	16.5	0.965	0.970	0.456						1.08	0.42	0.00
3,225.00	3,225.00	0.815	1.333	1.150	0.950	0.650	26.5	21.6	22.8	0.952	0.932	0.450						0.33	0.17	0.00
3,885.00	3,885.00									0.939		0.444						0.00	0.00	0.00
4,527.50	4,527.50									0.925		0.437						0.00	0.00	0.00
5,152.50	5,152.50									0.909		0.430						0.00	0.00	0.00
5,770.00	5,770.00									0.894		0.422						0.00	0.00	0.00

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

PROJECT INFORMATION	
Project Name	Bloomington Animal Shelter
Project No.	22-81-206-01
Project Location	18313 Valley Boulevard, Bloomington Area of San Bernardino County, California
Analyzed By	Sk Syfur Rahman
Reviewed By	Hashmi S. Quazi



Analysis Methods Used ==>>

	Liquefaction Triggering:		Seismic Settlements:	Cyclic Lateral Displacements:	Lateral Spreading:
	Boulanger-Idriss (2014)	Above GWL:	Pradel (1998)	Pradel (1998)	Zhang et al. (2004)
		Below GWL:	Ishihara and Yoshimine (1992)	Tokimatsu and Asaka (1998)	

REFERENCES:

1. Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.
2. Bray, J.D., and Sancio, R.B. (2006). "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotech. and Geoenv. Engineering, ASCE 132 (9), 1165-1177.
3. Cetin, K.O. and Seed, R.B., et al. (2004), "Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential," Journal of Geotech. and Geoenv. Engineering, ASCE 130 (12), 1314-1340.
4. Idriss, I.M. and Boulanger, R.W. (2008), "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute (EERI), Monograph MNO-12.
5. Ishihara, K. and Yoshimine, M. (1992), "Evaluation of settlements in sand deposits following liquefaction during earthquakes," Soils and Foundations, Japanese Geotechnical Society, 32 (1), 173-188.
6. Iwasaki, T., et al. (1978), "A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan," Proceedings Of 3rd International Conference of Microzonation, San Francisco, 885-896.
7. Olson, S.M. and Johnson, C.I. (2008), "Analyzing Liquefaction-Induced Lateral Spreads Using Strength Ratios," Journal of Geotech. and Geoenv. Engineering, ASCE 134 (8), 1035-1049.
8. Pradel, D. (1998), "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils," Journal of Geotechnical Engineering, ASCE 124 (4), pp. 364-368.
9. Seed, R.B. and Harder, L.F. (1990), "SPT-based analysis of cyclic pore pressure generation and undrained residual strength, Proceedings Of Seed Memorial Symposium, Vancouver, B.C., 351-376.
10. Tokimatsu, K. and Seed, H.B. (1987), "Evaluation of settlements in sands due to earthquake shaking," Journal of Geotechnical Engineering, ASCE 113 (GT8), 861-878.
11. Tokimatsu, K. and Asaka, Y. (1998), "Effects of liquefaction-induced ground displacementson pile performance in the 1995 Hyogoken-Nambu Earthquake," Soils and Foundations, Special Issue, Japan Geotechnical Society, 163-177.
12. Toprak, S. and Holzer, T.L. (2003), "Liquefaction Potential Index: Field Assessment," Journal of Geotechnical and Geoenviromental Engineering, ASCE 129 (4), 315-322.
13. Youd, T.L, Idriss, I.M., et al. (2001), "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF Workshops", Journal of Geotech. and Geoenv. Engineering, ASCE 127 (10), 817-833.
14. Zhang, G, Robertson, P.K. and Brachman, R.W.I. (2004), "Estimating liquefaction-induced lateral displacement using the standard penetration test or cone penetration test," Journal of Geotech. and Geoenv. Engineering, ASCE 130 (8), 861-871.

Appendix D

Percolation Testing

APPENDIX D

PERCOLATION TESTING

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

Upon completion of drilling the test holes, 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. Percolation testing was conducted the day following presoaking. More than 6 inches of water seeped away from the test holes in less than 25 minutes for 2 consecutive measurements, meeting the criteria for testing as “sandy soil”. During testing, the water level and total depth of the test hole were measured from the top of the pipe every 10 minutes for one hour. Following the completion of percolation testing, the pipe was removed from each test hole and the percolation test hole was backfilled with cutting soils 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 San Bernardino County guidelines. A factor of safety of 2 was applied to the measured infiltration rates to account for subsurface variations, uncertainty in the test method, and future siltation. The infiltration structure designer should determine whether additional design-related safety factors are appropriate.

The measured percolation test data, calculations and estimated infiltration rates are shown on Plates No. 1 and 4. The estimated and design infiltration rates at the test holes are presented in the following table.

Table D-1, Estimated Infiltration Rates

Percolation Test	Approx. Depth of Boring* (feet)	Predominant Soil Types (USCS)	Average Infiltration Rate (inches/hour) (FOS 2)
PT-01	5.3	Silty Sand (SM)	1.82
PT-02	10.2	Silty Sand (SM)	6.30

Based on the calculated infiltration rate during the final respective intervals in each test, a design infiltration rate of 1.82 and 6.30 (inches/hour) can be used for depth of 5 feet and 10 feet respectfully for selected percolation testing locations. Please note that infiltration rates may change if the soil type and location of the proposed system changes. If that is the case, then additional percolation testing should be performed in the required location.

Estimated Infiltration Rate from Percolation Test Data, PT-01

Project Name	Bloomington Animal Shelter
Project Number	22-81-206-01
Test Number	PT-01
Test Location	Southeast of site
Personnel	Stephen McPherson
Presoak Date	12/8/2022
Test Date	12/9/2022

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	62.5
Inside Diameter of Pipe, I (inches)	2.88
Outside Diameter of Pipe, O (inches)	3.13
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	25.00	11.40	40.80	25.00	51.10	21.70	29.40	36.40	3.68		1.84
2	25.00	5.88	37.44	50.00	56.62	25.06	31.56	40.84	3.54		1.77
3	10.00	8.40	24.72	60.00	54.10	37.78	16.32	45.94	4.09		2.04
4	10.00	8.40	24.00	70.00	54.10	38.50	15.60	46.30	3.88		1.94
5	10.00	8.40	23.64	80.00	54.10	38.86	15.24	46.48	3.77		1.89
6	10.00	8.40	23.40	90.00	54.10	39.10	15.00	46.60	3.70		1.85
7	10.00	8.40	23.16	100.00	54.10	39.34	14.76	46.72	3.64		1.82
8	10.00	8.40	23.16	110.00	54.10	39.34	14.76	46.72	3.64		1.82

Recommended Design Infiltration Rate (inches/hr)	**1.82**

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

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

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

Plate No.
1

Infiltration Rate versus Time, PT-01

Project Name	Bloomington Animal Shelter
Project Number	22-81-206-01
Test Number	PT-01
Test Location	Southeast of site
Personnel	Stephen McPherson
Presoak Date	12/8/2022
Test Date	12/9/2022

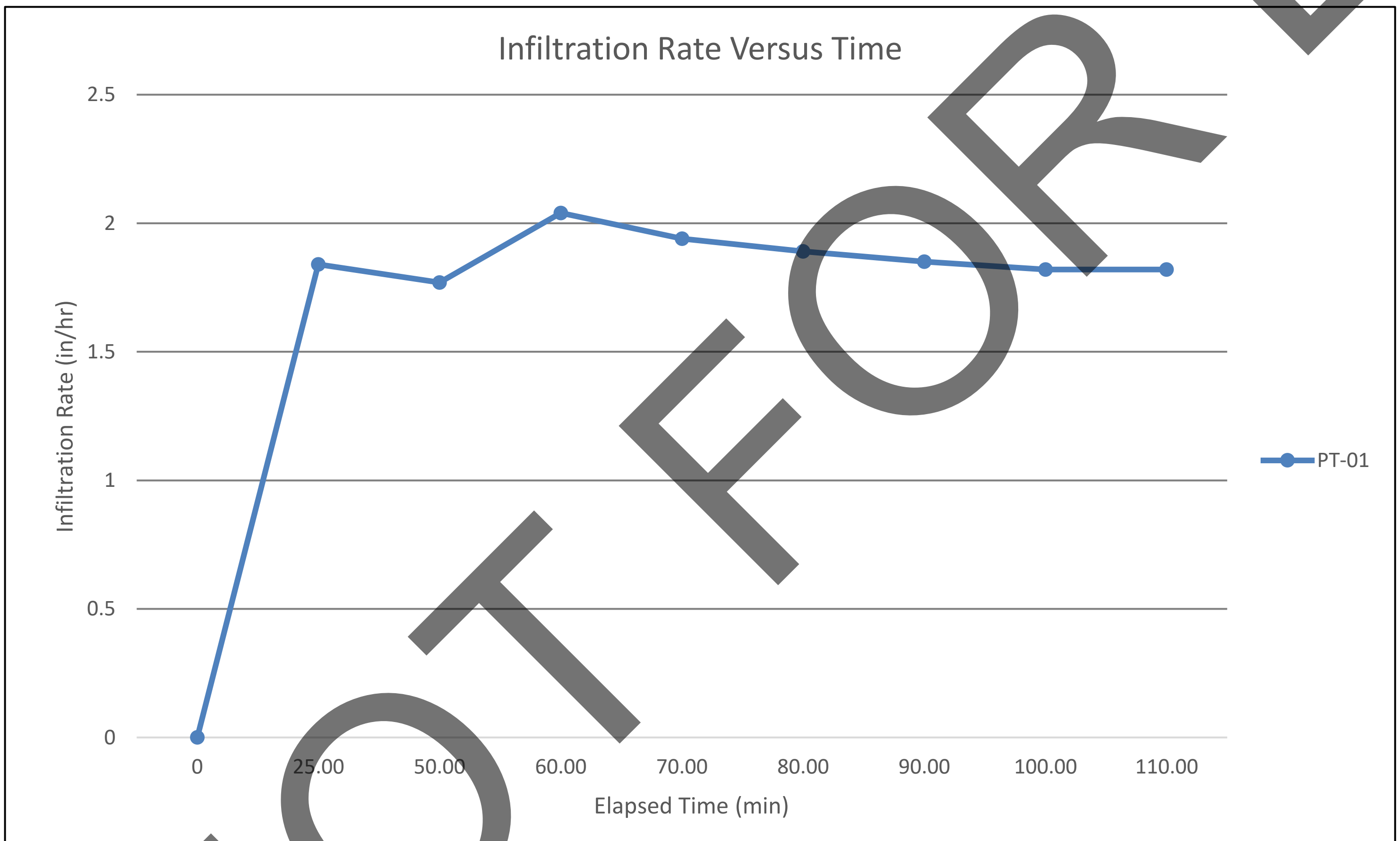


Plate No.
2

Estimated Infiltration Rate from Percolation Test Data, PT-01

Project Name	Bloomington Animal Shelter
Project Number	22-81-206-01
Test Number	PT-02
Test Location	Southwest of site
Personnel	Stephen McPherson
Presoak Date	12/8/2022
Test Date	12/9/2022

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	122.75
Inside Diameter of Pipe, I (inches)	2.88
Outside Diameter of Pipe, O (inches)	3.13
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	25.00	12.00	120.60	25.00	110.75	2.15	108.60	56.45	8.92		4.46
2	25.00	14.76	118.44	50.00	107.99	4.31	103.68	56.15	8.56		4.28
3	10.00	15.60	97.80	60.00	107.15	24.95	82.20	66.05	14.50		7.25
4	10.00	13.92	94.92	70.00	108.83	27.83	81.00	68.33	13.82		6.91
5	10.00	18.00	94.20	80.00	104.75	28.55	76.20	66.65	13.32		6.66
6	10.00	12.60	91.68	90.00	110.15	31.07	79.08	70.61	13.07		6.53
7	10.00	16.80	91.68	100.00	105.95	31.07	74.88	68.51	12.74		6.37
8	10.00	14.40	90.36	110.00	108.35	32.39	75.96	70.37	12.60		6.30

Recommended Design Infiltration Rate (inches/hr)	**6.30**

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

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

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

Plate No.
3

Infiltration Rate versus Time, PT-01

Project Name	Bloomington Animal Shelter
Project Number	22-81-206-01
Test Number	PT-02
Test Location	Southwest of site
Personnel	Stephen McPherson
Presoak Date	12/8/2022
Test Date	12/9/2022

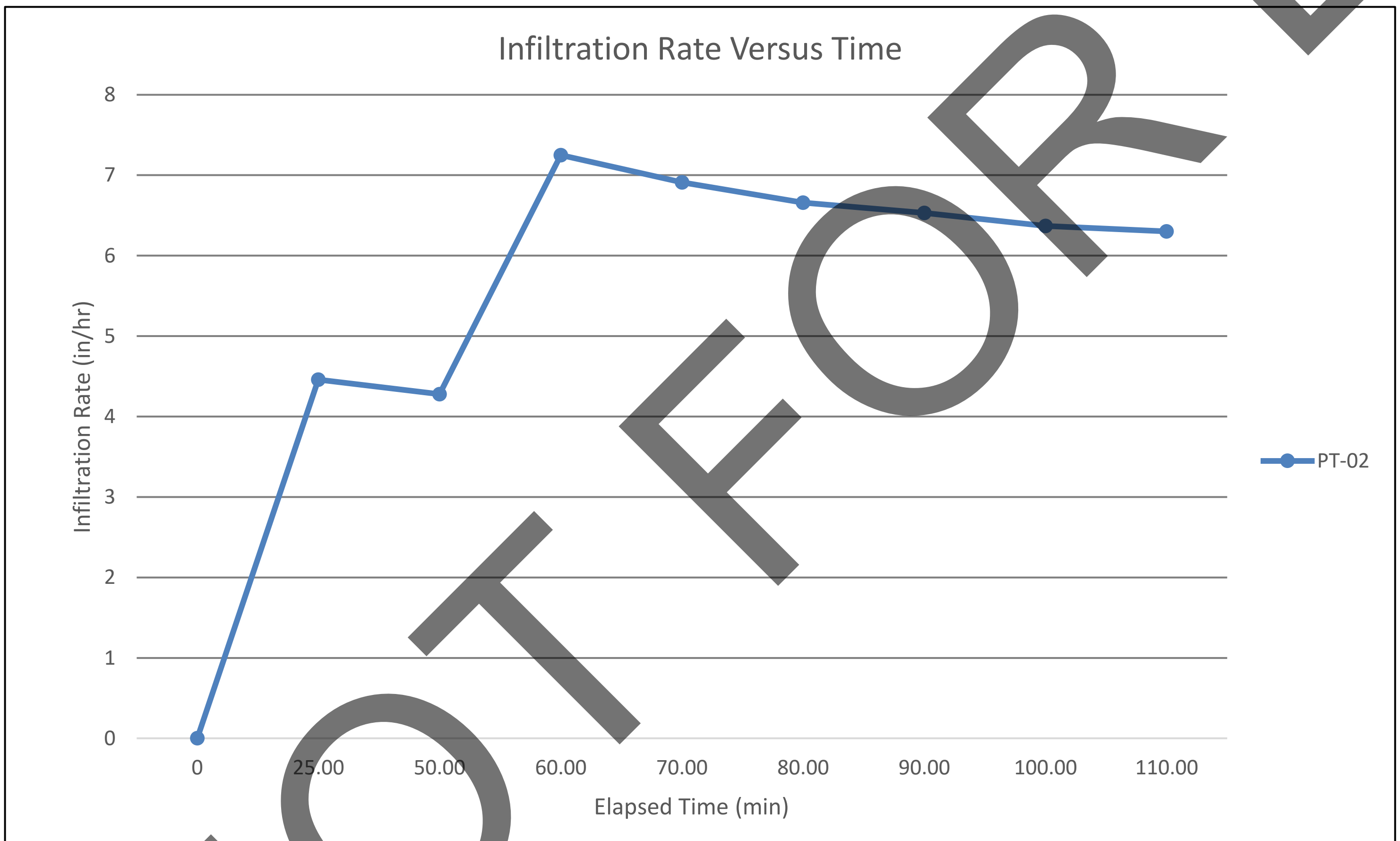


Plate No.
4