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SECTION H

GEOTECHNICAL REPORT

GLEN HELEN REGIONAL PARK MONUMENT PROJECT

FOR

GLEN HELEN REGIONAL PARK
SAN BERNARDINO, CALIFORNIA

PROJECT NO.'s: 1012115

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

GEOTECHNICAL INVESTIGATION REPORT

GLEN HELEN REGIONAL PARK MONUMENT PROJECT

2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California

CONVERSE PROJECT NO. 23-81-127-01

Prepared For:
SAN BERNARDINO CO. DEPARTMENT OF PUBLIC WORKS, SPECIAL DISTRICTS
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March 7, 2023

Converse Consultants

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

March 7, 2023

Mr. Charles Hernandez
Project Manager
San Bernardino County Department of Public Works
Special Districts
222 East Hospitality Lane, Second Floor
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Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Glen Helen Regional Park Monument
2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California
Converse Project No. 23-81-127-01

Dear Mr. Hernandez:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design of the monument replacement at the Glen Helen Park located in the City of San Bernardino, San Bernardino County, California. This report was prepared in accordance with our proposal dated February 1, 2023, and your Acceptance of Agreement Authorization to Proceed dated February 10, 2023, and Purchase Order (PO: 4100289178) dated March 03, 2023.

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

We appreciate the opportunity to be of service to San Bernardino County Department of Public Works - Special Districts. Should you have any questions, please do not hesitate to contact us at 909-474-2847.

CONVERSE CONSULTANTS

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

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

PROFESSIONAL CERTIFICATION

This report has been prepared by the following professionals 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

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APPENDICES

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for the Glen Helen Regional Park Monument Replacement Project (hereinafter "The Project") located in the City of San Bernardino, San Bernardino County, California. The project location is shown in Figure No. 1, *Approximate Project Location Map*.

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

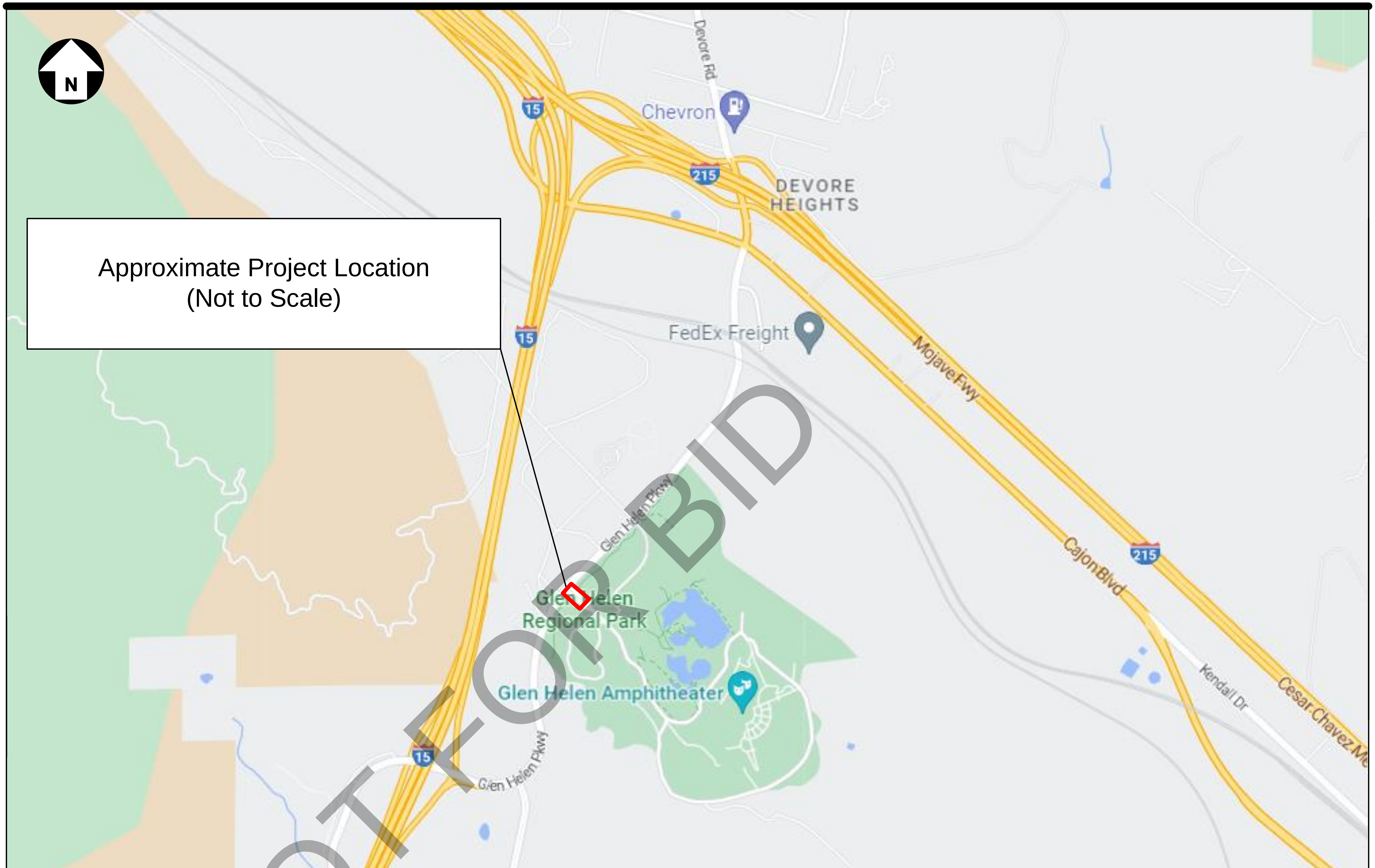
Converse also prepared the Geotechnical Investigation Report, Glen Helen Bridge Replacement Project, 2555 Glen Helen Parkway, City of San Bernardino, San Bernardino County, California, Converse Project No. 22-81-129-01, dated June 6, 2022. The findings of that investigation have been used for preparation of this report. This report is prepared for the project described herein and is intended for use solely by San Bernardino County Department of Public Works – Special Districts and their authorized agents for design purposes. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

2.0 SITE/PROJECT DESCRIPTION

The Glen Helen Regional Park Monument Project will consist of the design and construction of a gate entry monument sign to be placed in an existing planter with the existing wood constructed sign being removed. Additionally, the project will include the placement of overhead entry monument signage near the opening of the gate 1 entrance at the Glen Helen Regional Park located at 2555 Glen Helen Parkway, City of San Bernadino, San Bernardino, California.

The site is bounded to the northwest by Glen Helen Parkway, to the northeast and southwest by park landscape, and to the southeast by yellow park entry gates as well as a planter containing mailboxes. The approximate elevation of the site is 2039 feet above mean sea level (amsl). The coordinates for the project site are approximately 34.2085° north latitude and 117.4093° west longitude.

Currently, the site has existing landscape, pavement, and underground utilities. Two travel lanes run on both northwest and southeast bound directions separated by a planter containing the existing wood constructed sign. Pedestrian walkways, overhead utilities, and vegetation were observed near the street. The vegetation consists of a light to moderate growth of regularly maintained bushes and trees about the project site. To the



Project: Glen Helen Regional Park Monument
Location: 2555 Glen Helen Parkway
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For: San Bernardino County Department of Public Works - Special Districts

Approximate Project Location Map

Project No.
23-81-127-01

Converse Consultants

Figure No.
1

east of the site exists a gravel lot. *Photographs Nos. 1 and 2* depict the present the site conditions.

Photograph No. 1: Current planter containing the existing wood constructed sign, seen from Glen Helen Parkway.

Photograph No. 2: Present site conditions, facing south.

3.0 SCOPE OF WORK

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

3.1 Project Set-up

The project set-up consisted of the following tasks.

- Conducted a field reconnaissance with Charles Hernandez of the San Bernardino County Department of Public Works - Special Districts and marked the boring locations such that the drill rig access to all locations was 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.

3.2 Subsurface Exploration

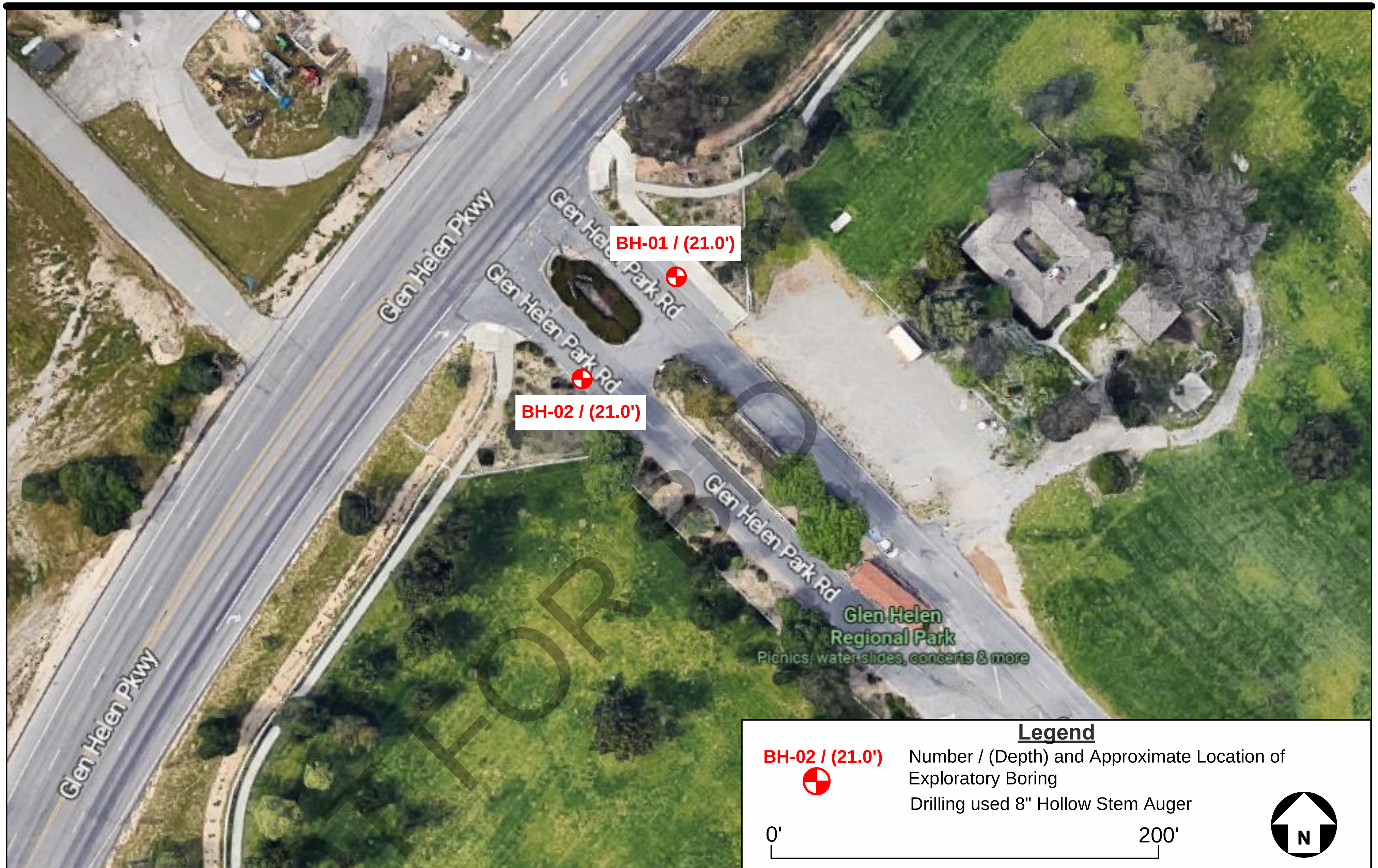
Two exploratory borings (BH-01 and BH-02) were drilled on February 23, 2023, to investigate the subsurface conditions. The borings were drilled with 8-inch diameter hollow-stem augers to depths of approximately 21.0 feet below ground surface (bgs).

The approximate boring locations are indicated in Figure No. 2, *Approximate Boring Locations Map*. For a description of the field exploration and sampling program, see Appendix A, *Field Exploration*.

3.3 Laboratory Testing

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

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



Project: Glen Helen Regional Park Monument

Location: 2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California

For: San Bernardino County Department of Public Works - Special Districts

Approximate Boring Locations Map

Project No. 23-81-127-01

Converse Consultants

Figure No. 2

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.*

3.4 Analysis and Report Preparation

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

4.0 SUBSURFACE CONDITIONS

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

4.1 Existing Pavement Sections

The measured asphalt concrete pavement thickness at each boring location is presented in the following table.

Table No. 1, Existing Pavement Sections

Boring No.	Cement Concrete Thickness (in.)	Aggregate Base Thickness (in.)
BH-01	4.0	4.0
BH-02	4.0	3.0

4.2 Subsurface Profile

Based on the exploratory borings and laboratory test results, the subsurface soil at the site consists primarily mixture of gravel, sand, and silt. Trace to little caliche, trace clays and gravels up to 2.5 inches in maximum dimension were also encountered in the borings.

Discernible fill soils were not identified in our subsurface exploration; however, the site may have been previously graded for the existing pavements 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 and A-3, *Logs of Borings*, in Appendix A, *Field Exploration*.

4.3 Groundwater

Groundwater was encountered during the investigation at the depth of approximately 21.0 feet below ground surface (bgs). Regional conditions were reviewed to estimate expected groundwater depths in the vicinity of the proposed project.
Regional groundwater data from the GeoTracker database (SWRCB, 2022) for locations within a one-mile radius of the project site was reviewed to evaluate the current and historical groundwater levels. The results of that search are listed below.

- GLEN HELEN REGIONAL PARK (Site No. T0607100575) located approximately 1,400 feet northwest of the project area reported groundwater at depths ranging from 54.2 to 68.5 feet bgs between 9/2010 and 3/2010.

Data in the following table was found on the National Water Information System (USGS, 2022).

Table No. 2, Summary of USGS Groundwater Depth Data

Alignment No.	Location	Groundwater Depth Range (ft. bgs)	Date Range
341249117241801	Approximately 2,550 Feet North of the Project Site.	28.2-84.2	1998-2008
341249117241101	Approximately 2,600 Feet Northeast of the Project Site.	35.5-66.3	1986-1998

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. No site with groundwater data was found within a 1.0-mile radius of the project site.

Based on available data, the historical high groundwater level reported at wells within approximately one mile of the site was approximately 28.2 feet bgs. Current groundwater is expected to be deeper than 21.0 feet bgs. 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 or irrigation events.

4.4 Expansive Soils

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

Based on the laboratory test result, the expansion index of the upper 5 feet of soils was 1.1, corresponding to a very low expansion potential.

4.5 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 0.2 and 0.4 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).

4.6 Excavatability

The surface and subsurface soil materials for the proposed improvements (pavement and grading) are expected to be excavatable by conventional heavy-duty earth moving equipment. <u>Excavation will be difficult if concentration of gravel is encountered.</u>

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

4.7 Subsurface Variations

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

5.0 ENGINEERING GEOLOGY

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

5.1 Regional Geology

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

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto 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.

5.2 Local Geology

Regional mapping (Morton & Miller 2006) indicates that the project site is primary underlain by late Holocene very young wash deposits consisting of unconsolidated sand and gravel deposits along with late Holocene and late Pleistocene young to very young alluvial-fan deposits consisting of unconsolidated to slightly coherent, essentially undissected deposits of silt, sand, gravel, cobbles, and boulders. The project is just south of the southern entrance to the Cajon Pass and the San Jacinto Fault Zone runs southeast-northwest through the project site.

6.0 FAULTING AND SEISMICITY

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 very high.

Table No. 5, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 100 kilometers of the proposed project (coordinate 34.2071N, 117.4057W). The data presented below was calculated using the National Seismic Hazard Maps Database 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	2.02	strike slip	241	n/a	7.88
S. San Andreas	3.17	strike slip	548	n/a	8.18
Cucamonga	4.42	thrust	28	5.0	6.70
Cleghorn	8.64	strike slip	25	3.0	6.80
North Frontal (West)	17.54	reverse	50	1.0	7.20
San Jose	28	strike slip	20	0.5	6.70
Sierra Madre Connected	31.99	reverse	76	2.0	7.30
Sierra Madre	31.99	reverse	57	2.0	7.20
Chino, alt 2	36.7	strike slip	29	1.0	6.80
Chino, alt 1	36.77	strike slip	24	1.0	6.70
Clamshell-Sawpit	40.37	reverse	16	0.5	6.70
Elsinore	44.99	strike slip	241	n/a	7.85
Helendale-So Lockhart	48.3	strike slip	114	0.6	7.40
Raymond	53.8	strike slip	22	1.5	6.80

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Puente Hills (Coyote Hills)	54.68	thrust	17	0.7	6.90
North Frontal (East)	57.85	thrust	27	0.5	7.00
Puente Hills (Santa Fe Springs)	64.14	thrust	11	0.7	6.70
Elysian Park (Upper)	65.61	reverse	20	1.3	6.70
Pinto Mtn	65.82	strike slip	74	2.5	7.30
Verdugo	69.19	reverse	29	0.5	6.90
San Joaquin Hills	69.22	thrust	27	0.5	7.10
Lenwood-Lockhart-Old Woman Springs	70.21	strike slip	145	0.9	7.50
Puente Hills (LA)	71.36	thrust	22	0.7	7.00
Johnson Valley (No)	75.46	strike slip	35	0.6	6.90
Hollywood	76.37	strike slip	17	1.0	6.70
Newport Inglewood Connected alt 2	80.08	strike slip	208	1.3	7.50
Newport Inglewood Connected alt 1	80.26	strike slip	208	1.3	7.50
Newport-Inglewood, alt 1	80.26	strike slip	65	1.0	7.20
San Gabriel	81.1	strike slip	71	1.0	7.30
Landers	81.32	strike slip	95	0.6	7.40
Santa Monica Connected alt 2	81.45	strike slip	93	2.4	7.40
Sierra Madre (San Fernando)	81.97	thrust	18	2.0	6.70
Newport-Inglewood (Offshore)	83.05	strike slip	66	1.5	7.00
Gravel Hills-Harper Lk	85.78	strike slip	65	0.7	7.10
So Emerson-Copper Mtn	87.47	strike slip	54	0.6	7.10
Northridge	89.92	thrust	33	1.5	6.90
Burnt Mtn	92.46	strike slip	21	0.6	6.80
Palos Verdes	93.43	strike slip	99	3.0	7.30
Palos Verdes Connected	93.43	strike slip	285	3.0	7.70
Santa Monica, alt 1	93.78	strike slip	14	1.0	6.60
Santa Monica Connected alt 1	93.78	strike slip	79	2.6	7.30
Eureka Peak	94.05	strike slip	19	0.6	6.70
Blackwater	95.52	strike slip	60	0.5	7.10
Calico-Hidalgo	95.91	strike slip	117	1.8	7.40

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

7.0 SEISMIC RECOMMENDATIONS

The primary and secondary seismic hazards are described as follows.

7.1 CBC Seismic Design Parameters

The 2022 CBC mapped acceleration parameters are provided in the following table. These parameters were determined using the *ATC Hazards by Location* website application, and in accordance with ASCE 7-16 Sections 11.4, 11.6, 11.8, 21.2, and 21.3.

Table No. 5, CBC Seismic Design Parameters

Parameter	Value
Site Coordinates	34.2085N, 117.4093W
Risk Category	II
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	2.449g
Mapped 1-second Spectral Response Acceleration, S_1	0.982g
Site Coefficient (Table 11.4-1), F_a	1.0
Site Coefficient (Table 11.4-2), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	2.449g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.669g
Design Spectral Response Acceleration for short period S_{ds}	1.633g
Design Spectral Response Acceleration for 1-second period, S_{d1}	1.113g
Site Modified Peak Ground Acceleration, PGA_M	1.134g

7.2 Liquefaction Potential

Liquefaction is defined as the phenomenon in which a cohesionless soil mass within the upper 50 feet of the ground surface, suffers a substantial reduction in its shear strength, due the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

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

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

The Glen Helen Park is located within a San Bernardino County-designated area liquefaction hazard zone and the generalized liquefaction susceptibility is high (San Bernardino County, 2010). Due to limitation of the field investigation site-specific *Liquefaction and Seismic Settlement Analysis* was not performed. However, due to the presence of cohesive soil and medium dense to dense granular and groundwater deeper than 21.0 feet bgs., liquefaction settlement will likely be low.

7.3 Seismic Settlement

Seismically induced settlement may occur in areas where there are relatively loose, dry, granular soils, or where liquefaction occurs. Due to limitation of the field investigation site-specific *Liquefaction and Seismic Settlement Analysis* was not performed. However, due to the presence of cohesive soil and medium dense to dense granular particles, Seismic settlement will likely be low.

7.4 Surface Fault Rupture

The site is located within a currently designated State of California and San Bernardino County Earthquake Fault Zone (CGS, 1995; San Bernardino County, 2010a). The Glen Helen Fault within the San Jacinto Fault Zone runs southeast-northwest through the northeast portion of the project site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered high. However, based on the type of proposed bridge improvements and the intended use, a detailed fault study is not considered necessary.

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.

7.5 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 the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Due to the topographic conditions of the site, lateral spreading is not considered a potential hazard at the site.

7.6 Tsunami/Seiches

Based on the inland location of the project site, tsunamis do not pose a hazard to this site. Seiching is possible with the nearby lake.

8.0 LABORATORY TEST RESULTS

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

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the subsurface soils. Test 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 soils were determined in accordance with ASTM Standard D2216 and D2937. Dry densities of the maximum explored depth of soils at the site ranged from 115 to 125 pcf with moisture contents ranging from 6 to 17 percent. Results are presented in the logs of borings in Appendix A, *Field Exploration.*

- **Expansion Index (EI)** – One representative bulk soil sample from the upper 5 feet of the site materials was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result indicated an expansion index of 1.1, corresponding to very low expansion potential.

- **Collapse Potential (CL)** – The collapse potential of two relatively undisturbed samples were tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The test results showed collapse potential values of 0.2 and 0.4 percent, indicating a slight collapse potential.

- **Grain Size Analysis (PA)** – Three representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*

- **Maximum Dry Density and Optimum Moisture Content (CP)** – Typical moisture-density relationship of one representative soil sample was performed in accordance with ASTM Standard D1557. The test result is presented in Drawing No. B-2, *Moisture-Density Relationship Result*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 128.5 pounds per cubic feet (pcf), with optimum moisture content of 9.0 percent. With rock corrections applied, the values are 131.6 pcf and 8.1 percent.

- **Direct Shear (DS)** – Two direct shear tests were performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The results of the direct shear tests are presented in Drawings No. B-3 and B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

8.2 Chemical Testing – Corrosivity Evaluation

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

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-02	1-5	7.5	31	22	5440

9.0 EARTHWORK RECOMMENDATIONS

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

9.1 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 structures.

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

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

9.2 Remedial Grading

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

Table No. 7, Overexcavation Depths

Structure/Pavement	Minimum Excavation Depth
Footings	18 inches below footings or 3 feet below ground surface, whichever is deeper
Slab-on-grade	18 inches below slab bottom
Pavement	12 inches below finish grade or existing grade, whichever is deeper

The overexcavation the below footing and pavement should be uniform. Under slab and pavement overexcavation should extend at least 1 foot beyond the edge of the pavement. Overexcavation below footings should extent horizontally equal to the vertical depth. The overexcavation bottom should be scarified and compacted as described in Section 9.4, *Compacted Fill Placement*.

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

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

9.3 Engineered Fill

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the project site are 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 maximum dimension.
- Rocks larger than 1 inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 30 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 40 percent fines (passing #200 sieve).

Based on field investigation and laboratory testing results, the on-site soils may be suitable as fill materials provided proper screening will be performed to remove the large size particles.

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

9.4 Compacted Fill Placement

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

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

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method, unless a higher compaction is specified herein. At least the upper 12 inches of subgrade soils below pavement finish grade should be compacted to at least 95 percent of the laboratory maximum dry density.

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

9.5 Site Drainage

Adequate positive drainage should be provided away from the site and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. Surface drainage should be directed to suitable non-erosive devices.

10.0 DESIGN RECOMMENDATIONS

Recommendations for the design and construction of the proposed structure are presented in the following sections. The recommendations provided are based on the assumption that, in preparing the site, the above earthwork recommendations will be implemented.

10.1 Shallow Foundation Design Parameters

The proposed gate entry monument may be supported on continuous and/or isolated spread footings while the proposed overhead entry monument may be supported on isolated spread footings. The design of the shallow foundations should be based on the recommended parameters presented in the table below.

Table No. 8, Recommended Foundation Parameters

Parameter	Value
Minimum continuous footing width	18 inches
Minimum isolated footing width	18 inches
Minimum continuous and/or isolated footing depth of embedment below lowest adjacent grade (proposed gate entry monument)	18 inches
Minimum isolated footing depth of embedment below lowest adjacent grade (proposed overhead entry monument)	24 inches
Allowable net bearing capacity	2,500 psf

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

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

10.2 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.

10.2.1 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 recommended lateral earth pressures for the site are presented in the following table.

Table No. 9, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure (psf/ft. depth)
Active earth conditions (wall is free to deflect at least 0.001 radian)	45
At-rest (wall is restrained)	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.

10.2.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by friction acting at the base of foundations and by passive earth pressure. Coefficients of friction of 0.30 between formed concrete and soil and 0.35 between steel and soil may be used. A passive earth pressure of 250 psf per foot of depth may be used for the sides of footings poured against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,500 psf.

These lateral resistances may be increased by 33 percent for seismic forces. Due to the low overburden stress of the soil at shallow depth, the upper one foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

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.

10.3 Settlement

The total settlement from static structural loads of single footing is anticipated to be less than 1 inch.

Due to limitation of the field investigation site-specific *Liquefaction and Seismic Settlement Analysis* was not performed. However, due to the presence of cohesive soil and medium dense to dense granular and groundwater deeper than 21.0 feet bgs., liquefaction settlement will likely be low.

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.

10.4 Soil Corrosivity

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

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

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

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

Table No. 10, Correlation Between Resistivity and Corrosion

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

The measured value of the minimum electrical resistivities of the sample when saturated was 5,440 ohm-cm. This indicates that the soils tested are moderately corrosive to ferrous metals in contact with the soil (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.

11.0 CONSTRUCTION RECOMMENDATIONS

11.1 General

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

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

Depending on the sequence of construction, excavations may be required near existing streets or structures, which may require vertical side wall excavation. Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

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

11.2 Temporary Sloped Excavations

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

Table No. 11, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
Silty Sand (SM) and Sandy Silt (ML)	C	0-4	Vertical
		4-10	1.5:1

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

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. Design recommendations for temporary shoring are provided in the following section.

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.

11.3 Shoring Design

Temporary shoring will be required where open sloped excavations will not be feasible due to unstable soils or due to nearby existing utilities, pavement, or structures. Temporary shoring may consist of conventional soldier piles and lagging or sheet piles. The shoring for the pipe excavations may be laterally supported by walers and cross bracing or may be cantilevered. Drilled excavations for soldier piles will require the use of drilling fluids to prevent caving and to maintain an opened hole for pile installation.

The active earth pressure behind any shoring depends primarily on the allowable movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures.

The lateral earth pressures to be used in the design of shoring is presented in the following table.

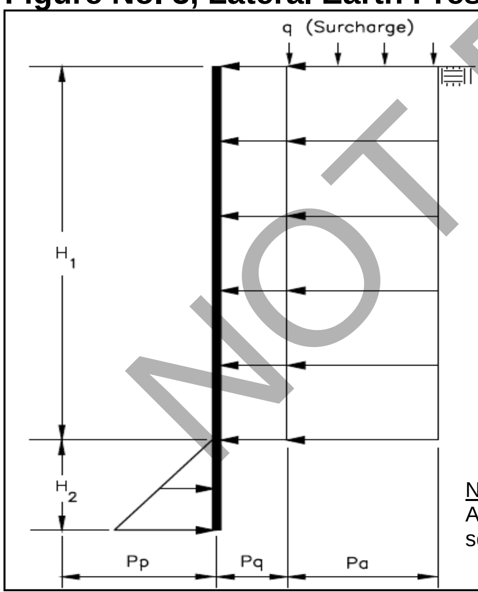
Table No. 12, Lateral Earth Pressures for Temporary Shoring

Lateral Resistance Soil Parameters*	Values
Active Earth Pressure (Braced Shoring) (psf) (A)	30
Active Earth Pressure (Cantilever Shoring) (psf) (B)	45
At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)	65
Passive earth pressure (psf per foot of depth) (D)	250
Maximum allowable bearing pressure against native soils (psf) (E)	2,500
Coefficient of friction between sheet pile and native soils, fs (degree) (F)	0.20

* Parameters A through F are used in Figures No. 3 and 4 below.

Restrained (braced) shoring systems should be designed based on Figure No. 3, *Lateral Earth Pressures for Temporary Braced Excavation* to support a uniform rectangular lateral earth pressure.

Figure No. 3, Lateral Earth Pressures for Temporary Braced Excavation



Total Earth Pressure, P

P = Pq + Pa

Pq = 0.5q - incremental surcharge pressure

Pa = (A)H_1 - active earth pressure (Braced walls)

Lateral Pressure Resistance

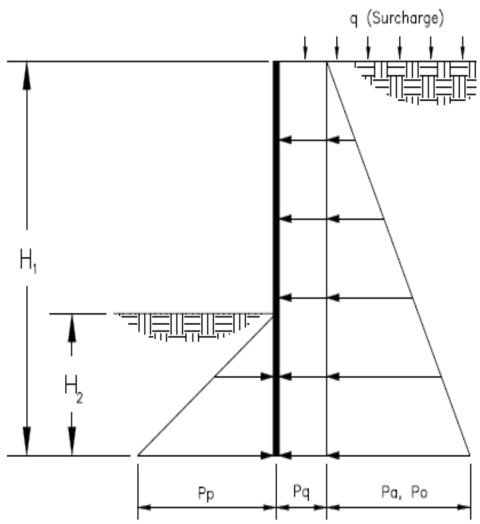
Pp = (D) H_2 ≤ (E) psf - passive earth pressure (on native soils)

μ = (F) - ultimate friction coefficient between steel sheet piles and soil

Note:
All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).

Unrestrained (cantilever) design of cantilever shoring consisting of soldier piles spaced at least two diameters on-center or sheet piles, can be based on Figure No. 4, *Lateral Earth Pressures on Temporary Cantilever Wall*.

Figure No. 4, Lateral Earth Pressures on Temporary Cantilever Wall



Total Earth Pressure, P

P = Pq + Pa, Po

$P_q = 0.5q$ - incremental surcharge pressure

$P_a = (B)H_1$ - active earth pressure (Un-restrained)

$P_o = (C)H_1$ - at rest earth pressure (Restrained)

Lateral Pressure Resistance

$P_p = (D)\ H_2 \leq (E)$ psf - passive earth pressure (on native soils)

μ = (F) - ultimate friction coefficient between steel

Note:
All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).

The provided pressures assume no hydrostatic pressures. If hydrostatic pressures are allowed to build up, the incremental earth pressures below the ground-water level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

Passive resistance includes a safety factor of 1.5. The upper 1 foot for passive resistance should be ignored.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation. As previously mentioned, all shoring should be designed and installed in accordance with state and federal safety regulations.

The contractor should have provisions for soldier pile and sheet pile removal. All voids resulting from removal of shoring should be filled. The method for filling voids should be selected by the contractor, depending on construction conditions, void dimensions and available materials. The acceptable materials, in general, should be non-deleterious, and able to flow into the voids created by shoring removal (e.g., concrete slurry, “pea” gravel, etc).

Excavations should not extend below a 1:1 (horizontal:vertical) plane extending from the bottom of any existing structures, utility lines or streets. Any proposed excavation should not cause loss of bearing and/or lateral supports of the existing utilities or streets.

If the excavation extends below a 1:1 (horizontal:vertical) plane extending from the bottom of the existing structures, utility lines or streets, a maximum of 10 feet of slope face parallel to the existing improvement should be exposed at a time to reduce the potential for instability. Backfill should be accomplished in the shortest period of time and in alternating sections.

12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

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

13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by San Bernardino County Department of Public Works - Special Districts and their authorized agents, to assist in the design and construction of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

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

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

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

14.0 REFERENCES

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Appendix A

Field Exploration

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling soil borings. During the site reconnaissance, the surface conditions were noted, and the borings were marked at the locations selected by Charles Hernandez with the San Bernardino County Department of Public Works - Special Districts. The approximate boring locations were established in the field by reference to the existing street and sidewalk along with other visible features. The locations should be considered accurate only to the degree implied by the method used.

Two exploratory borings (BH-01 and BH-02) were drilled on February 23, 2023, to investigate the subsurface conditions. The borings were drilled to the depths of approximately 21.0 feet below ground surface (bgs). Details of these borings are presented in Table No. A-1, *Summary of Borings* and Drawings No. A-2 and A-3, *Logs of Borings.*

Table No. A-1, Summary of Borings

Boring No.	Boring Depth (ft, bgs)		Groundwater Depth (ft., bgs)	Date Completed
	Proposed	Completed		
BH-01	20.0	21.0	Not Encountered	02/23/2023
BH-01	20.0	21.0	Not Encountered	02/23/2023

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

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

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 were backfilled with soil cuttings mixed with cement, compacted by pushing down with an auger using the drill rig weight, and the surface patched with cold-mix asphalt. If construction is delayed, the surface may settle over time. We recommend the owner monitor the boring locations and backfill any depression 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 Drawings No. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For the logs of borings, see Drawings No. A-2 and A-3, *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

Converse Consultants

Glen Helen Regional Park Monument
2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California
San Bernardino County Department of Public Works - Special Districts

Project No. 23-81-127-01

Drawing No. A-1a

Project ID: 23-81-127-01.GPJ;Template: KEY

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: 2/23/2023 Logged by: Aleksey Zhukov Checked By: Hashmi S. Quazi

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

Ground Surface Elevation (ft): 2032 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
	This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
0	**4" ASPHALT CONCRETE / 4" AGGREGATE BASE**						
	ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.25" in maximum dimension - sub-angular to sub-rounded, trace clay, trace caliche, medium dense, moist, brownish black to orangish brown.		X				CP, EI, PA
		X	X	6/8/12	16	115	
5	**SANDY SILT (ML):** fine to coarse-grained sand, scattered gravel up to 0.75" in maximum dimension - sub-angular to sub-rounded, trace clay, trace caliche, hint of black oxidation spotting, medium plasticity, stiff, moist, olive orangish brown with hint of orange mottling.	X	X	6/8/14	16	115	CL, PA
	@ 7.5': few caliche, no oxidation spotting, low plasticity, very stiff, olive orangish brown with orange mottling.	X	X	8/12/14	16	117	DS
10	@ 10': no gravel, no clay, medium plasticity.	X		8/13/18	12	116	
15	**SILTY SAND (SM):** fine to coarse-grained, little caliche, weakly cemented, very dense, moist, olive orangish brown with orange mottling.	X		15/27/50-5.5"	11	125	
20	**SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, gravel up to 2.5" in maximum dimension - angular to sub-rounded, little caliche, moderately cemented, very dense, moist, olive orangish brown with orange mottling.	X		20/50-6"	6	119	

End of Boring at 21.0 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings mixed with cement and compacted by pushing down with augers using the drill rig weight and surface patched with cold-mix asphalt on 02/23/2023.

Log of Boring No. BH-02

Date Drilled: 2/23/2023 Logged by: Aleksey Zhukov Checked By: Hashmi S. Quazi

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

Ground Surface Elevation (ft): 2034 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
		4" ASPHALT CONCRETE / 3" AGGREGATE BASE						
		ALLUVIUM: **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.25" in maximum dimension - sub-angular to sub-rounded, trace clay, trace caliche, medium dense, moist, dark brown to orangish brown.						CR, PA
					7/10/12	16	117	
5		**SANDY SILT (ML):** fine to coarse-grained sand, scattered gravel up to 0.5" in maximum dimension - sub-angular to sub-rounded, trace clay, trace caliche, medium plasticity, stiff, moist, olive orangish brown with hint of orange mottling. @ 7.5': no clay, few caliche, hint of black oxidation spotting, low plasticity, very stiff, olive orangish brown with orange mottling.			5/9/12	16	115	
					9/16/20	14	122	CL
10					12/12/14	17	115	DS
15		**SILTY SAND (SM):** fine to coarse-grained, little caliche, weakly cemented, dense, moist, olive orangish brown with orange mottling.			12/20/27	14	120	
20		**SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, gravel up to 2" in maximum dimension - angular to sub-rounded, little caliche, moderately cemented, very dense, moist, olive orangish brown with orange mottling.			29/50-5.5"	8	120	

End of Boring at 21.0 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings mixed with cement and compacted by pushing down with augers using the drill rig weight and surface patched with cold-mix asphalt on 02/23/2023.

Appendix B

Laboratory Testing Program

APPENDIX B

LABORATORY TESTING PROGRAM

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

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

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

Expansion Index

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

Table No. B-1, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-01	1-5	Silty Sand (SM)	1.1	Very Low

Soil Corrosivity

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

Table No. B-2, Summary of Soil Corrosivity Test Result

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (% by weight)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-02	1-5	7.5	31	22	5,440

Collapse Potential

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

Table No. B-3, Collapse Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-01	5.0-6.5	Sandy Silt (ML)	-0.4	Slight
BH-02	7.5-9.0	Sandy Silt (ML)	-0.2	Slight

Grain-Size Analyses

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

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

Boring No.	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-01	1-5	Silty Sand (SM)	13.0	44.6	42.4	
BH-01	5-10	Sandy Silt (ML)	1.0	45.1	53.9	
BH-02	1-5	Silty Sand (SM)	10.0	46.4	43.6	

Maximum Density and Optimum Moisture Content

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

Table No B-5, Summary of Moisture-Density Relationship Result

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb./cft)
BH-01	1-5	Silty Sand (SM)	9.0 (8.1*)	128.5 (131.6*)

(*Rock correction BH-01@1' to 5'-Optimum moisture=8.1%, Maximum Density=131.6 lb/cft)

Direct Shear

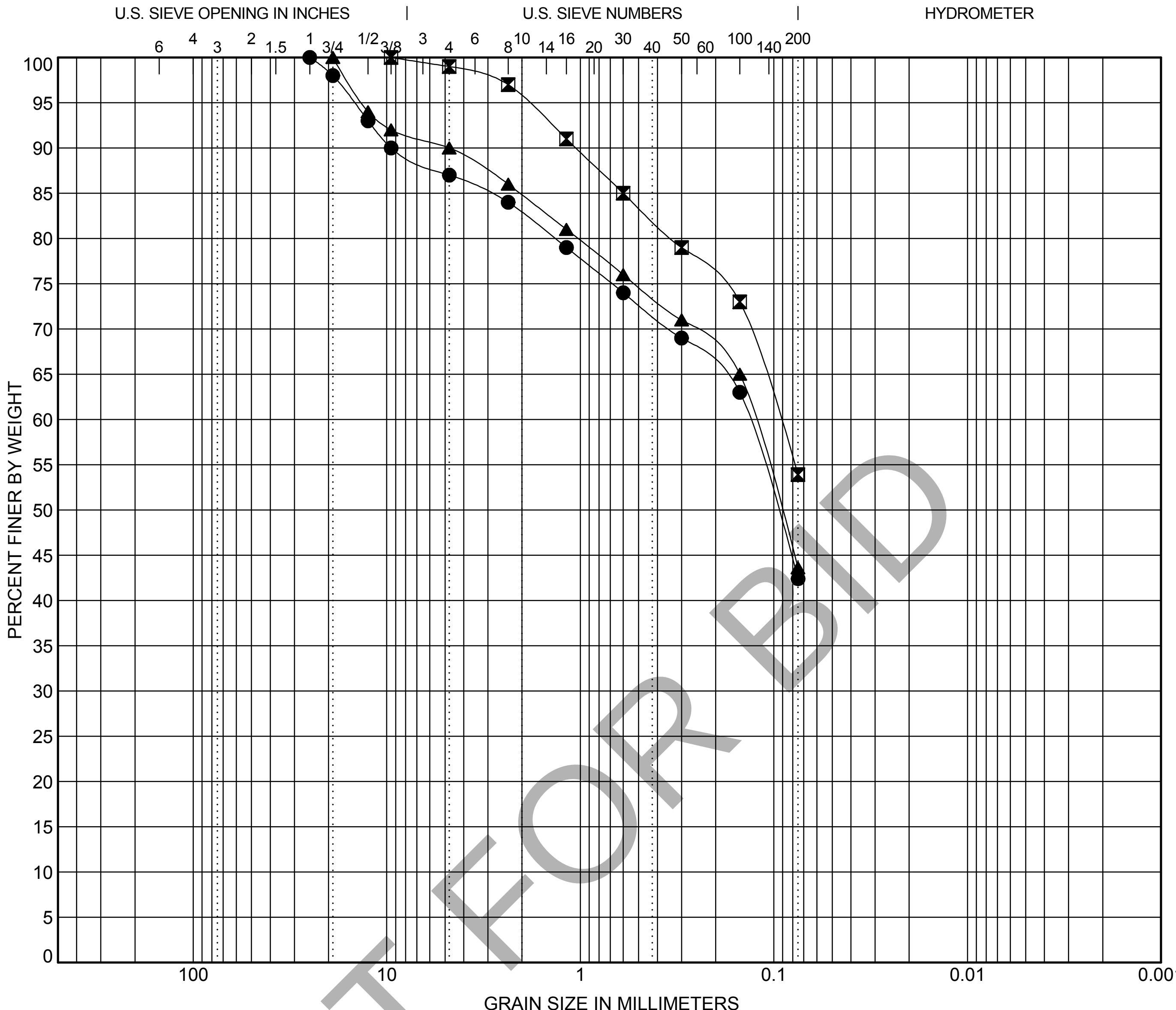
Two direct shear tests were performed on relatively undisturbed samples under soaked conditions in accordance with ASTM Standard 3080. For each test, 3 samples contained in a brass sampler ring were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.01 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 results, including sample density and moisture content, see Drawings No. B-3 and B-4*, Direct Shear Test Results,* and in the following table.

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

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-01	7.5-9.0	Sandy Silt (ML)	33	190
BH-02	10.0-11.5	Sandy Silt (ML)	32	90

Sample Storage

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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	BH-01	1-5	SILTY SAND (SM)					
⊠	BH-01	5-10	SANDY SILT (ML)					
▲	BH-02	1-5	SILTY SAND (SM)					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	BH-01	1-5	25	0.136			13.0	44.6	42.4	
⊠	BH-01	5-10	9.5	0.094			1.0	45.1	53.9	
▲	BH-02	1-5	19	0.128			10.0	46.4	43.6	

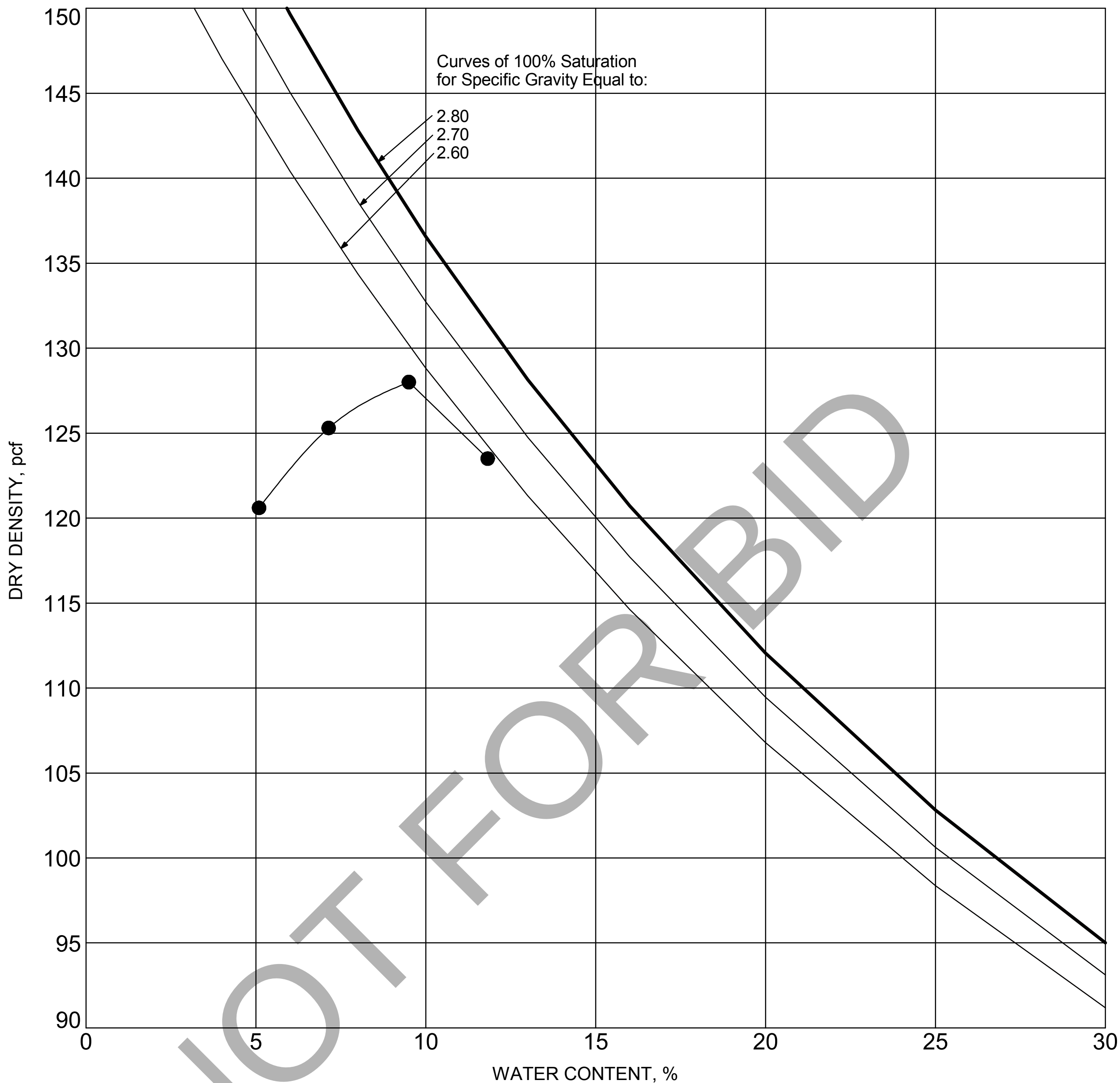
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Glen Helen Regional Park Monument
2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Department of Public Works - Special Districts

Project No. 23-81-127-01

Drawing No. B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**1-5**	**SILTY SAND (SM), BROWNISH BLACK TO ORANGISH BROWN**	**D1557 Method B**	**9 (8.1*)**	**128.5(131.6*)**

***Rock correction BH-01@1' to 5'-Optimum moisture=8.1%, Maximum Density=131.6 lb/cft**

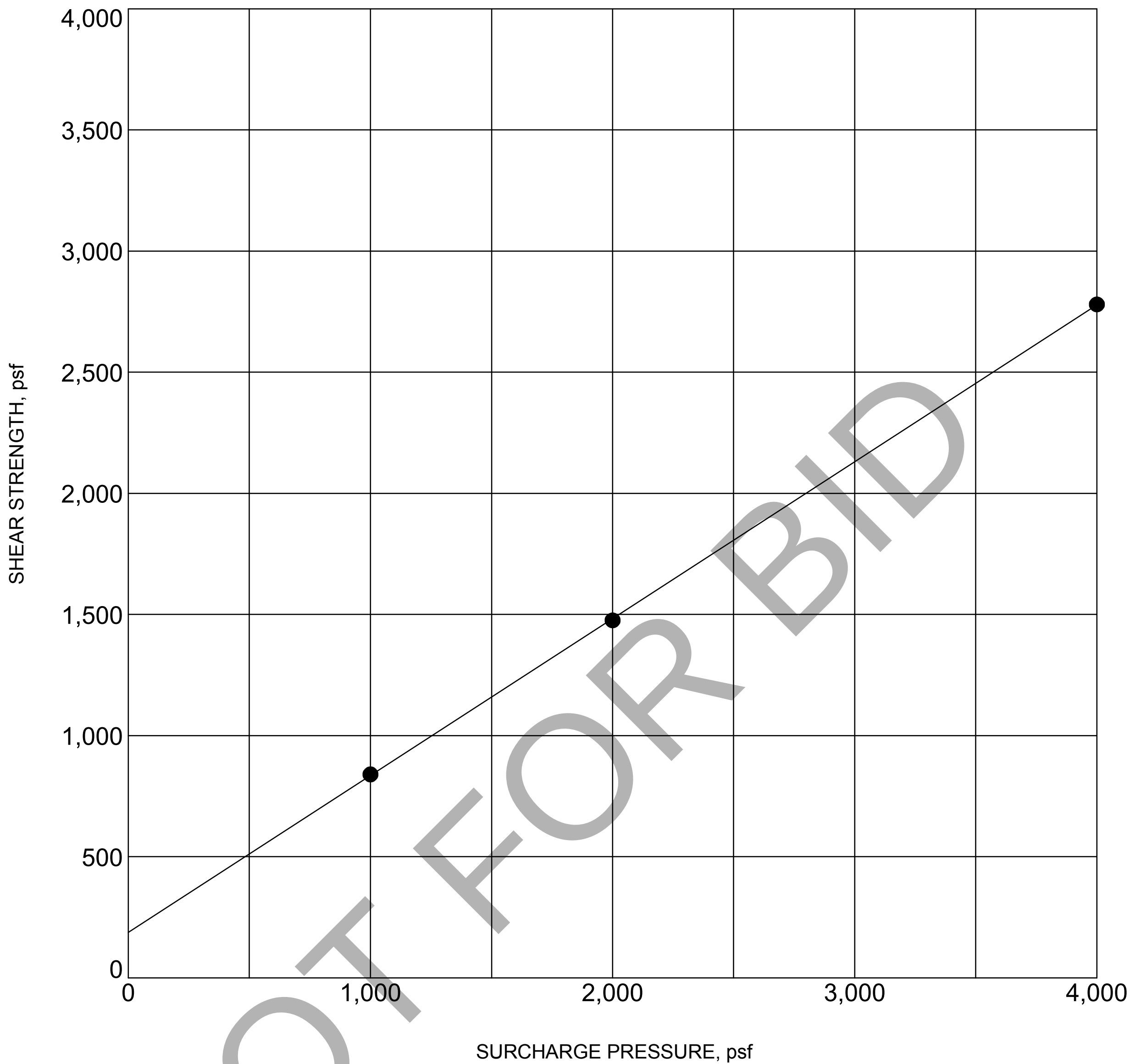
MOISTURE-DENSITY RELATIONSHIP RESULT

Converse Consultants

Glen Helen Regional Park Monument
2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Department of Public Works - Special Districts

Project No. **23-81-127-01**

Drawing No. **B-2**



BORING NO. :	BH-01	DEPTH (ft) :	7.5-9.0
DESCRIPTION :	SANDY SILT (ML)		
COHESION (psf) :	190	FRICTION ANGLE (degrees):	33
MOISTURE CONTENT (%) :	16.0	DRY DENSITY (pcf) :	116.8

NOTE: Ultimate Strength.

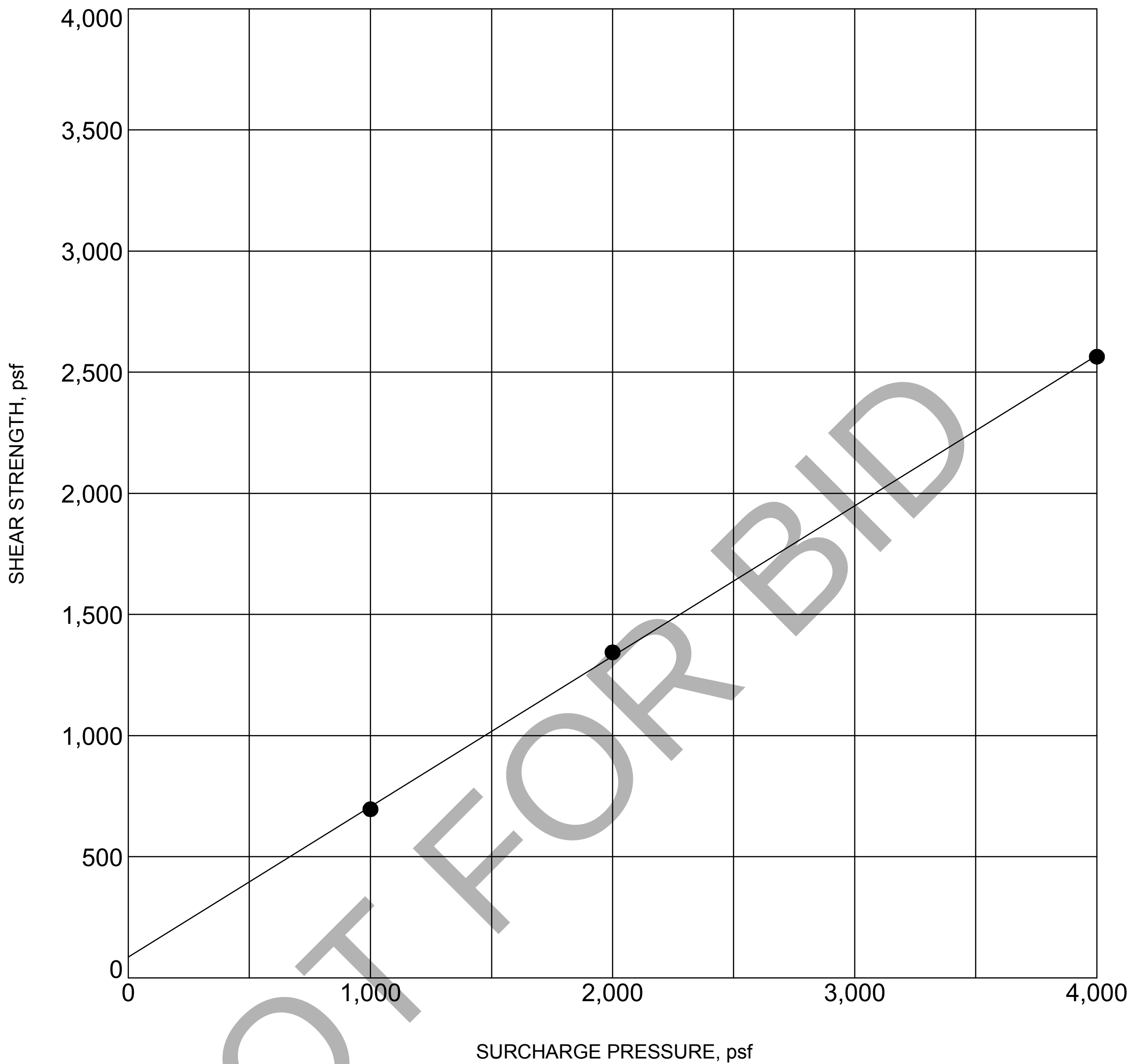
DIRECT SHEAR TEST RESULTS

Converse Consultants

Glen Helen Regional Park Monument
2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Department of Public Works - Special Districts

Project No. **23-81-127-01**

Drawing No. **B-3**



BORING NO. :	BH-02	DEPTH (ft) :	10.0-11.5
DESCRIPTION :	SANDY SILT (ML)		
COHESION (psf) :	90	FRICTION ANGLE (degrees):	32
MOISTURE CONTENT (%) :	17.0	DRY DENSITY (pcf) :	114.7

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Glen Helen Regional Park Monument
2555 Glen Helen Parkway
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Department of Public Works - Special Districts

Project No. **23-81-127-01**

Drawing No. **B-4**