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

GEOTECHNICAL INVESTIGATION REPORT

WELL 152 EQUIPMENT BUILDING (W2255)
31111 Greenspot Road
City of Highland, San Bernardino County, California

CONVERSE PROJECT NO. 25-81-144-01



Prepared For:
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May 7, 2026

Converse Consultants

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

May 7, 2026

Mr. Shane Bloomfield, PE
Senior Engineer
Albert A. Webb Associates
3788 McCray Street
Riverside, CA 92506

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Well 152 Equipment Building (W2255)
31111 Greenspot Road
City of Highland, San Bernardino County, California
Converse Project No. 25-81-144-01

Dear Mr. Bloomfield:

Converse Consultants (Converse) is pleased to submit this Geotechnical Investigation Report to assist with the design and construction for the proposed equipment housing building and associated internal piping at the Well 152 site located at 31111 Greenspot Road in the City of Highland, San Bernardino County, California. The report was prepared in accordance with our proposal dated March 17, 2026, and your Single-Project Subconsultant Agreement dated March 13, 2026.

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 specifically dealing with large sized material, are incorporated into the design and construction of the project.

We appreciate the opportunity to be of service to Albert A. Webb Associates and the East Valley Water District. (EVWD) 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
AR/JP/JW/HSQ/kvg

PROFESSIONAL CERTIFICATION

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

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



Jason Bragg
Staff Geologist



Anthony Rosas
Staff Engineer









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

Catherine Nelson, PG
Project Geologist

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation which was performed for the East Valley Water District Well 152 Equipment Building, located in the City of Highland, San Bernardino County, California. The approximate location of the booster station site is shown on Figure No. 1, *Approximate Site Location Map*.

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

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

2.0 PROJECT DESCRIPTION

The drilling and equipping of proposed Well No. 152 will occur at East Valley Water District's (EVWD) headquarters at 31111 Greenspot Road, Highland, California. The project will consist of 2 primary phases.

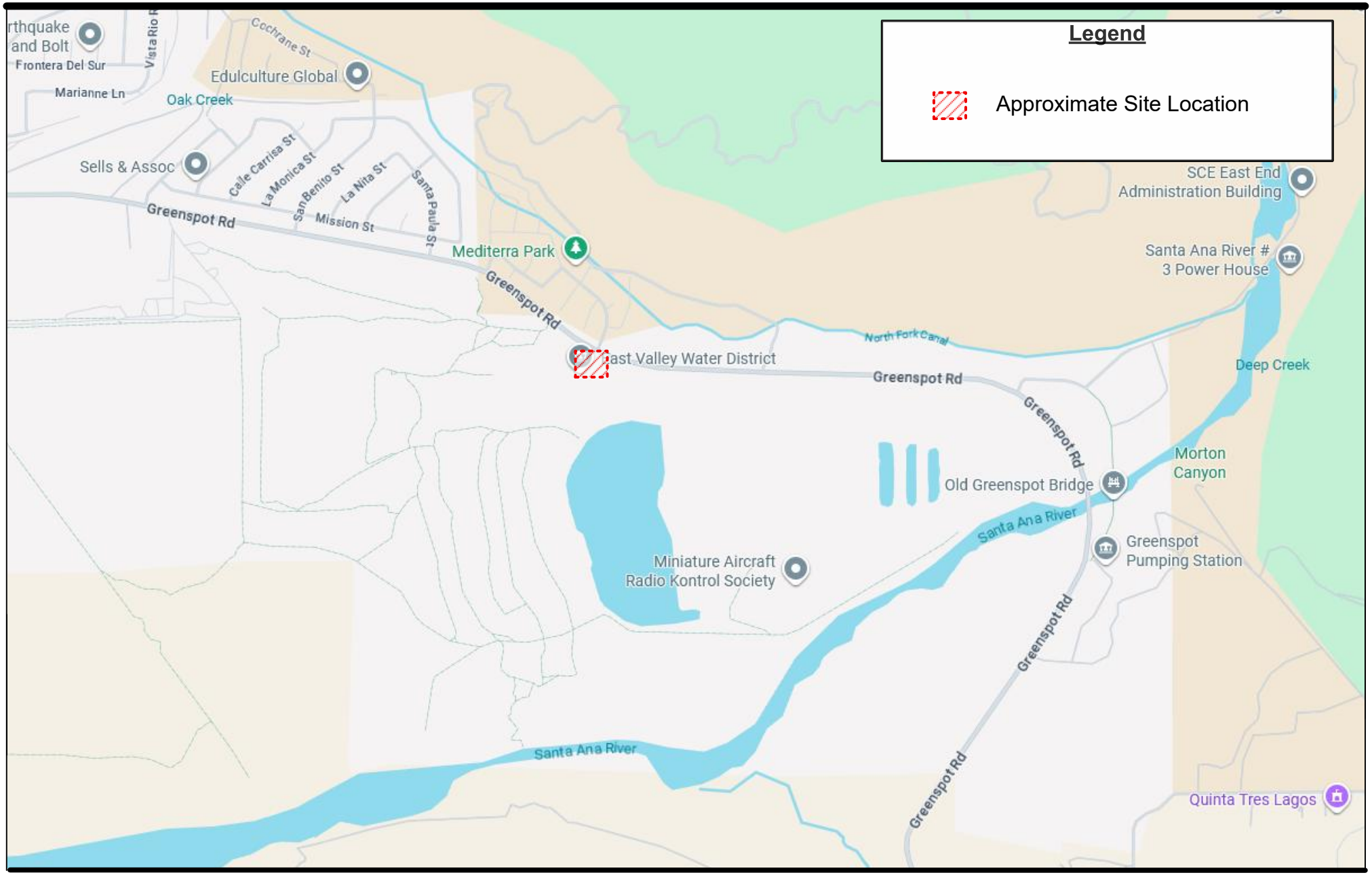
Phase 1 - Drilling and equipment design (Converse will not be involved in this phase).

Phase 2 – Construction of the Well No. 152 Equipment building and the onsite pipeline will be initiated.

The design will consist of an approximately 800 square feet above grade masonry building, sodium hypochlorite dosing system, mechanical piping, electrical, HVAC, and instrumentation. Approximately 800 feet of onsite discharge pipeline will be constructed leading north out of the well site to Greenspot Road.

3.0 SITE DESCRIPTION

The site of the Well 152 Equipment Building will be located at the south side of the equipment yard, within the EVWD Headquarters. The site is approximately 0.08 acres and is bounded by the equipment yard to the north and east, a fence and a flowing channel to the south, and a percolation basin to the west. The site is undeveloped besides some stockpiles of dirt and cobbles. Many large boulders up to approximately 4 feet in maximum dimension are present within the project limits.



Project: Well 152 Equipment Building

Location: 31111 Greenspot Road
City of Highland, San Bernardino County, California

For: Albert A. Webb Associates

Approximate Site Location Map

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Figure No.
1

The onsite pipeline will run under the south side of the equipment lot and then turn north and run up the east side of the equipment yard into Greenspot Road. This area is covered by asphalt concrete. For the current condition of the site see Photograph No.1 and No. 2 below.



Photograph No. 1 shows the conditions of the ground within the site facing east.



Photograph No. 2 shows the condition of the ground surface at the site, note the large number of cobbles and boulders that are present.

4.0 SCOPE OF WORK

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

4.1 Project Set-up

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

- Prepared the boring location map based on the exhibit provided by Shane Bloomfield with Webb.
- Conducted a site reconnaissance with Nathan Carlson with EVWD and staked/marked the boring locations such that drill rig access to all the locations were 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.

- Notified East Valley Water District prior to drilling to clear the boring locations of any conflict with existing private underground utilities.
- Engaged a California-licensed driller to drill the exploratory boring.

4.2 Subsurface Exploration

Two exploratory borings were drilled on March 25, 2026, to investigate the subsurface conditions. It was planned to drill the borings to 50 and 20 feet below the existing ground surface, however due to the refusal in cobbles and boulders the borings could only be penetrated to 5 feet below the ground surface. The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for sampling of soils. Drilling summary is presented below.

Table No. 1, Summary of Borings

Boring No. (Location)	Boring Depth (ft, bgs)		Groundwater Depth (ft, bgs)	Date Completed
	Proposed	Completed		
BH-01 (Equipment Building)	50.0	5.0	Not Encountered	3/25/2026
BH-02 (Pipeline)	20.0	5.0	Not Encountered	3/25/2026

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

4.3 Laboratory Testing

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

- *In-situ* moisture contents and dry density (ASTM D2216-19 and ASTM D7263)
- Soil Expansion Index (ASTM D4829-21)
- R-value (R) (ASTM D2844/D2844M-18E01)
- Sand Equivalent (ASTM D2419-22)
- Soil corrosivity (CTM 422, 417, 532, and 643)
- Passing Sieve No.200 (ATSM D1140-17)
- Maximum dry density and optimum moisture content (ASTM D1557-12R21)
- Direct Shear (ASTM D3080/D3080M-23)

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*.

Legend

BH-01 / 5.0'

Number / Depth and Approximate Location of Exploratory Boring



811 Know what's below. Call before you dig.

REV.	DATE	DESCRIPTION	BY

DRAWN BY: ## DATE: 2/12/2026 CHECKED BY: ## DATE:

PREPARED UNDER THE SUPERVISION OF: BY: ENGINEER, R.C.E. NO. 77435 DATE:

ALBERT A. WEBB ASSOCIATES
ENGINEERING CONSULTANTS
3788 McCRAY STREET
RIVERSIDE, CA. 92506
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FAX (951) 788-1256
BENCH MARK: SEE TITLE SHEET

APPROVED FOR THE DISTRICT: BY: ENGINEER, R.C.E. NO. ## DATE:

EVWD East Valley Water District
3654 E. HIGHLAND AVENUE, SUITE 18
HIGHLAND, CALIFORNIA 92346-2607

EAST VALLEY WATER DISTRICT
WELL NO. 152
PRELIMINARY SITE PLAN

E.V.W.D. DWG. NO. ##
E.V.W.D. JOB NO. W2231
SCALE:
DWG. NO.:
SHEET NO. 1 OF 1

Project: Well 152 Equipment Building

Location: 31111 Greenspot Road
City of Highland, San Bernardino County, California

For: Albert A. Webb Associates

Approximate Boring Locations Map

Project No.
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Converse Consultants

Figure No.
2

4.4 Report Preparation

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

5.0 SITE CONDITIONS

5.1 Subsurface Profile

Based on the exploratory borings and laboratory test results, the subsurface soil within the project area primarily consisted of a silty sand. Little to some gravels, cobbles, and boulders up to 4-feet in maximum dimension were also present.

Discernible fill soils were not identified in our subsurface exploration; however, the site may have been previously graded for the existing development (pavement) and fill soil is likely present in some locations. If present, the fill soils are likely derived from on-site sources and are similar to the native 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, Log of Borings, in Appendix A, Field Exploration.

5.2 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.

Table No. 2, Expansion Index vs Potential

Expansion Index	Potential Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Based on the laboratory test result, the expansion index (EI) of the upper 5 feet of site soils was 0, corresponding to very low expansion potential.

5.3 Excavatability

The surface and subsurface soil materials for the proposed project are not expected to be excavatable by conventional heavy-duty earth moving equipment. Excavation will be difficult where concentration of gravel, cobbles and/or boulders are encountered. The contractor must select appropriate means and methods to excavate through the hard materials.

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

5.4 Groundwater

Groundwater was not encountered during the field investigation up to a depth of 5.0 feet bgs. For comparison, several regional and state groundwater databases were accessed for project site groundwater conditions.

Regional groundwater data from the GeoTracker database (SWRCB, 2026) was reviewed to evaluate the current and historical groundwater levels. No site with groundwater data was identified within a 1.0-mile radius of the project site.

The National Water Information System (USGS, 2026) 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 in Table No. 3, *Summary of USGS Groundwater Depth Data,* below.

Table No. 3, Summary of USGS Groundwater Depth Data

Site Number	Location	Groundwater Depth Range (ft. bgs)	Date Range
340620117065001	Approximately 2,900 feet northeast of project site.	20-68	1928-2008
340618117064501	Approximately 3,000 feet northeast of project site.	18-133	1959-1988
340540117072201	Approximately 3,000 feet northeast of project site.	130-318	1997-2008
340541117074405	Approximately 3,700 feet northeast of project site.	14-85	2023-2026

The California Department of Water Resources database (DWR, 2026) was reviewed for historical groundwater data from sites within a 1.0-mile radius of the project site. One

site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Details of that record are listed below.

- Well No. Well North Fork #1 (Station 341047N1171105W001), located approximately 4,500 feet northeast of the project site, reported groundwater at a depth ranging from 55 to 78 feet bgs between 2011 and 2022.
- Well No. SBVWCD#1 (Station 341060N1171270W001), located approximately 1,400 feet northwest of the project site, reported groundwater at a depth ranging from 74 to 323 feet bgs between 2011 and 2025.
- Well No. SAR1 (Station 341040N1171337W001), located approximately 2,300 feet west of the project site, reported groundwater at a depth ranging from 69 to 335 feet bgs between 2011 and 2025.

Based on available data, the historical high groundwater level reported within approximately one mile of the site was deeper than approximately 14 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.

Figure No. 3, *Geology and Groundwater Map*, depicts the geologic setting and near site groundwater locations and depths for the project site.

5.5 Subsurface Variations

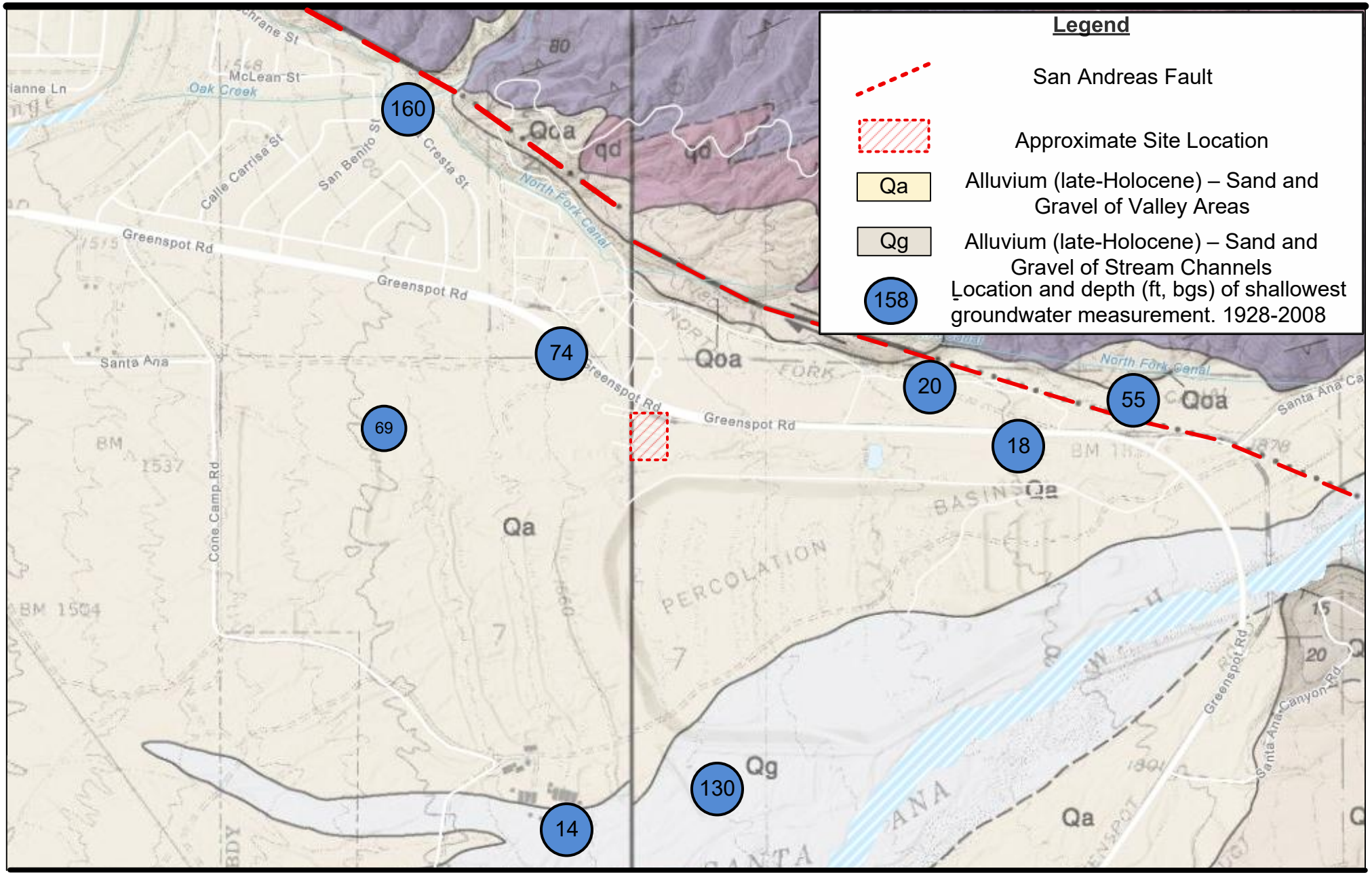
Based on the results of our subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project area should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care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

6.0 SEISMICITY

The seismic design coefficients are presented in the following subsections.

6.1 CBC Seismic Design Parameters

Seismic parameters based on the 2025 California Building Code (CBSC, 2025) and ASCE 7-22 are provided in the following table. These parameters were determined using a coordinate (34.1036° N, -117.1258 W) and the ASCE Online Hazard Tool.



Project: Well 152 Equipment Building

Location: 31111 Greenspot Road
City of Highland, San Bernardino County, California

For: Albert A. Webb Associates

Geology And Groundwater Map

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Figure No.
3

Table No. 4, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.1036° N, -117.1258 W
Site Class	D
Risk Category	III
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	2.66g
Mapped 1-second Spectral Response Acceleration, S_1	1.01g
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.79g
MCE 1-second period Spectral Response Acceleration, SM_1	2.75g
Design Spectral Response Acceleration for short period S_{DS}	1.86g
Design Spectral Response Acceleration for 1-second period, S_{D1}	1.84g
Site Modified Maximum Peak Ground Acceleration, PGA_M	0.97g

** Stiff Soil Classification*

6.2 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 Earthquake or San Bernardino County Fault Zone (CGS, 2007, San Bernardino County, 2025). 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 a presently unrecognized fault beneath 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 and 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 in a designated State of California Liquefaction Zone (CGS, 2025), but it is in a County of San Bernardino High Liquefaction area (SBC,

2022). Based on the very dense soil conditions encountered as well as current ground water being deeper than approximately 120 feet, the potential of liquefaction induced settlement is anticipated to be low.

***Seismic Settlement**:* Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. Based on the dense soil conditions encountered, the potential of seismic settlement is anticipated to be negligible.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or after earthquakes in areas of significant relief. The project site is not adjacent to any steep slopes. The risk of landslides at this project site is considered low.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials 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 lack of liquefaction potential, the risk for lateral spreading to affect the site is considered low.

Tsunamis: Tsunamis are tidal waves generated in large bodies of water by fault displacement or major ground movement. Based on the 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 indicates that the project site is located near the Santa Ana River infiltration basin and the Seven Oaks Dam Reservoir. The risk of seiches at these locations is unknown. The risk of seiches is not known

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is located within a State of California designated dam inundation zone (DOSD, 2026). The site is located within the flood zone of the Seven Oaks Dam. The risk of earthquake-induced flooding affecting the project site is considered high

7.0 LABORATORY TEST RESULTS

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

7.1 Physical Testing

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

- *In-situ* Moisture and Dry Density – *In-situ* dry density and moisture content of the soils were determined in accordance with ASTM Standard D2216-19 and D7263-21. The dry density of the upper 5 feet soils at the site soil was 115 pcf with a moisture content of 6 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Expansion Index (EI) – One representative soil sample was tested to evaluate the expansion potential in accordance with ASTM Standard D4829-21. The test result showed an EI of 0, corresponding to very low expansion potential.
- Sand Equivalent (SE) – One representative bulk soil sample was tested to evaluate sand equivalent (SE) in accordance with the ASTM Standard D2419-22 test method. The measured sand equivalent test was 48.
- R-Value (R) – One representative bulk sample was tested in accordance with the ASTM Standard D2844/D2844M-18E01. The result of the R-value test was 50.
- Passing Sieve No.200 (WA) – Two representative soil samples were tested to determine the percent of material passing through a No.200 Sieve in accordance with the ASTM Standard D1140-25. The test results are presented in *Passing Sieve No.200 Test Results* in Appendix B, *Laboratory Testing Program*.
- Maximum Dry Density and Optimum Moisture Content (CP) – The moisture-density relationship of a representative soil sample was tested in according to ASTM Standard D1557-12R21 and the results are presented in Drawing No. B-1, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 129.0 pounds per cubic feet (pcf) with optimum moisture content of 9.0 percent.
- Direct Shear – One direct shear test was performed in accordance with ASTM Standard D3080/D3080M-23 on relatively undisturbed ring samples. The result of the direct shear test is presented in Drawing No. B-2, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

7.2 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. These tests were performed by Keegan Labs, in accordance with California Tests 643, 422, and 417. The test results are presented in Appendix B, *Laboratory Testing Program* and summarized below.

- The pH measurement of the sample was 8.3.
- The soluble sulfate content of the sample was 13 ppm.
- The chloride concentration of the sample was 14 ppm.

- The minimum electrical resistivity of the sample when saturated was 29,100 ohm-cm.

8.0 EARTHWORK RECOMMENDATIONS

Earthwork recommendations for the project are presented below.

8.1 General

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

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

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

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

8.2 Overexcavation/Removal

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

Table No. 5, Overexcavation Depths

Structure	Minimum Excavation Depth
Footings	18 inches below footings bottom or 30 inches below existing ground surface, whichever is deeper
Slabs-on-grade	12 inches below slab bottom

The depth of over excavation below the footings and slabs should be uniform. The over excavation should extend to at least 2 feet beyond the footprint of the footings and slabs. The over excavation bottom should be scarified and compacted as described in Section 8.4, *Compacted Fill Placement*.

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

The contractor should determine the best manner to conduct the excavations, such that there are no losses of bearing and/or lateral support to the existing structures or utilities. Consideration should be given to using slot cuts or other excavation methods which preserve lateral support during excavation operations near the existing structures.

8.3 Engineered/Structural 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 site are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including cleaning roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-site soils used as fill should meet the following criteria.

- No particles larger than 3 inches in 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 20 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Based on field the investigation and laboratory test results, on-site soils are suitable as fill materials provided proper screenings will be performed to remove large sized particles to meet above-mentioned criteria and applicable corrosion mitigations are performed.

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

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

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein.

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.

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.

8.5 Site Drainage

Adequate positive drainage should be provided away from the structure and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. The building pad should have a gradient of at least 2 percent towards drainage facilities. The drainage gradient should be 1 percent for paved areas and 2 percent for landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

8.6 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.

8.6.1 Pipeline Subgrade Preparation

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

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

8.6.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Pipe bedding should follow EVWD Standards. If additional recommendations beyond EVWD Standards are needed, the following specifications can be used during the placement of pipe bedding.

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. 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.

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

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

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.

8.6.3 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 site soil free of oversized particles and deleterious matter may be used to backfill the trench zone. Trench backfill should follow EVWD Standards. If additional recommendations are needed, the following specifications can be used during the placement of trench backfill.

- 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.

9.0 DESIGN RECOMMENDATIONS

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

9.1 Shallow Foundation Design Parameters

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

Table No. 6, Shallow Foundation Design Parameters

Parameter	Value
Minimum continuous spread footing width	18 inches
Minimum isolated footing width	18 inches
Minimum continuous or isolated footing depth of embedment below lowest adjacent grade	18 inches
Allowable net bearing capacity	3,000 psf

The allowable bearing capacity can be increased by 500 psf with each foot of additional embedment up to a maximum of 4,000 psf.

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

9.2 Mat Foundation Design Parameters

The proposed pump slab may be designed as mat foundation. The modulus of subgrade reaction (k) for design of flexible mat foundation was estimated from the available soil compressibility data and published charts. For design of flexible mat foundation for the Pressure Reducing/Booster Station 6711 site, the following equation may be used.

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

Where:
k= vertical modulus of subgrade reaction for mat foundation, kips per cubic foot (kcf)
k_1= 150 kcf, normalized modulus of subgrade reaction for 1-square-foot footing
B= foundation width, feet

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

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

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

An allowable net bearing capacity of 3,000 psf may be used for mat foundations founded on compacted soil. The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. The mat foundation dimensions, and reinforcement should be based on structural design. For design purposes, the self-weight of the mat foundation can be negligible.

9.3 Lateral Earth Pressures and Resistance to Lateral Loads

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

9.3.1 Active Earth Pressures

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

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

Loading Conditions	Lateral Earth Pressure (psf)
Active earth conditions (wall is free to deflect at least 0.001 radian)	40
At-rest (wall is restrained)	60

These pressures assume no surcharge and no hydrostatic pressure. If water pressure is allowed to build up behind the walls, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the walls. Specific surcharge load is not known at this time. If a uniform surcharge load is applied on a structure or wall, the lateral earth pressure due to surcharge load should be taken conservatively as 50 psf or uniform surcharge applied to the wall backfill surface within the limits of the active failure wedge multiplying by the lateral earth pressure coefficients.

9.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 275 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 3,000 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 slab.

9.4 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 8.4 *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 (150 kcf) of the supporting materials and should be designed by a structural engineer. Slab should be monolithically constructed with the footings and grade beams.

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

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

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

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

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

9.5 Settlement

The total settlement of mat foundation from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizonal distance of 30 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.

9.6 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. 8, Soil Parameters for Pipe Design

Soil Parameters	Value
Average compacted fill total unit weight (assuming 92% relative compaction), γ (pcf)	127.0
Angle of internal friction of soils, ϕ	32
Soil cohesion, c (psf)	50
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)	3,000
Coefficient of passive earth pressure, Kp	3.25
Coefficient of active earth pressure, Ka	0.31
Modulus of Soil Reaction E’ (psi)	1,500

9.7 Bearing Pressure for Anchor and Thrust Blocks

An allowable net bearing pressure presented in Table No. 8, *Soil Parameters for Pipe Design* may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 18 inches wide.

If normal code requirements are applied for design, the above recommended bearing capacity and passive resistances may be increased by 33 percent for short duration loading such as seismic or wind loading.

9.8 Soil Corrosivity

The results of chemical testing of 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 general discussion pertaining to soil corrosivity are presented below.

The sulfate contents of the sampled soil correspond to American Concrete Institute (ACI) exposure category S0 for the sulfate concentration (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 will be exposed to moisture from precipitation and irrigation. Based on the project location and the results of chloride testing of the 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 Keegan Labs LLC, 2026, the following table provides general guidelines of soil corrosion based on electrical resistivity.

Table No. 9a, Correlation Between Resistivity and Corrosion

Electrical Resistivity of Soil	
Soil Resistivity (ohm-cm) per Caltrans CT 643	**Corrosivity Category**
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severe Corrosive

Table No. 9b, Correlation Between Sulfates and Corrosion

Sulfate in Soil per ACI 318	
Sulfate Level (mg/kg)	**Classification**
Less than 1,000 (<0.1%)	Negligible
1,000 to 2,000 (0.1 to 0.2%)	Moderate
2,000 to 20,000 (0.2 to 2.0%)	Severe
Over 20,000 (>2.0%)	Very Severe

Table No. 9c, Correlation Between Chlorides and Corrosion

Chloride in Soil	
Concentration (mg/kg)	**Classification**
ND to 100	Negligible
100 to 350	Little Added Concern
350 to 500	Potential Concern
Over 500	Definite Concern

Table No. 9d, Correlation Between pH and Corrosion

pH	
pH Range	**Description**
<3.5	Ultra Acidic
3.5 to 4.4	Extremely Acidic
4.5 to 5.0	Very Strongly Acidic
5.1 to 5.5	Strongly Acidic*
5.6 to 6.0	Moderately Acidic
6.1 to 6.5	Slightly Acidic
6.6 to 7.3	Neutral
7.4 to 7.8	Slightly Alkaline
7.9 to 8.4	Moderately Alkaline
8.5 to 9.0	Strongly Alkaline
>9.0	Very Strongly Alkaline

**This range and below should have total acidity testing performed to evaluate corrosivity*

The measured value of the minimum electrical resistivity of the sample when saturated was29,100 Ohm-cm. This indicates that the soil tested on the site is Mildly Corrosive to ferrous metals in contact with the soils (Keegan, 2026). 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.

9.9 Pavement Recommendations

One soil sample was tested to determine the R-value of the subgrade soils. Based on laboratory testing, the R-value was 50. 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, 2023), Chapter 630 with a safety factor of 0.2 for asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table below.

Table No. 10, Recommended Preliminary Pavement Sections

R-value 50	Traffic Index (TI)	Pavement Section		
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
	5	3.0	3.0	4.5
	6	3.5	3.5	6.0
	7	4.0	4.5	7.0
	8	4.5	6.0	8.0

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

Prior to placement of aggregate base or full asphalt concrete, at least the upper 12 inches of subgrade soils should be scarified, moisture-conditioned if necessary, and recompacted to at least 95 percent of the laboratory maximum dry density as defined by ASTM Standard D1557 test method.

Base materials should conform Section 200-2.2,"Crushed Aggregate Base," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2021) and should be placed in accordance with Section 301-2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302-5 of the SSPWC.

10.0 CONSTRUCTION RECOMMENDATIONS

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

10.1 General

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

Sloped excavations may not be feasible in locations adjacent to existing utilities (if any). Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing utilities 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.

10.2 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 Sand with Silt (SP-SM)	C	0-3	Vertical
Silty Sand (SM) and Sand with Silt (SP-SM)	C	3-5	1.1:1
Silty Sand (SM) and Sand with Silt (SP-SM)	C	5-10	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 trenches should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring can be provided if necessary.

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

10.3 Shoring Design

Temporary shoring will be required where open sloped excavations will not be feasible due to unstable soils or due to nearby existing structures or facilities. Temporary shoring may consist of conventional soldier piles and lagging or sheet piles or any piles selected by contractor. 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.

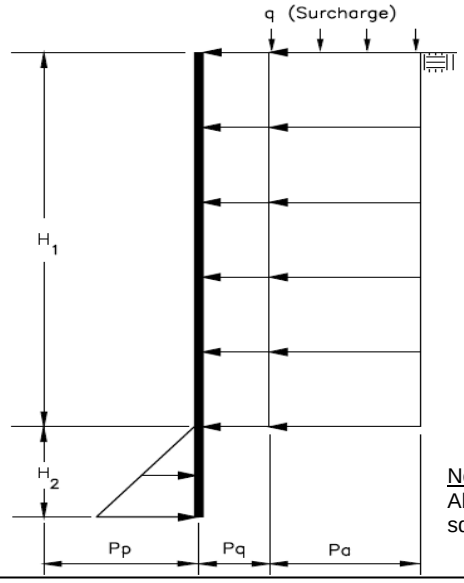
Table No. 12, Lateral Earth Pressures for Temporary Shoring

Lateral Resistance Soil Parameters*	Value
Active Earth Pressure (Braced Shoring) (psf) (A)	28
Active Earth Pressure (Cantilever Shoring) (psf) (B)	39
At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)	60
Passive earth pressure (psf per foot of depth) (D)	275
Maximum allowable bearing pressure against native soils (psf) (E)	3,000
Coefficient of friction between sheet pile and native soils, fs (F)	0.25

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

Restrained (braced) shoring systems should be designed based on Figure No. 4, *Lateral Earth Pressures for Temporary Braced Excavation* to support a uniform rectangular lateral earth pressure.

Figure No. 4, 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

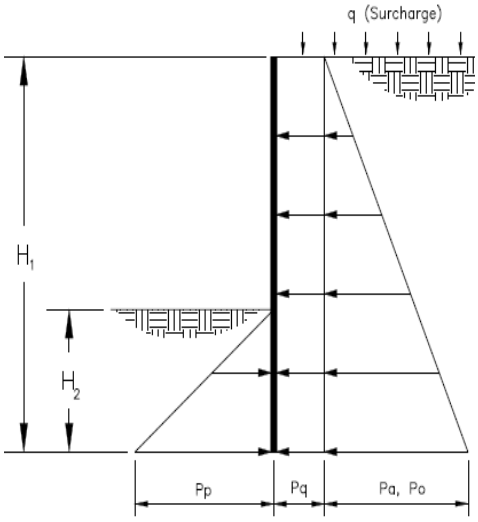
Pp = (D) H_2 ≤ (E) psf - passive earth pressure (on native soils)

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. 5, *Lateral Earth Pressures on Temporary Cantilever Wall*.

Figure No. 5, Lateral Earth Pressures on Temporary Cantilever Wall



Total Earth Pressure, P

P = Pq + Pa, Po

Pq = 0.5q - incremental surcharge pressure

Pa = (B)H_1 - active earth pressure (Un-restrained)

Po = (C)H_1 - at rest earth pressure (Restrained)

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).

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 unless the surface is confined by a pavement or slab.

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 for the proposed pipeline should not extend below a 1:1 horizontal: vertical (H: V) 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 support of the existing utilities or streets.

If the excavation extends below a 1:1 (H: V) 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.

11.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

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

12.0 CLOSURE

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

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

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

13.0 REFERENCES

AMERICAN CONCRETE INSTITUTE (ACI), 2022, Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary, 2022.

AMERICAN SOCIETY OF CIVIL ENGINEERS (ASCE), 2022, Minimum Design Loads for Buildings and Other Structures, SEI/ASCE Standard No. 7-22, dated 2022.

AMERICAN SOCIETY OF CIVIL ENGINEER'S (ASCE), 2026, ASCEE Hazard Tool (https://ascehazardtool.org) accessed in April 2026

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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 approximate locations shown on the exhibit provided by Shane Bloomfield with Webb Associates. The boring locations were established in the field using approximate distances from existing local structures and features as a guide and should be considered accurate only to the degree implied by the method used to locate them. The locations were cleared from conflict with existing utilities by Underground Service Alert and staff from the East Valley Water District.

Two exploratory borings (BH-01 and BH-02) were drilled on March 25, 2026, to investigate the subsurface conditions. The borings were drilled to a depth of 5.0 feet below ground surface (bgs). The details of the boring are shown below.

Due to the amount of gravel, cobble, and boulders drilling could not be advanced beyond 5.0 feet bgs, so the borings where terminated before the planed depths where reached.

Table No. A-1, Summary of Borings

Boring No.	Boring Depth (ft, bgs)		Groundwater Depth (ft, bgs)	Date Completed
	Proposed	Completed		
BH-01 (Equipment Building)	50.0	5.0	Not Encountered	3/25/2026
BH-02 (Pipeline)	20.0	5.0	Not Encountered	3/25/2026

The borings were advanced using a track-mounted drill rig equipped with 8-inch diameter hollow-stem augers for sampling of soils. 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 and disturbed 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 log. Samples were retained in brass rings (2.4 inches inside diameter and 1.0 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk samples of typical soil types were also obtained in plastic bags.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between drive samples are indicated on the log 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 and compacted by pushing down with an auger using the drill rig weight. Where asphalt was penetrated, the pavement section was replaced with quickset concrete that was dyed black to match the existing surface. If construction is delayed, the surface of the borings may settle over time. We recommend the owner monitors the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing Nos. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For the log of borings, see Drawing No. A-2 and A-3, *Log of Borings*. All elevations are based on Google Earth.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

FIELD AND LABORATORY TESTS

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

BORING LOG SYMBOLS

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

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For: Albert A. Webb Associates

Project No. **25-81-144-01**

Drawing No. **A-1a**

Project ID: 25-81-144-01.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 (Equipment Buliding)

Dates Drilled: 3/25/2026 Logged by: Jason Bragg Checked By: Catherine Nelson

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

Ground Surface Elevation (ft): 1684 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 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 **SAND with SILT (SP-SM):** fine to coarse-grained, mostly gravel, cobbles, and boulders up to 4.0 feet maximum dimension, very dense, moist, brown.	■	▨	5/7/50-1"	6	115	EI, CR, WA, CP DS
5		End of Boring at 5.0 feet, ended early due to refusal on boulders. Groundwater not encountered. Backfilled with soil cuttings and covered on 3/25/2026.						

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Drawing No. A-2

Log of Boring No. BH-02 (Pipeline)

Dates Drilled: 3/25/2026 Logged by: Jason Bragg Checked By: Catherine Nelson

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

Ground Surface Elevation (ft): 1689 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring 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.						
		3' ASPHALT CONCRETE/ 6' AGGRAGATE BASE						
		ALLUVIUM **SILTY SAND (SM):** fine to coarse-grained, mostly gravel, cobbles, and boulders up to 4.0 feet maximum dimension, very dense, moist, brown.						SE, R, WA
5		End of Boring at 5.0 feet, ended early due to refusal on boulders. Groundwater not encountered. Backfilled with soil cuttings mixed with cement, compacted useing an auger with the weight of the drill rig, and patched with cemend dyed black to match the existing surface on 3/25/2026.						

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Project No. 25-81-144-01

Drawing No. A-3

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 Log of Boring, 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 D2216-19 and ASTM D7263 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Log of Boring in Appendix A, *Field Exploration.*

Expansion Index (EI)

One sample was tested to evaluate the expansion potential in accordance with ASTM Standard D4829-21. The test result is presented in the following table.

Table No. B-1, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-01	0.0 - 5.0	Sand with Silt (SP-SM)	0	Very Low

Sand Equivalent (SE)

One representative soil sample was tested in accordance with the ASTM Standard D2419-22 test method to determine the sand equivalent. The test result is presented in the following table.

Table No. B-2, Sand Equivalent Test Results

Boring No. Street	Depth (feet)	Soil Description	Sand Equivalent
BH-02	0.3 - 5.0	Silty Sand (SM)	48

Soil Corrosivity Test (CR)

One representative soil sample was tested to determine minimum electrical resistivity (wet condition), pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of this test was to determine the corrosion potential of site soils when placed in contact with common construction materials. The test was

performed by Keegan Labs in accordance with Caltrans Tests 643, 422 and 417. Test result is presented in the following table.

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

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-01	0-5	8.3	13	14	29,100

R-value (R)

One representative bulk soil samples were tested in accordance with ASTM Standard D2844_D2844M-18E01 for resistance value (R-value). The tests provide a relative measure of soil strength for use in pavement design. The test results are presented in table No. B-3, *R-Value Test Result* below.

Table No. B-4, R-Value Test Results

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-02	0.3-5.0	Silty Sand (SM)	50

Percent Passing No. 200 Sieve (WA)

Two percent passing (No. 200 sieve) tests were performed on select samples in accordance with ASTM Standard D1140-25. The test results of the percent passing #200 sieves are presented in the following table.

Table No. B-5, Percentage Passing No. 200 Sieve Results

Boring No.	Depth (feet)	Description	Passing #200 Sieve (%)
BH-01	0.0-5.0	Sand with Silt (SP-SM)	9.5
BH-02	0.3-5.0	Silty Sand (SM)	12.4

Maximum Density and Optimum Moisture Content (CP)

The laboratory maximum dry density-optimum moisture content relationship test was performed on one representative bulk soil sample. The test was conducted in accordance with the ASTM Standard D1557-12R21 test method. The test results are presented in Drawing No. B-1, *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-01	0.0-5.0	Sand with Silt (SP-SM)	9.0	129.0

Direct Shear (DS)

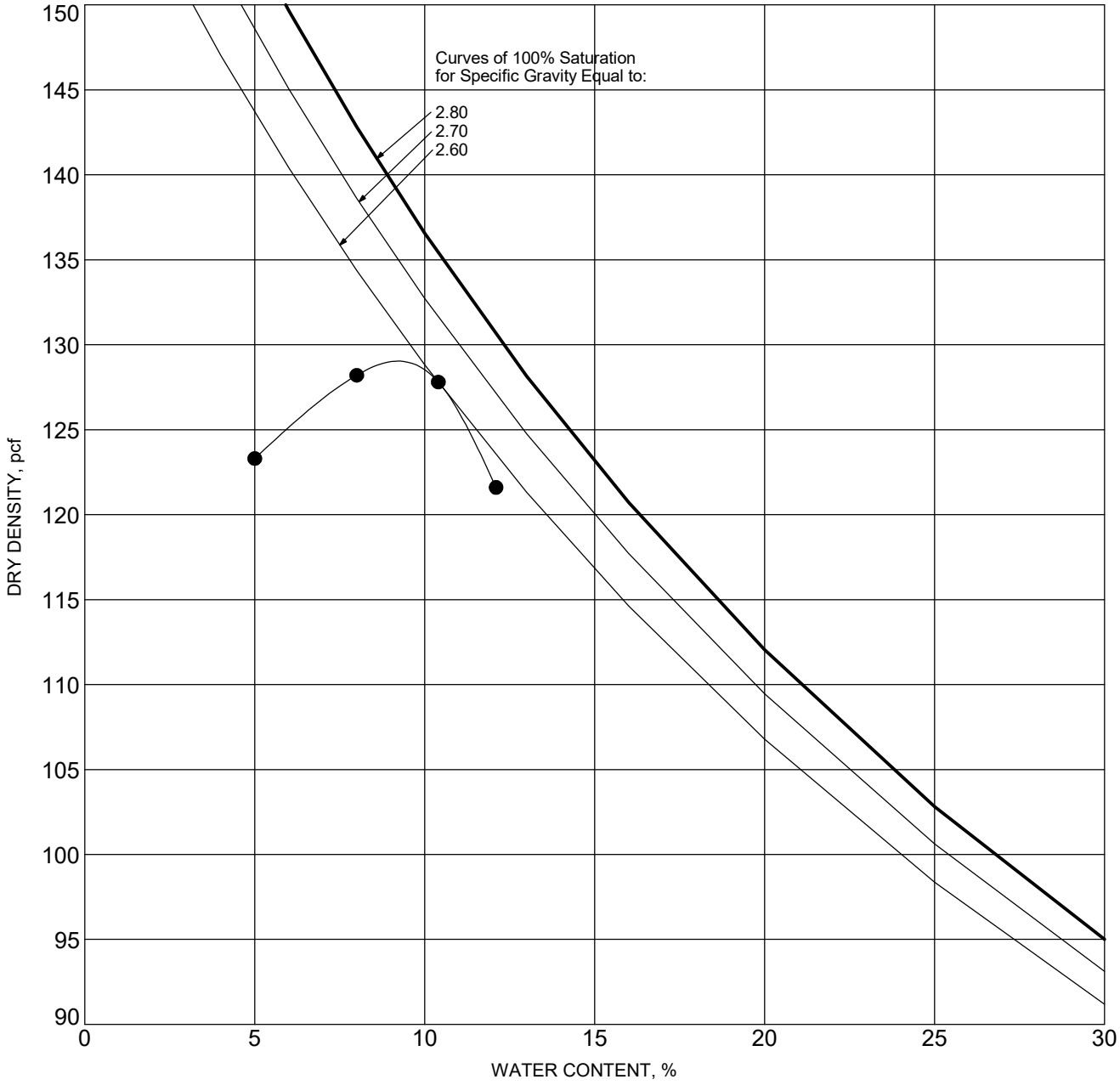
One direct shear test was performed on relatively undisturbed samples in accordance with ASTM D3080/D3080M-23. For the test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated 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 Drawing No. B-2*, 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-01	2.5-4.0	Sand with Silt (SP-SM)	32	140

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.



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**0.0-5.0**	**Sand with Silt (SP-SM), Brown**	**D1557 B**	**9**	**129**

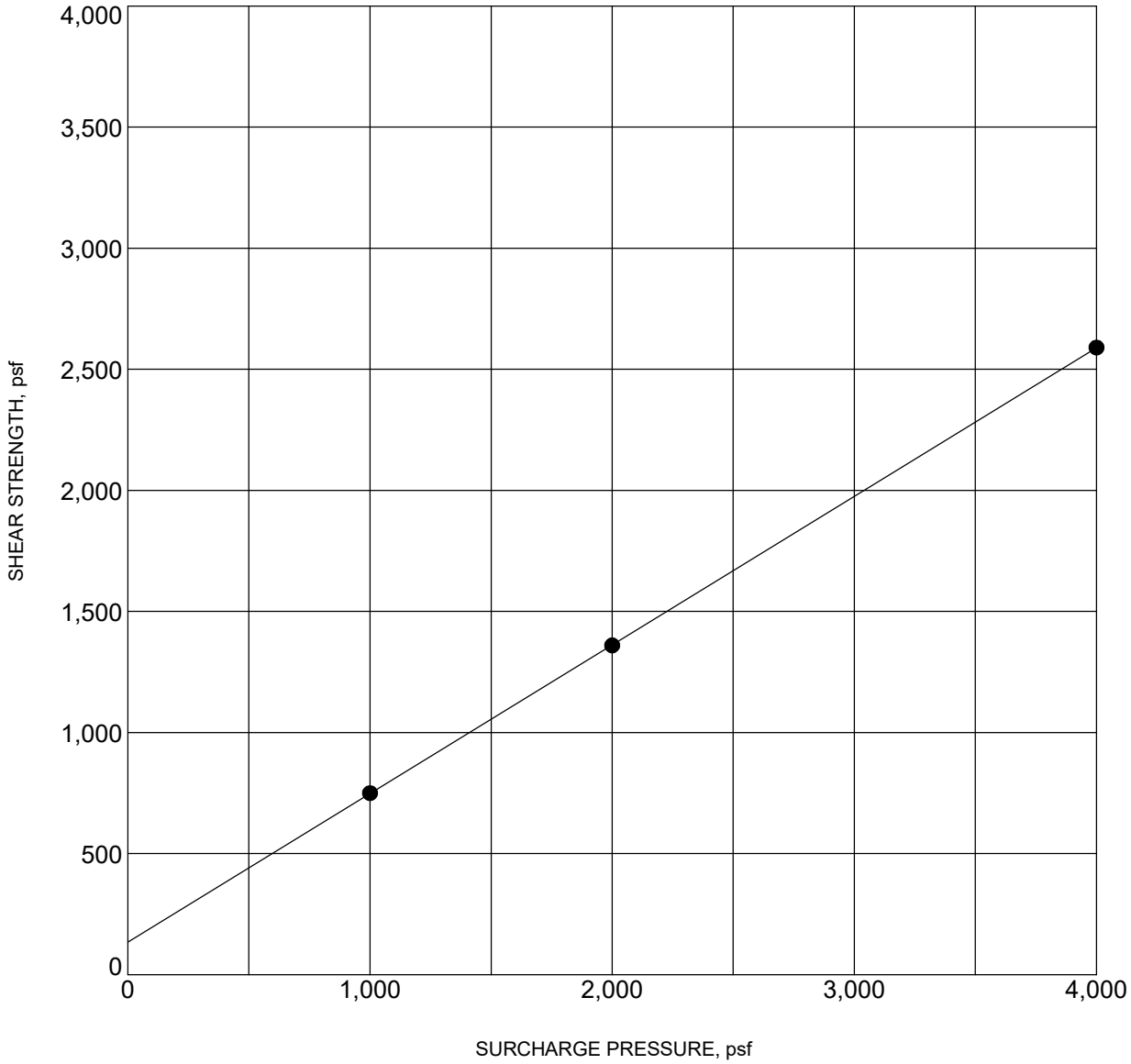
MOISTURE-DENSITY RELATIONSHIP RESULTS

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Drawing No. **B-1**



BORING NO.	:	**BH-01**	DEPTH (ft)	:	**2.5 - 3.6**
DESCRIPTION	:	**Sand with Silt (SP-SM)**			
COHESION (psf)	:	**140**	FRICTION ANGLE (degrees):		**32**
MOISTURE CONTENT (%)	:	**3.0**	DRY DENSITY (pcf)	:	**123.0**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

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Drawing No. **B-2**