APPENDIX A

Geotechnical Investigation Report – Booster Pump Station, Converse Consultants

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

GEOTECHNICAL INVESTIGATION REPORT

STEEPLECHASE AND KALMIA BOOSTER PUMP STATION (BPS) REPLACEMENT

25565 KALMIA AVENUE
CITY OF MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 20-81-256-02

Prepared For:

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Presented By:

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March 2, 2022

March 2, 2022

Mr. Jerry Pascoe, PE, GE
Principal Engineer
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Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
Converse Project No. 20-81-256-02

Dear Mr. Pascoe:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of the Steeplechase and Kalmia Booster Pump Station (BPS) Replacement Project, located in the City of Moreno Valley, Riverside County, California. This report was prepared in accordance with our proposal dated October 12, 2020, and your Agreement Between Consultant and Subconsultant dated May 4, 2021.

Based upon our field investigation, laboratory data, and analyses, the proposed project 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 Gannett Fleming, Inc., and the Eastern Municipal Water District (EMWD). Should you have any questions, please do not hesitate to contact us at 909-796-0544.

CONVERSE CONSULTANTS

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

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HSQ/SR/CN/kvg

PROFESSIONAL CERTIFICATION

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

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

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for the Steeplechase and Kalmia Booster Pump Station (BPS) Replacement Project, located in the City of Moreno Valley, Riverside County, California. The project location is shown in Figure No. 1, *Approximate Project Location Map*.

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

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

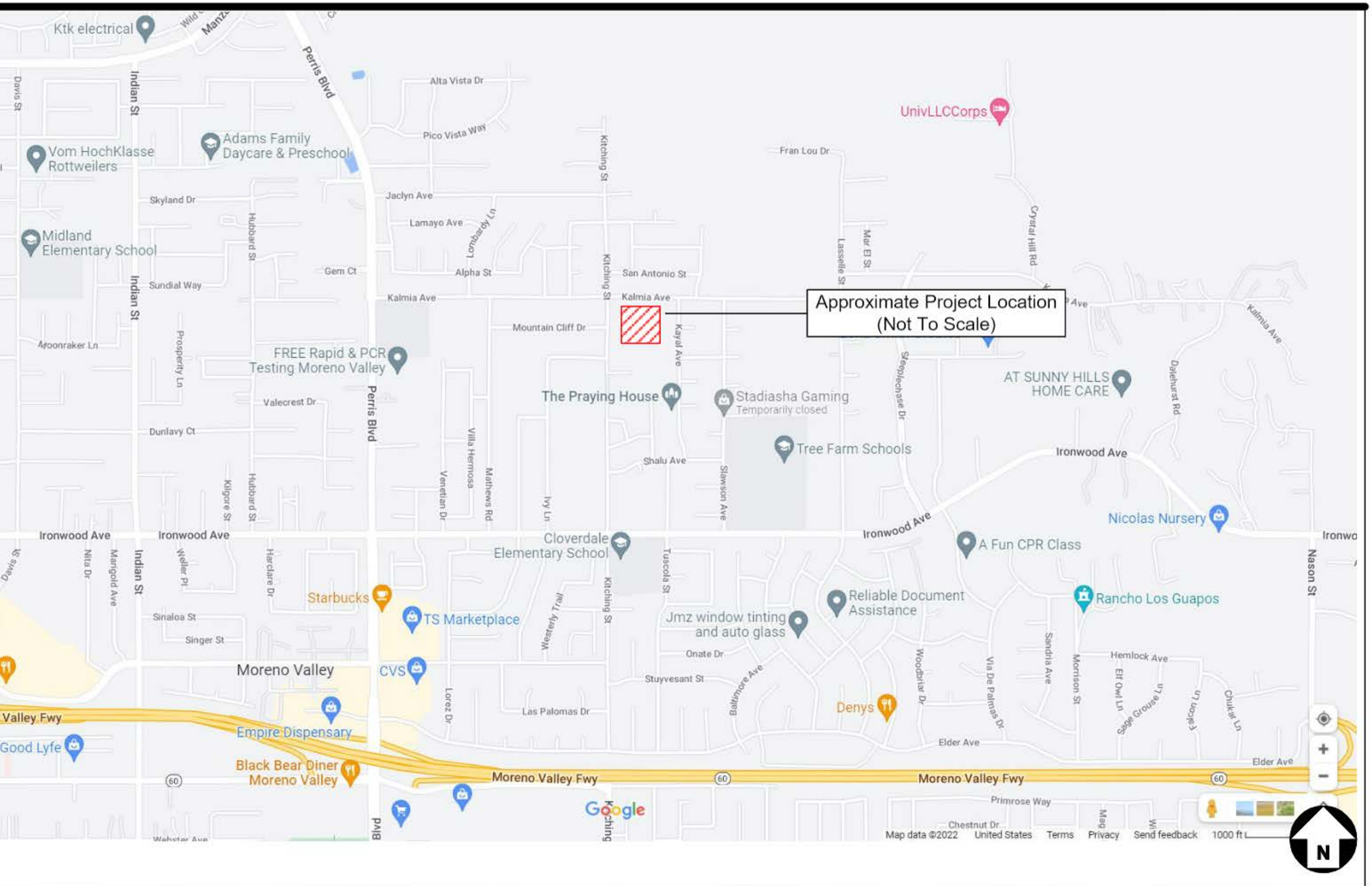
2.0 PROJECT DESCRIPTION

Based on the information provided by Gannett Fleming, Inc., the project will consist of the construction of a new BPS. The BPS will likely be a 40’ x 20’ masonry block wall building with slab on grade. It will be founded on shallow footings. Associated with the BPS there will be yard piping. Depth to pipe invert will be about 5 feet below existing ground surface.

3.0 SITE DESCRIPTION

The site is located at 25565 Kalmia Avenue in the City of Moreno Valley, Riverside County, California. The site is bounded by Kalmia Avenue on the north, residential properties to the east, west and south (separated by a standard chain link from all the sides). The site is presently occupied by a water reservoir. The property can be divided into two sections: front and back. The description of the site is as follows.

- Front: Landscaped lot fully secured on all sides by a standard 6-foot chain link fence, with angled barbed wire top. A paved access road surrounds the existing tank. Several above ground appurtenances exist around the premises. A 7-foot-high cinder block wall surrounding existing cell phone signal tower to the southwest of existing tank. Steep slopes surround the north and east of the property.
- Back: Undeveloped, graded dirt lot fully secured on all sides by a standard 6-foot-high chain link fence with angled barbed wire top. Several below ground appurtenances traverse the subsurface.
- Access to entire project site off Kalmia Avenue via locked gate (front). Access to future BPS location through secondary locked gates (back).
- No overhead utilities. Several large trees are present in front, no trees in back.
- Photograph No. 1 and 2 depict the present site conditions.



ject: Steeplechase and Kalmia Booster Pump Station (BPS) Replacement

cation: City of Moreno Valley, Riverside County, California

r: Gannett Fleming, Inc.

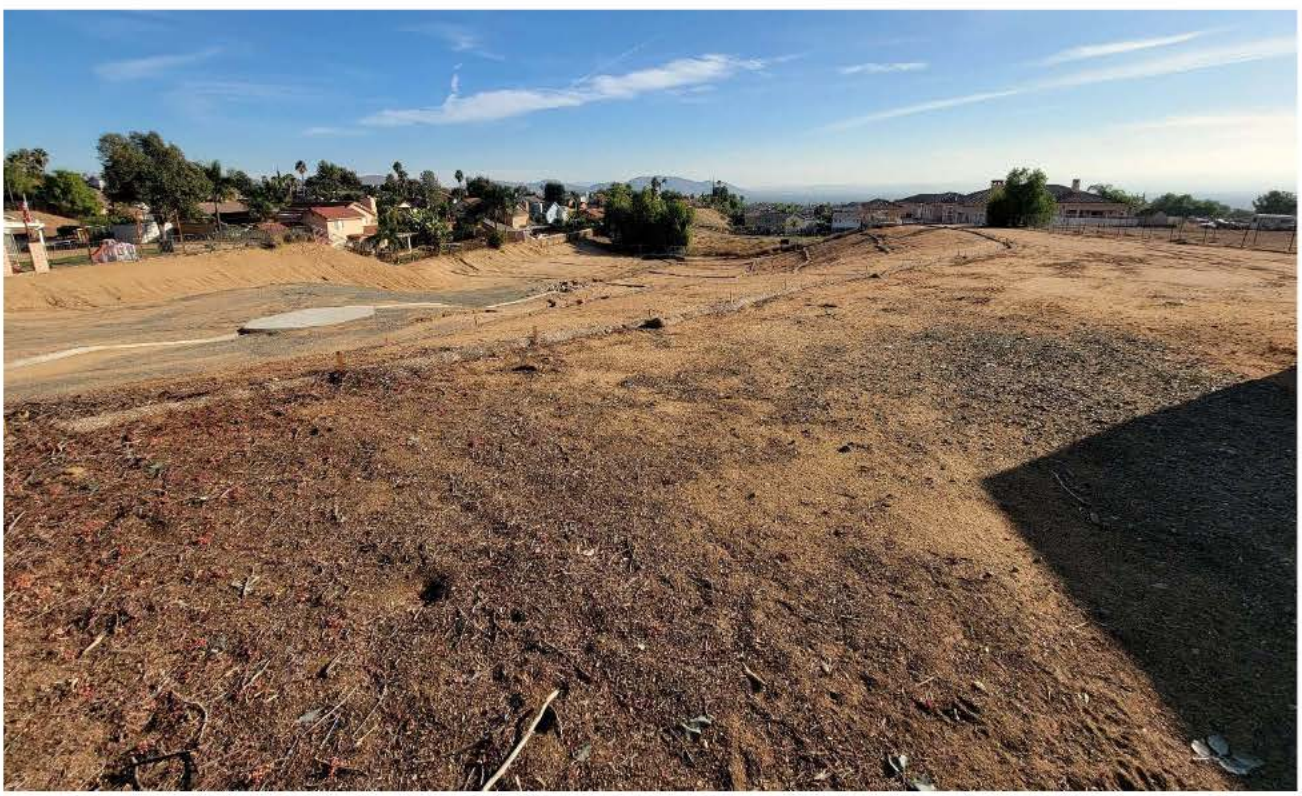
Approximate Project Location Map

Project No.
20-81-256-02

Converse Consultants

Figure No.
1

Photograph No. 1: Entrance to existing tank site, facing south.



Photograph No. 2: Rear vacant lot, facing south.

4.0 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.

4.1 Project Set-up

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

- Conducted a field reconnaissance and staked/marked the borings at locations selected by Ms. Carolina Cubides with Gannett Fleming, Inc. such that drill rig access to all the locations was available.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring locations of any conflict with existing underground utilities.
- Engaged a California-licensed driller to drill exploratory borings.

4.2 Subsurface Exploration

Three exploratory borings (BH-01 through BH-03) were drilled on December 28, 2021, to investigate the subsurface conditions. The borings were drilled to depths between 15.5 feet and 51.5 feet below ground surface (bgs).

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*.

4.3 Laboratory Testing

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

- *In*-situ moisture contents and dry densities (ASTM D2216 and ASTM D2937)
- 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)
- Consolidation (ASTM D2435)

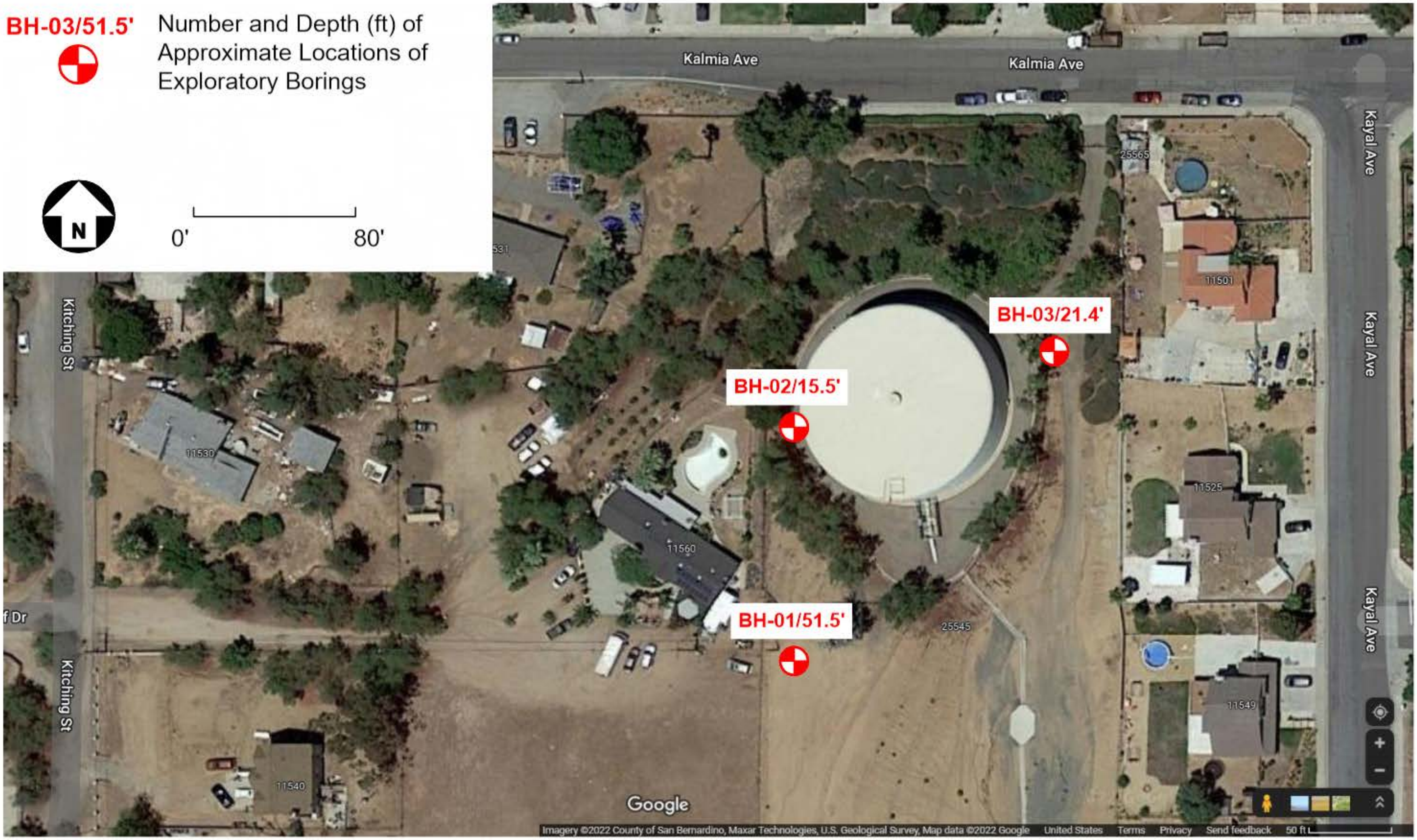
For *in-situ* moisture and dry density data, see the Logs of Borings in Appendix A, *Field Exploration*. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program.*

EXPLANATION

BH-03/51.5' Number and Depth (ft) of Approximate Locations of Exploratory Borings

N

0' 80'



ject: Steeplechase and Kalmia Booster Pump Station (BPS) Replacement

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Approximate Boring Locations Map

Project No. 20-81-256-02

Converse Consultants

Figure No. 2

4.4 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.

5.0 LABORATORY TEST RESULTS

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

5.1 Physical Testing

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.

- *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 upper 10 feet soils of the site ranged from 111 to 124 pcf with moisture contents ranging from 2 to 13 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Collapse Potential – The collapse potential of three 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 of 1.6, 1.9 and 0.3 percent, indicating slight collapse potential.
- Grain Size Analysis – Three representative soil samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – The moisture-density relationship of a representative soil sample was tested in according to ASTM Standard D1557 and the results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program*. The laboratory maximum dry density was 133.0 pounds per cubic feet (pcf) with optimum moisture content of 7.5 percent.
- Direct Shear – One direct shear test was performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The results of the direct shear tests are presented in Drawing No. B-3, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.
- Consolidation – One consolidation test was performed on relatively undisturbed samples of the site soils, in accordance with ASTM Standard D2435. The test results are shown on Drawing No. B-4, *Consolidation Test Results,* in Appendix B, *Laboratory Testing Program*.

5.2 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 purposes of these tests were to determine the corrosion potential of site soils when placed in contact with common pipe materials. These tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California 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 16 ppm (0.0016 percent by weight).
- The chloride concentration of the sample was 26 ppm.
- The minimum electrical resistivity of the sample when saturated was 2,717 ohm-cm.

6.0 SITE CONDITIONS

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

6.1 Subsurface Profile

Based on the exploratory borings and laboratory test results, the subsurface soils at the site encountered in the borings at various depths consists primarily of a mixture of sand, silt, trace clay, and gravel. Scattered to few gravel up to 2 inches in maximum dimension was observed in all the borings. The soils were slightly indurated below a depth of 25 feet.

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

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

6.2 Groundwater

Groundwater was not encountered in any of the borings to the maximum explored depth of 51.5 feet bgs. The coordinates of 33.953058N, 117.215849W were used to research and identify comparable groundwater levels.

The State Water Resources Control Board's GeoTracker Database (SWRCB, 2021) was reviewed to establish current and historic groundwater levels. Within a 1.0-mile radius of the centralized coordinates, no site with groundwater data was identified.

The National Water Information System (USGS, 2021) was reviewed to establish current and historic groundwater levels. Within a 1.0-mile radius of the centralized coordinates, no site with groundwater data was identified.

The California Department of Water Resources database (DWR, 2021) was reviewed to establish current and historic groundwater levels. Within a 1.0-mile radius of the centralized coordinates, no site with groundwater data was identified.

Based on available data, current groundwater is expected to be deeper than about 51.5 feet bgs. Groundwater is not expected to be encountered during the construction of the project. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation.

6.3 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade. Depending on the extent and location below finish subgrade, expansive soils can have a detrimental effect on structures. Expansion index of the site soils was not determined. However, based on the soil type and experience with similar projects, the expansion index of site soil should be less than 20, corresponds to very low expansion potential. This should be verified during the site grading.

6.4 Collapse Potential

Soil deposits subjected to collapse/hydro-consolidation generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon 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. 1, *Collapse Potential Values*.

Table No. 1, 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 6.6, 1.9 and 0.3 percent), a slight problem is anticipated at the site. Collapse potential distress is typically considered a concern when collapse potential is over 2% (LA County, 2013).

6.5 Excavatability

The subsurface materials at the site are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. <u>However, excavation will be difficult if concentration of gravel is encountered, as well as very dense soils below a depth of approximately 5 feet to 10 feet.</u>

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

6.6 Subsurface Variations

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

7.0 ENGINEERING GEOLOGY

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

7.1 Regional Geology

The project 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, Elsinore, and San Andreas fault zones (CGS, 2007), all of which have been known to be active during Quaternary time.

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

The project is located within the north-central portion of the Perris Block region of the Peninsular Ranges province. The Perris Block is a relatively stable structural block bounded by the active Elsinore and San Jacinto fault zones to the west and east, and the Chino and Temecula basins to the north and south, respectively. The Perris Block has low relief and is roughly rectangular in shape.

7.2 Local Geology

- The project site and alignments are anticipated to be primarily underlain by middle to early Pleistocene, very old alluvial fan deposits (Qvof). These deposits are mostly moderately to well consolidated silt, sand, gravel, and conglomerate.
- Tonalite granite (bedrock) is exposed approximately 1,500 feet northwest of the project site and is potentially present at shallow depths nearby.

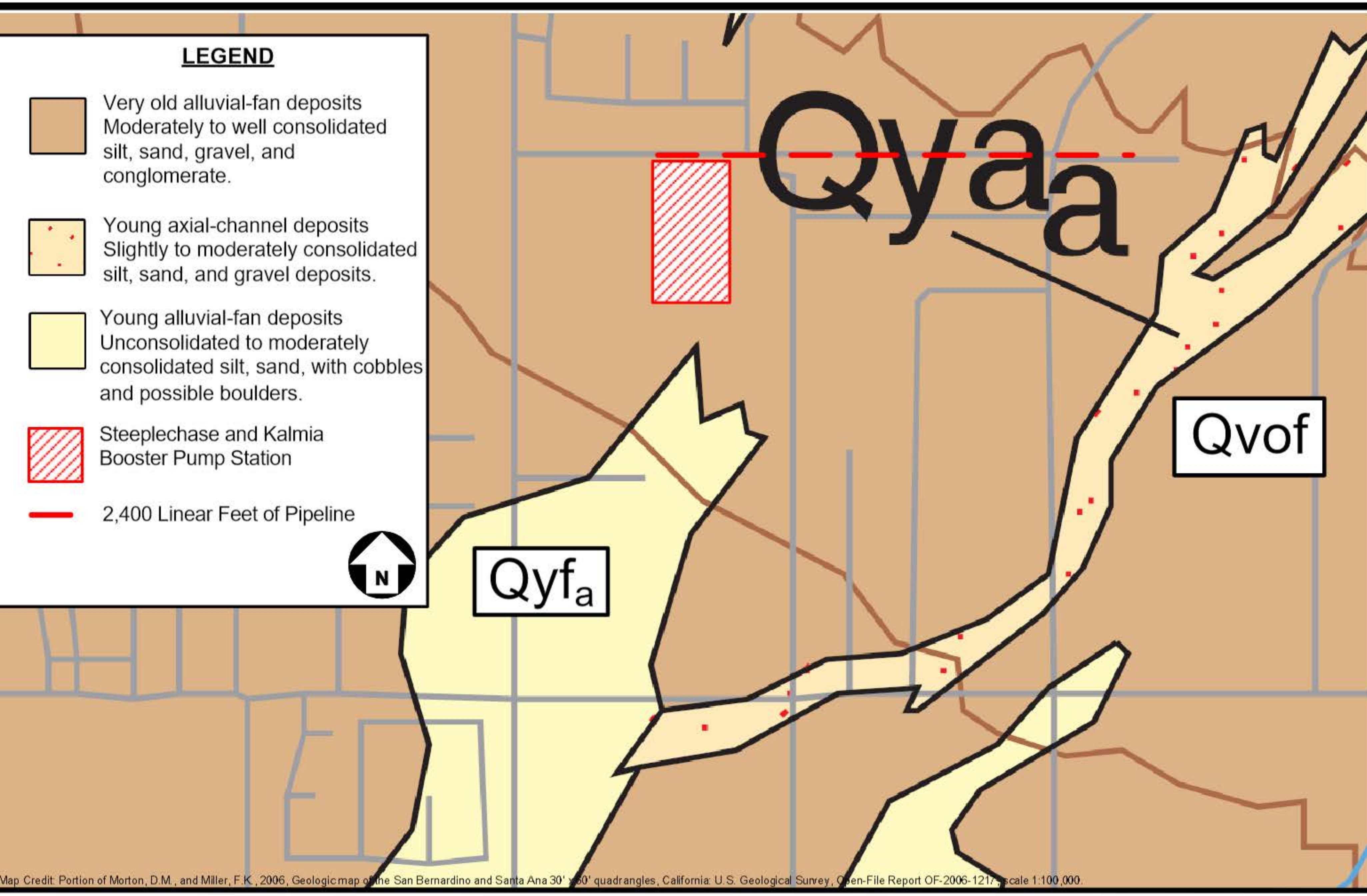
The site and surrounding local geology are shown on Figure 3, *Geological Reference Map*.

8.0 FAULTING AND SEISMICITY

Nearby active faults, seismicity, and their impact on the project site and alignment are discussed in the following sections.

8.1 Faulting

No portion of the project site is located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2021). The nearest



Project: Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
Location: City of Moreno Valley, Riverside County, California

Geological Reference Map

Project No. 20-81-256-02

For: Gannett Fleming, Inc.

Converse Consultants

Figure No. 3

active fault zone is a Riverside County Fault Zone located approximately 1.5 miles to the northeast and the San Jacinto Fault Zone located approximately 2.3 miles northeast of Kalmia Avenue.

8.2 CBC Seismic Design Parameters

CBC seismic design parameters based on the 2019 California Building Code (CBC, 2019), ASCE 7-16 and site coordinates 33.953058N latitude and 117.215849W longitude are provided in the following table. These parameters were determined using the ATC Hazards online calculator.

Table No. 2, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	33.953058N, 117.215849W
Site Class	D
Risk Category	III
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	2.029g
Mapped 1-second Spectral Response Acceleration, S_1	0.804g
Site Coefficient (from Table 11.4-1), F_a	1.0
Site Coefficient (from Table 11.4-2), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.029g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.367g
Design Spectral Response Acceleration for short period S_{DS}	1.353g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.911g
Site Modified Maximum Peak Ground Acceleration, PGA_M	0.942g

8.3 Secondary Effects of Seismic Activity

Generally, in addition to ground shaking, effects of seismic activity on a structure/pipeline may include surface fault rupture, soil liquefaction and dry seismic settlement, landslides and lateral spreading, and flooding due to earthquake-induced dam failure. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture: No portion of the project site/alignment is located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2021). The potential for surface rupture resulting from the movement of nearby or distant faults is not known with certainty but is considered very low.

Dynamic Settlement (Liquefaction and Dry Seismic Settlement): Liquefaction is defined as the phenomenon in which a soil mass within about 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 occurs during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows.

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

Based on a review of state and county data, the risk of liquefaction is considered to be low to moderate at this project site. Based on a site-specific settlement analysis presented in Appendix C, *Liquefaction and Settlement Analysis*, we estimate that the liquefaction induced settlement of the site is negligible.

Landslides and Lateral Spreading: Seismically induced landslides and other slope failures are common occurrences during or after earthquakes in areas of significant relief. No portion of the project site is located within a currently designated State of California or Riverside County Landslide Zone (CGS, 2007; Riverside County, 2021). Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. The potential for landslides or lateral spreading at this project site is considered very low.

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

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Due to the distance to large bodies of water, the site is not at risk of seiching.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is not located within a designated dam inundation area (DSOD, 2021).

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. All debris and deleterious material should be removed from the site.

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 utilities (if any).

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

Table No. 3, Overexcavation Depths

Structure/Pavement	Minimum Excavation Depth
Footings (building)	2 feet below footing bottom or 5 feet below existing ground surface, whichever is deeper
Slab-on-grade	15 inches below slab bottom or 2 feet below existing ground surface, whichever is deeper
Walls (footings if any)	15 inches below footings bottom or 2 feet below existing ground surface, whichever is deeper
Pavements	12 inches below finish grade

The overexcavation below the footings, slab and pavements should be uniform. The overexcavation should extend to at least 3 feet beyond the footprint of the building footings, 2 feet beyond the slab and wall footings and at least 1 foot beyond the edge of the pavements. The overexcavation bottom should be scarified and compacted as described in Section 9.4, *Compacted Fill Placement*.

9.3 Engineered/Structural Fill

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

- No particles larger than 3 inches in largest dimension.
- Rocks larger than 1 inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 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 investigation and laboratory testing results, on-site soils may be suitable as structural/engineered fill materials.

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

9.4 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 6 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

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

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein. At least the upper 1 foot of subgrade soils underneath pavements intended to support vehicle loads should be scarified, moisture conditioned, and compacted to at least 95 percent of the laboratory maximum dry density.

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

9.5 Shrinkage and Subsidence

The volume of excavated and recompacted soils may be expected to decrease as a result of grading. The shrinkage would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. For preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- An average shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) of 5 to 8 percent can be used for the upper 5 feet of soils for preliminary earthwork planning.
- Subsidence (defined as the settlement of native materials from the equipment load applied during grading) would depend on the construction methods including type of equipment utilized. For estimation purposes, ground subsidence may be taken as 0.1 to 0.15 feet.

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

9.6 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 in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

9.7 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.

9.7.1 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 an adequate bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

9.7.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Pipe bedding should follow EMWD Standards. If additional recommendations beyond EMWD 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 may not be considered in selecting the gradation of any imported bedding material.

9.7.3 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 oversize particles and deleterious matter may be used to backfill the trench zone. Trench backfill should follow EMWD Standards. For trenching excavation into bedrock, see Section 6.5, *Excavatability*. Based on the pipe profile (cover 5 feet bgs), trenching recommendations for pipelines below groundwater is not required. If additional recommendations beyond EMWD Standards 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.

10.0 DESIGN RECOMMENDATIONS

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

10.1 Shallow Foundation Design Parameters

The proposed BPS pads and walls (if any) may be supported on continuous and/or isolated spread footings. The design of the shallow foundations should be based on the recommended parameters presented in the table below.

Table No. 4, Recommended Foundation Parameters

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

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 lateral earth pressures are presented in the following table.

Table No. 5, 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.

A uniform lateral pressure of 100 psf should be considered to account for normal vehicular and construction traffic within 10 feet of the structures.

10.2.2 Resistance to Lateral Loads

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 270 psf per foot of depth may be used for the sides of the footing 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 3,000 psf.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.

Due to the low overburden stress of the soil at shallow depth, the upper 1 foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

10.3 Settlement

The total settlement of mat foundation/footing from static structural loads and short-term settlement of properly compacted fill is anticipated to be 0.5 inch or less. The differential settlement resulting from static loads is anticipated to be 0.25 inches or less over a horizontal distance of 40 feet.

Our analysis of the potential dynamic settlement is presented in Appendix C, *Liquefaction and Settlement Analysis*. We estimate that the potential for liquefaction induced settlement and dry seismic settlement for the site is negligible.

10.4 Pipe Design for Underground Utilities

Structural design of pipes requires proper evaluation of all possible loads acting on pipes. The stresses and strains induced on buried pipes 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 the pipe design are provided in Table No. 6, *Soil Parameters for Pipe Design* below.

Where pipes are connecting to rigid structures near, or at its lower levels, and then are subjected to significant loads as the backfill is placed to finish grade, we recommend that provisions be incorporated in the design to provide support of these pipes where they exit the structure. Consideration can be given to flexible connections, concrete slurry support beneath the pipes where they exit the structures, overlaying and supporting the pipes with a few inches of compressible material, (i.e., Styrofoam, or other materials), or other techniques. Automatic shutoffs should be installed to limit the potential leakage in the event of damage in a seismic event.

Table No. 6, Soil Parameters for Pipe Design

Soil Parameters	Value
Average compacted fill total unit weight (assuming 92% relative compaction), γ (pcf)	131.5
Angle of internal friction of soils, ϕ	32
Soil cohesion, c (psf)	0
Coefficient of friction between concrete and native soils, fs	0.35
Coefficient of friction between CML&C steel, PVC or HDPE pipe and native soils, fs	0.25
Bearing pressure against native soils (psf)	2,500
Coefficient of passive earth pressure, Kp	3.25
Coefficient of active earth pressure, Ka	0.31
Modulus of Soil Reaction E' (psi)	1,500

10.5 Soil Corrosivity

The results of chemical testing of one representative soil sample were evaluated for corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program,* and general discussion pertaining to soil corrosivity 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 Romanoff, 1957, the following table provides general guideline of soil corrosion based on electrical resistivity.

Table No. 7, Correlation Between Resistivity and Corrosion

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

The measured value of the minimum electrical resistivity of the sample when saturated was 2,717 Ohm-cm. This indicates that the soil tested of the site is moderately corrosive to ferrous metals in contact with the soils (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site and site soils.

10.6 Flexible Pavement Recommendations

Based on the soil type and experience with similar type of projects, an R-value of 30 was assumed. For pavement design, we have utilized a design subgrade R-value of 30 and design Traffic Indices (TIs) ranging from 5 to 8.

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

Table No. 8, Recommended Preliminary Asphalt Concrete Pavement Sections

Design R-value	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
30	5	3.0	5.0	6.0
	6	3.5	7.0	7.0
	7	4.0	9.0	8.5
	8	5.0	11.0	10.0

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

Prior to placement of aggregate base, at least 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 with the City of Moreno Valley Standards should be placed in accordance with corresponding section of the Public Works Standards "Greenbook" latest version.

Asphaltic concrete materials should conform to the City of Moreno Valley Standards or corresponding section of the Greenbook and should be placed accordingly.

10.7 Rigid Pavement Recommendations

Based on the soil type and experience with similar type of projects, an R-value of 30 to 40 can be assumed. For pavement design, we have utilized a design subgrade R-value of 30 and design Traffic Indices (TIs) ranging from 5 to 8. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table.

Table No. 9, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
30	5.0	6.5
	6.0	7.0
	7.0	7.5
	8.0	7.5

Prior to placement of aggregate base, 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.

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

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

The concrete pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,000 psi. The third point method of testing beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are three inches and one inch, respectively.

Transverse contraction joints should not be spaced more than 15 feet and should be cut to a depth of ¼ the thickness of the slab. Longitudinal joints should not be spaced more than 12 feet apart. A longitudinal joint is not necessary in the pavement adjacent to the curb and gutter section.

Concrete materials should conform to Section 201 of the 2018 Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2018), and concrete pavement should be constructed in accordance with Section 302-6, “Portland Cement Concrete Pavement” of the SSPWC.

10.8 Concrete Flatwork

Except as modified herein, concrete walks, driveways, access ramps, curb and gutters should be constructed in accordance with Section 303-5, *Concrete Curbs, Walks, Gutters, Cross-Gutters, Alley Intersections, Access Ramps, and Driveways*, of the Standard Specifications for Public Works Construction (Public Works Standards, 2018).

The subgrade soils under the above structures should consist of compacted fill placed as described in this report. Prior to placement of concrete, the upper 2 feet of subgrade soils should be moisture conditioned within 3 percent of optimum moisture content for coarse-grained soils and 0 to 2 percent above optimum for fine-grained soils and compacted to at least 95% of the laboratory maximum dry density.

The cement concrete thickness of driveways for passenger vehicles should be at least 4 inches, or as required by the civil or structural engineer. Transverse control joints for driveways should be spaced not more than 10 feet apart. Driveways wider than 12 feet should be provided with a longitudinal control joint.

Concrete walks subjected to pedestrian and bicycle loading should be at least 4 inches thick, or as required by the civil or structural engineer. Transverse joints should be spaced 15 feet or less and should be cut to a depth of one-fourth the slab thickness.

Positive drainage should be provided away from all driveways and sidewalks to prevent seepage of surface and/or subsurface water into the concrete base and/or subgrade.

11.0 CONSTRUCTION RECOMMENDATIONS

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

11.1 General

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

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

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

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

11.2 Temporary Sloped Excavations

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

Table No. 10, Slope Ratios for Temporary Excavations

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

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

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

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

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 Gannett Fleming, Inc. 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. Field 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 it will be 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 site a 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 locations selected by Ms. Carolina Cubides with Gannett Fleming, Inc. The approximate boring locations were established in the field with reference to existing site plans and other visible features. The locations should be considered accurate only to the degree implied by the method used.

Three exploratory borings (BH-01 through BH-03) were drilled on December 28, 2021, to investigate the subsurface conditions. The borings were drilled to depths between 15.5 feet and 51.5 feet below ground surface (bgs).

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

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

Standard Penetration Testing (SPT) was also performed in accordance with the ASTM Standard D1586 test method at 10-foot intervals beginning at 20 feet bgs in boring BH-01 using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between drive samples are indicated on the logs at the top of the next drive sample.

Following the completion of logging and sampling, the borings (BH-01 and BH-03) were backfilled with soil cuttings and compacted by pushing down with an auger using drill rig weight. Since BH-02 was on an asphalt concrete surface, so the surface was patched with cold asphalt concrete. If construction is delayed, the surface of the borings may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawing Nos. A-2 through A-4 *Logs of Borings*.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

FIELD AND LABORATORY TESTS	
C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeablility (ASTM D 2434)
PA	Particle Size Analysis (ASTM D 6913 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (ASTM D 2419)
SG	Specific Gravity (ASTM D 854)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166)
	Unconfined Compression - Rock (ASTM D 7012)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 2937)

BORING LOG SYMBOLS

DRILLING METHOD SYMBOLS			
Auger Drilling	Mud Rotary Drilling	Dynamic Cone or Hand Driven	Diamond Core

SAMPLE TYPE

- STANDARD PENETRATION TEST Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
- DRIVE SAMPLE 2.42" I.D. sampler (CMS).
- DRIVE SAMPLE No recovery
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

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

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

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

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

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

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

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

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

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Log of Boring No. BH-01

Dates Drilled: 12/28/2021 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi,

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

Ground Surface Elevation (ft): 1933 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
		VERY OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 0.5 inch in maximum dimensions, dense, moist, orangish brown.						CR, PA, CP
					18/30/30	3	112	CL
5		- increased coarse content, light grayish brown			9/16/20	3	113	DS
		- medium dense			13/16/19	2	111	PA
10		**SAND WITH SILT (SP-SM):** fine to coarse-grained, dense, moist, grayish brown.			15/22/30	2	114	
15		- increased coarse content, scattered gravel up to 0.5 inch in maximum dimension, very dense			29/32/31	2	111	
20					15/13/17			
25		**SILTY SAND (SM):** fine to coarse-grained, slightly indurated, very dense, moist, orangish brown.			50-6"	6	111	
30		- trace clay, increased fines content			22/34/22			

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. 20-81-256-02 Drawing No. A-2a

Log of Boring No. BH-01

Dates Drilled: 12/28/2021 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi,

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

Ground Surface Elevation (ft): 1933 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
		VERY OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 2 inches maxium dimension, slightly indurated, very dense, moist, orangish brown.			27/33/44	6	113	
40					25/40/40			
45					50-6"	9	110	
50		- dense			15/23/26			
		End of boring at 51.5 feet bgs. No groundwater was encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/28/2021.						

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No.	Drawing No.
20-81-256-02	A-2b

Log of Boring No. BH-02

Dates Drilled: 12/28/2021 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi,

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

Ground Surface Elevation (ft): 1952 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
		3.5" ASPHALT CONCRETE/ 20" AGGREGATE BASE						
		VERY OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 0.5 inches in maximum dimension, medium dense, moist, reddish-brown.			6/7/10	7	114	PA CL
5		- rootlets, trace clay			7/12/14	8	124	
		- trace caliche, very dense, moist, light orangish brown			40/50-4"	7	120	
10		- dark orangish brown			50-5.5"	8	113	
15		- reddish brown			50-6"	6	118	

End of boring at 15.5 feet bgs.
No groundwater was encountered.
Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight and suface patched with cold asphalt concrete on 12/28/2021.

Log of Boring No. BH-03

Dates Drilled: 12/28/2021 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi,

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

Ground Surface Elevation (ft): 1946 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
		VERY OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, trace clay, very dense, moist, reddish-brown.						
					15/27/50-3"	7	120	
5		- increased coarse content			21/50-6"	13	113	CL
		- orangish brown			31/50-6"	8	119	
10		- severely desiccated, pinhole porosity			37/50-6"	11	118	
15		- increased fines content, light grayish brown			24/30/50	5	126	
20					21/40/50-4"	7	120	
		End of boring at 21.3 feet bgs. No groundwater was encountered. Borehole backfilled with soil cuttings and compacted by pushing down with an auger using the drill rig weight on 12/28/2021.						

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No.	Drawing No.
20-81-256-02	A-4

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.*

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 these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. The tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with Caltrans Test Methods 643, 422 and 417. Test results are presented in the following table.

Table No. B-1, 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	16	26	2,717

Collapse

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

Table No. B-2, Collapse Test Results

Boring No.	Depth (ft)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-01	2.5-4.0	Silty Sand (SM)	-1.6	Slight
BH-02	2.5-4.0	Silty Sand (SM)	-1.9	Slight
BH-03	5.0-6.0	Silty Sand (SM)	-0.3	Slight

Grain-Size Analysis

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 results are presented in the below table.

Table No. B-3, Grain Size Distribution Test Results

Boring No.	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-01	0-5	Silty Sand (SM)	5.0	64.0	31.0	
BH-01	5-10	Silty Sand (SM)	8.0	78.0	14.0	
BH-02	2-5	Silty Sand (SM)	10.0	75.0	15.0	

Maximum Dry Density and Optimum Moisture Content

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

Table No B-4, Summary of Moisture-Density Relationship Test Results

Boring No.	Depth (ft)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-01	0-5	Silty Sand (SM), Orangish Brown	7.5	133.0

Direct Shear

One direct shear test was performed on relatively undisturbed representative ring samples under soaked moisture condition in accordance with the ASTM D3080 procedure. 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-3, *Direct Shear Test Results,* and the following table.

Table No. B-5, Summary of Direct Shear Test Results

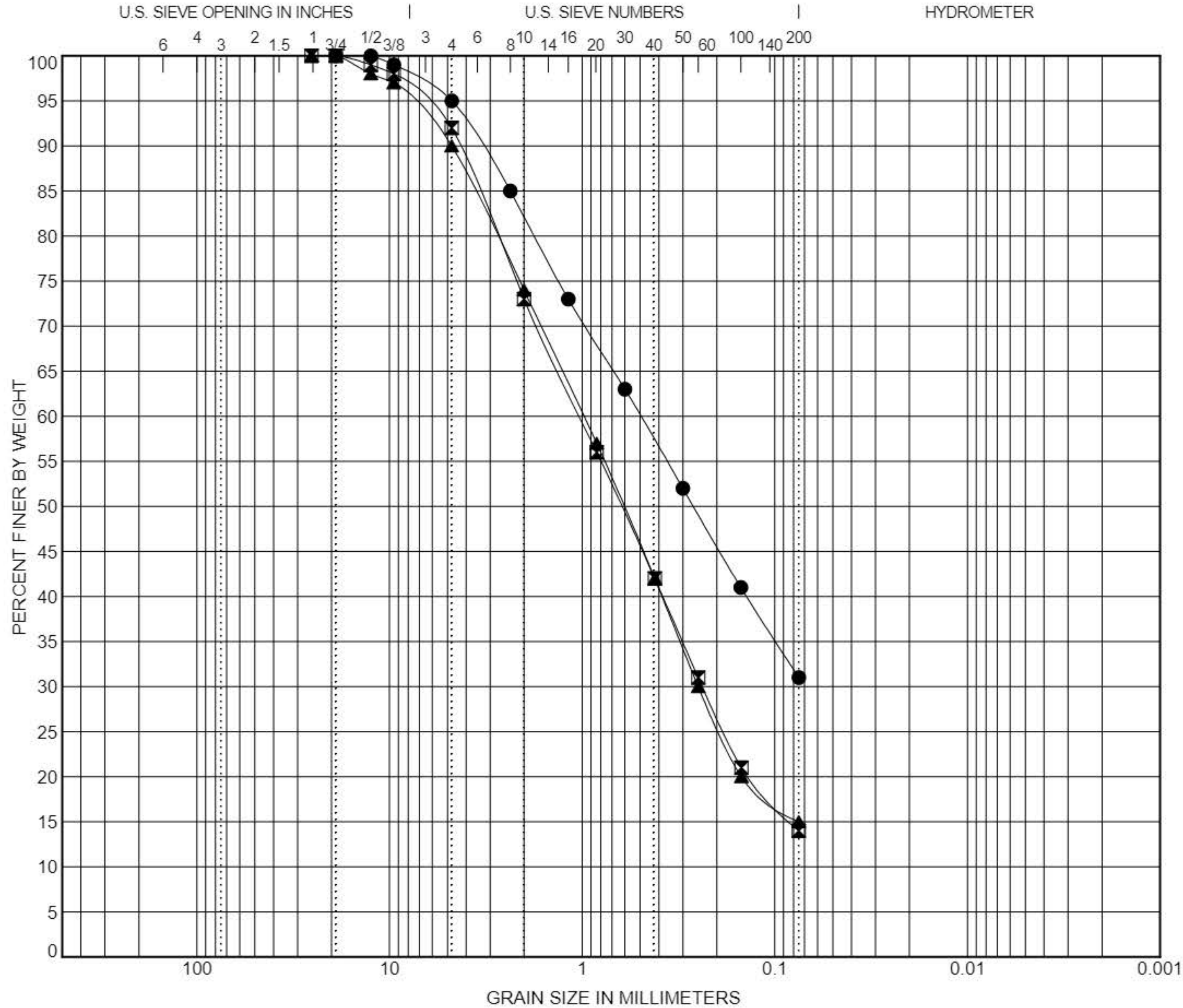
Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-01	5.0-6.5	Silty Sand (SM)	34	10

Consolidation Test

One consolidation test was conducted in accordance with ASTM Standard D2435 method. Data obtained from this test performed on one relatively undisturbed ring sample was used to evaluate the settlement characteristics of the on-site soils under load. Preparation for this test involved trimming the sample, placing it in a 1-inch-high brass ring, and loading it into the test apparatus, which contained porous stones to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. For test result, including sample density and moisture content, see Drawing No. B-4, *Consolidation Test Results*.

Sample Storage

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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-01**	**0-5**	**SILTY SAND (SM)**					
⧗	**BH-01**	**5-10**	**SILTY SAND (SM)**					
▲	**BH-02**	**2-5**	**SILTY SAND (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-01**	**0-5**	**19**	**0.497**			**5.0**	**64.0**	**31.0**	
⧗	**BH-01**	**5-10**	**25.4**	**1.03**	**0.237**		**8.0**	**78.0**	**14.0**	
▲	**BH-02**	**2-5**	**25.4**	**0.979**	**0.25**		**10.0**	**75.0**	**15.0**	

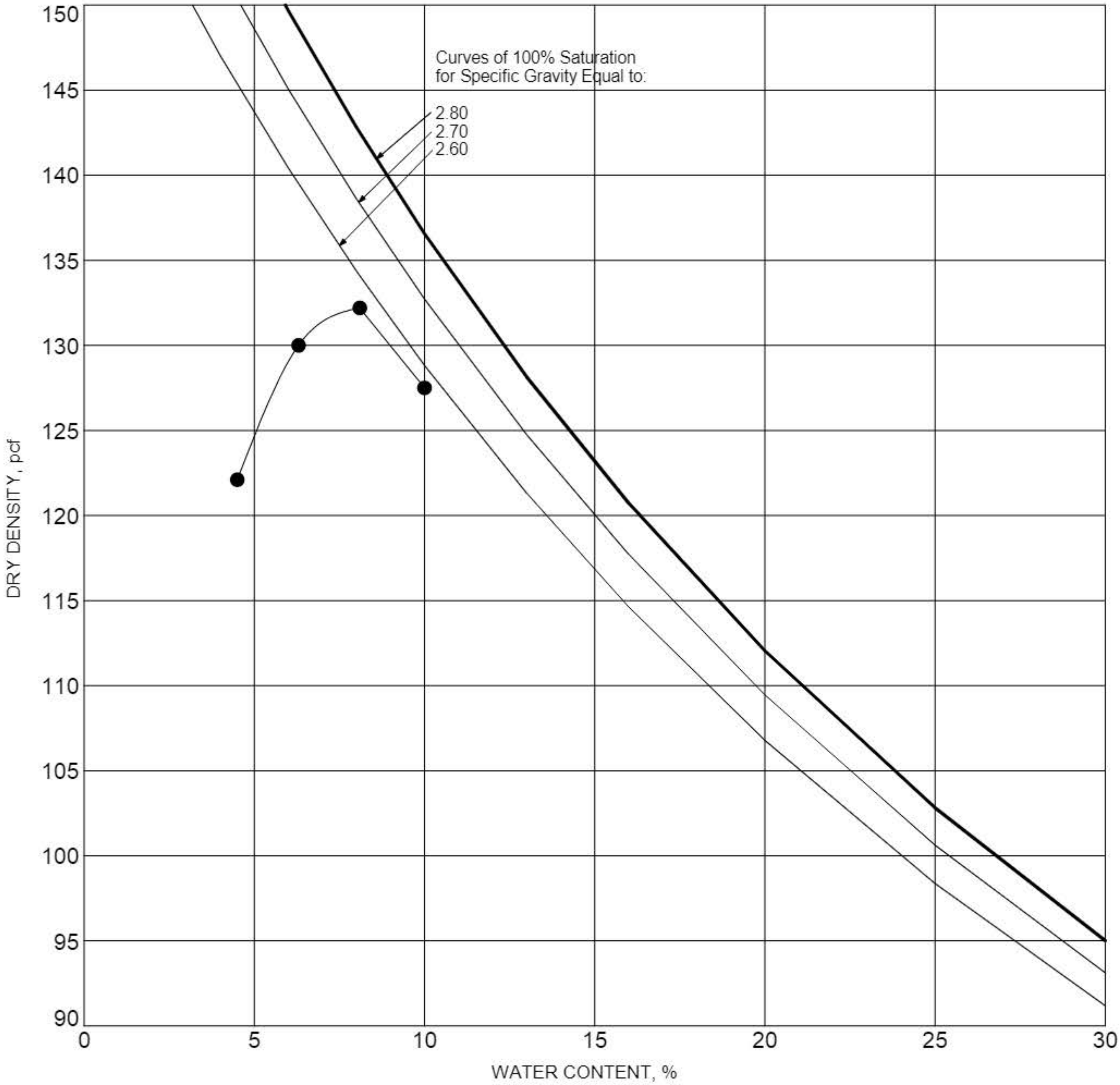
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. 20-81-256-02

Drawing No. B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-01	0-5	SILTY SAND (SM), Orangish Brown	D1557 - A	7.5	133.0

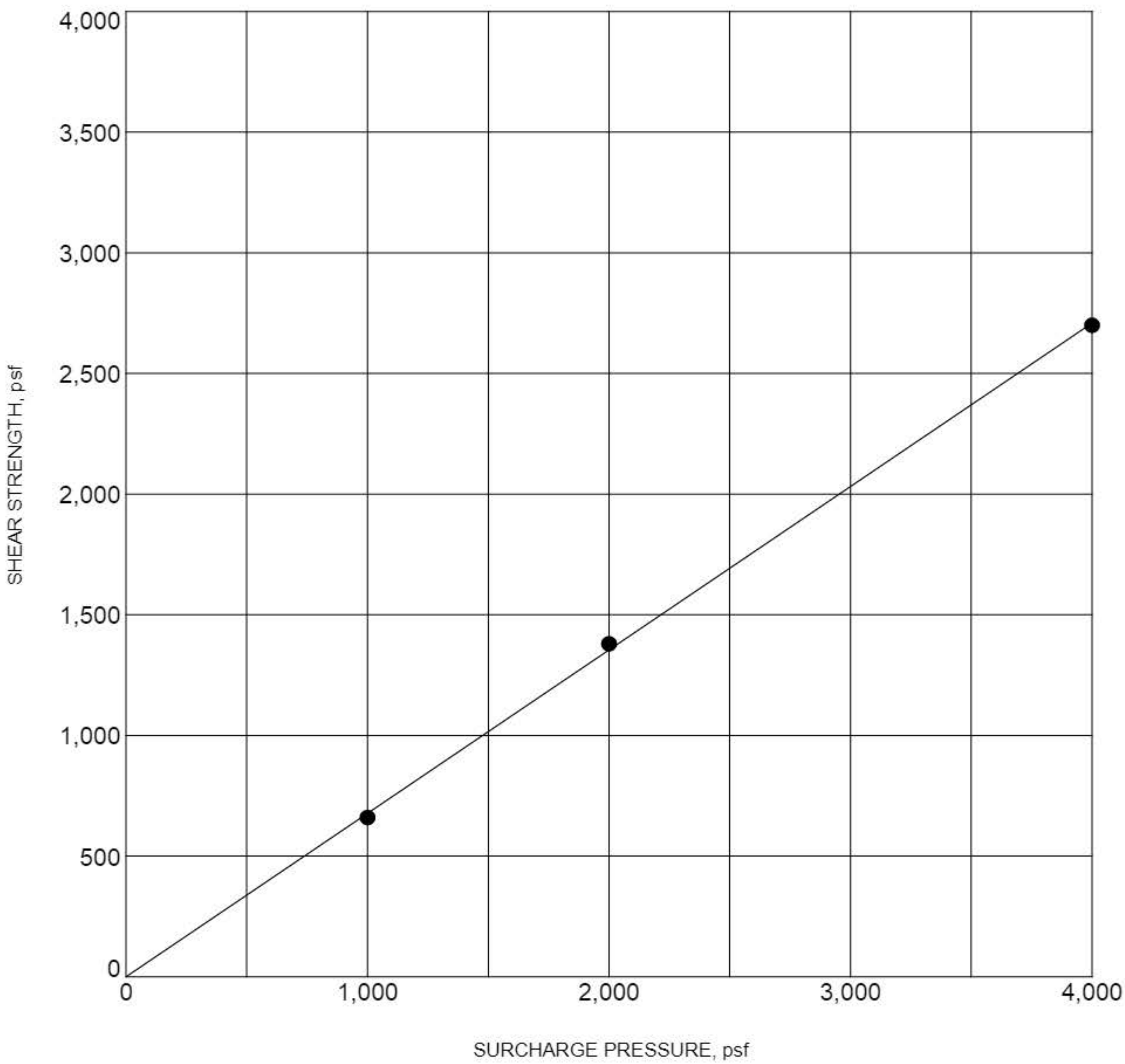
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. 20-81-256-02

Drawing No. B-2



BORING NO.	:	BH-01	DEPTH (ft)	:	5.0-6.5
DESCRIPTION	:	SILTY SAND (SM)			
COHESION (psf)	:	10	FRICTION ANGLE (degrees):		34
MOISTURE CONTENT (%)	:	3.1	DRY DENSITY (pcf)	:	112.7

NOTE: Ultimate Strength.

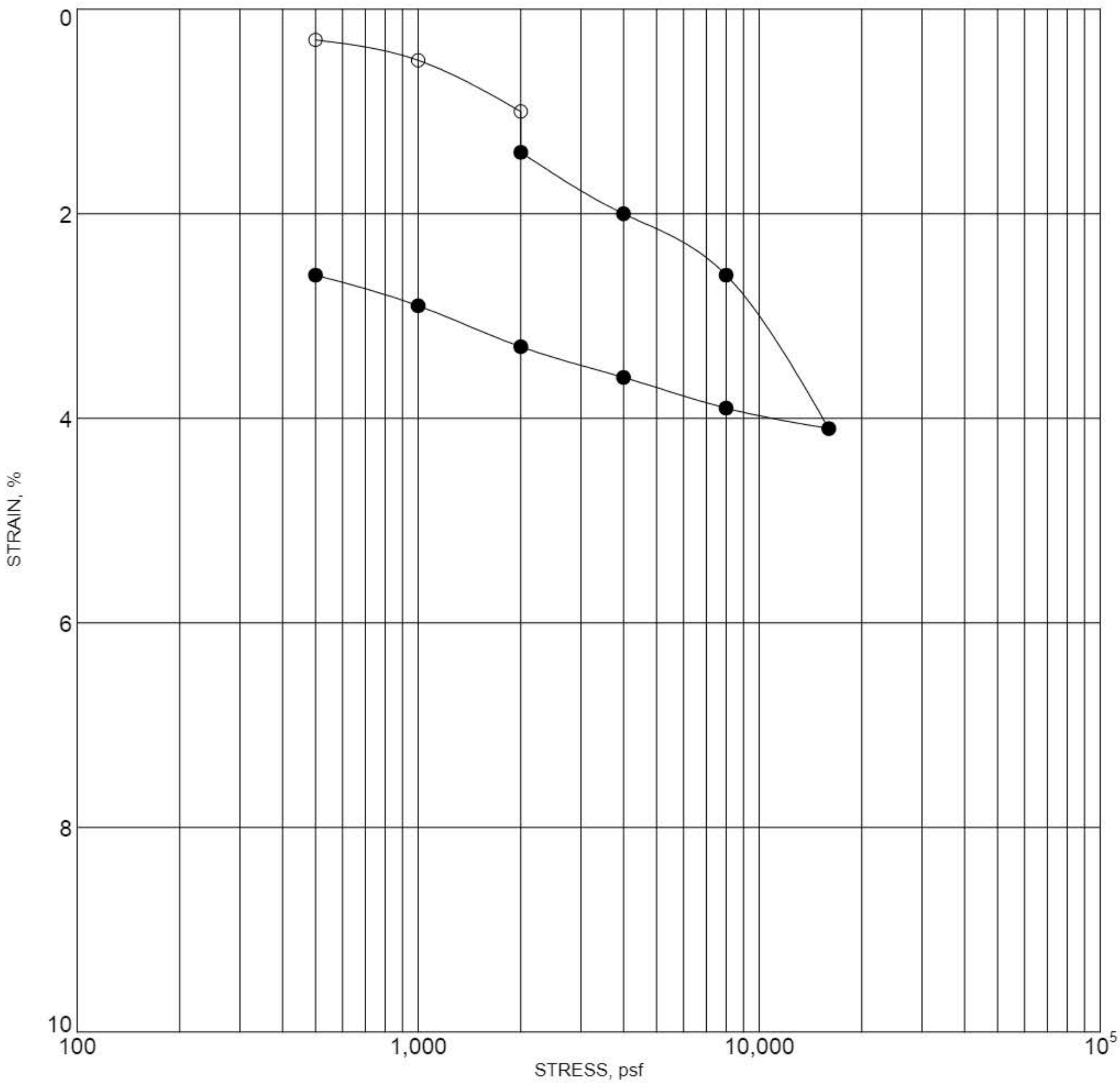
DIRECT SHEAR TEST RESULTS

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. 20-81-256-02

Drawing No. B-3



BORING NO. :	BH-02	DEPTH (ft) :	7.5-9.0
DESCRIPTION :	SILTY SAND (SM)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	7	120	49	0.374
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
25565 Kalmia Avenue
City of Moreno Valley, Riverside County, California
For: Gannett and Fleming, Inc.

Project No. 20-81-256-02

Drawing No. B-4

Appendix C

Liquefaction and Settlement Analysis

APPENDIX C

LIQUEFACTION AND SETTLEMENT ANALYSIS

The subsurface data obtained from the boring BH-01 was used to evaluate the liquefaction potential and associated dry seismic settlement when subjected to ground shaking during earthquakes.

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

A peak ground acceleration (PGA_M) of 0.942g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA was based on the 2019 CBC seismic design parameters presented in Section 8.2, *CBC Seismic design Parameters.*

The result of our analysis is presented on Plates No. C-1 through C-3 and summarized in the following table.

Table C-1, Estimated Dynamic Settlements

Location	Groundwater Conditions	Groundwater Depth (feet bgs)	Dry Seismic Settlement (inches)	Liquefaction Induced Settlement (inches)
BH-01	Current	> 51.5	negligible	negligible
	Historical			

Based on our analysis, the potential for liquefaction induced settlement and dry seismic settlement for the site is negligible.

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

PROJECT INFORMATION	
Project Name	Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
Project No.	20-81-256-02
Project Location	City of Moreno Valley, Riverside County, California
Analyzed By	Syfur Rahman
Reviewed By	Md Zahangir Alam

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

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	8.10
Peak Ground Acceleration, A_{max}	0.94 g
Factor of Safety Against Liquefaction, FS	1.30

BORING DATA AND SITE CONDITIONS			
Boring No.	BH-01		
Ground Surface Elevation	1,937.00 feet		
Proposed Grade Elevation	1,937.00 feet		
GWL Depth Measured During Test	51.50 feet		
GWL Depth Used in Design	51.50 feet		
Borehole Diameter	8.00 inches		
Hammer Weight	140.00 pounds		
Hammer Drop	30.00 inches		
Hammer Energy Efficiency Ratio, ER (%)	80.00 %		
Hammer Distance to Ground Surface	5.00 feet		
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)		
- Ground Slope, S (%)	<<= Leave this blank		
- Free Face Distance to Slope Height Ratio, (L/H)	<<= Leave this blank	Set H to zero =>>	50.00 feet

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Screening *Susceptible Soil? (Y, N)*	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	5.00	SM	Y	115.4	MCal	60.00	31.00
5.00	10.00	SM	Y	114.8	MCal	36.00	14.00
10.00	15.00	SM	Y	116.3	MCal	52.00	14.00
15.00	20.00	SM	Y	113.0	MCal	63.00	14.00
20.00	25.00	SM	Y	113.0	SPT1	30.00	14.00
25.00	30.00	SM	Y	117.3	MCal	50.00	14.00
30.00	35.00	SM	Y	117.3	SPT1	56.00	14.00
35.00	40.00	SM	Y	119.9	MCal	77.00	14.00
40.00	45.00	SM	Y	119.9	SPT1	80.00	14.00
45.00	50.00	SM	Y	119.8	MCal	50.00	14.00

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

PROJECT INFORMATION	
Project Name	Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
Project No.	20-81-256-02
Project Location	City of Moreno Valley, Riverside County, California
Analyzed By	Syfur Rahman
Reviewed By	Md Zahangir Alam

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	8.10
Peak Ground Acceleration, A_{max}	0.94 g
Factor of Safety Against Liquefaction, FS	1.30

BORING DATA AND SITE CONDITIONS		
Boring No.	BH-01	
Ground Surface Elevation	1,937.00 feet	
Proposed Grade Elevation	1,937.00 feet	
GWL Depth Measured During Test	51.50 feet	
GWL Depth Used in Design	51.50 feet	
Borehole Diameter	8.00 inches	
Hammer Weight	140.00 pounds	
Hammer Drop	30.00 inches	
Hammer Energy Efficiency Ratio, ER	80.00 %	
Hammer Distance to Ground Surface	5.00 feet	
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)	
- Ground Slope, S	0.00 %	
- Free Face (L/H) Ratio	N/A	H = 50 feet
Average Total Unit Weight of New Fill	120.00 pcf	(assumed)

SUMMARY OF RESULTS

Severity of Liquefaction:

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

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

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

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

NOTES AND REFERENCES

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

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

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

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

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

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

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field SPT Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	5.00	SM	Y	115.36	MCal	60.00	31.00
5.00	10.00	SM	Y	114.80	MCal	36.00	14.00
10.00	15.00	SM	Y	116.28	MCal	52.00	14.00
15.00	20.00	SM	Y	113.02	MCal	63.00	14.00
20.00	25.00	SM	Y	113.00	SPT1	30.00	14.00
25.00	30.00	SM	Y	117.34	MCal	50.00	14.00
30.00	35.00	SM	Y	117.30	SPT1	56.00	14.00
35.00	40.00	SM	Y	119.89	MCal	77.00	14.00
40.00	45.00	SM	Y	119.90	SPT1	80.00	14.00
45.00	50.00	SM	Y	119.80	MCal	50.00	14.00

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

Depth to Top of Soil Layer (feet)	Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. for Vert. Stress C_N	SPT Corr. for Hammer Energy C_E	SPT Corr. for Borehole Size C_B	SPT Corr. for Rod Length C_R	SPT Corr. for Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ
0.00	288.40	288.40	1.401	1.333	1.150	0.750	0.650	44.9	62.8	68.2	1.000	1.100
5.00	863.80	863.80	1.302	1.333	1.150	0.800	0.650	28.7	37.4	40.3	0.995	1.100
10.00	1,441.50	1,441.50	1.086	1.333	1.150	0.850	0.650	44.1	47.9	50.8	0.986	1.098
15.00	2,014.75	2,014.75	0.999	1.333	1.150	0.950	0.650	59.7	59.6	62.5	0.976	0.998
20.00	2,579.80	2,579.80	0.928	1.333	1.150	0.950	1.000	43.7	40.5	43.4	0.965	0.924
25.00	3,155.65	3,155.65	0.876	1.333	1.150	0.950	0.650	47.3	41.5	44.4	0.952	0.863
30.00	3,742.25	3,742.25	0.944	1.333	1.150	1.000	1.000	85.9	81.0	83.9	0.939	0.812
35.00	4,335.23	4,335.23	0.891	1.333	1.150	1.000	0.650	76.7	68.4	71.3	0.925	0.768
40.00	4,934.70	4,934.70	1.104	1.333	1.150	1.000	1.000	122.7	135.5	138.4	0.909	0.729
45.00	5,533.95	5,533.95	0.719	1.333	1.150	1.000	0.650	49.8	35.8	38.7	0.894	0.695

Depth to Top of Soil Layer (feet)	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
0.00	0.612			Unsaturated Soil			0.00	0.00	0.00
5.00	0.609			Unsaturated Soil			0.00	0.00	0.00
10.00	0.604			Unsaturated Soil			0.00	0.00	0.00
15.00	0.598			Unsaturated Soil			0.00	0.00	0.00
20.00	0.591			Unsaturated Soil			0.00	0.00	0.00
25.00	0.583			Unsaturated Soil			0.00	0.00	0.00
30.00	0.575			Unsaturated Soil			0.00	0.00	0.00
35.00	0.566			Unsaturated Soil			0.00	0.00	0.00
40.00	0.557			Unsaturated Soil			0.00	0.00	0.00
45.00	0.547			Unsaturated Soil			0.00	0.00	0.00

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

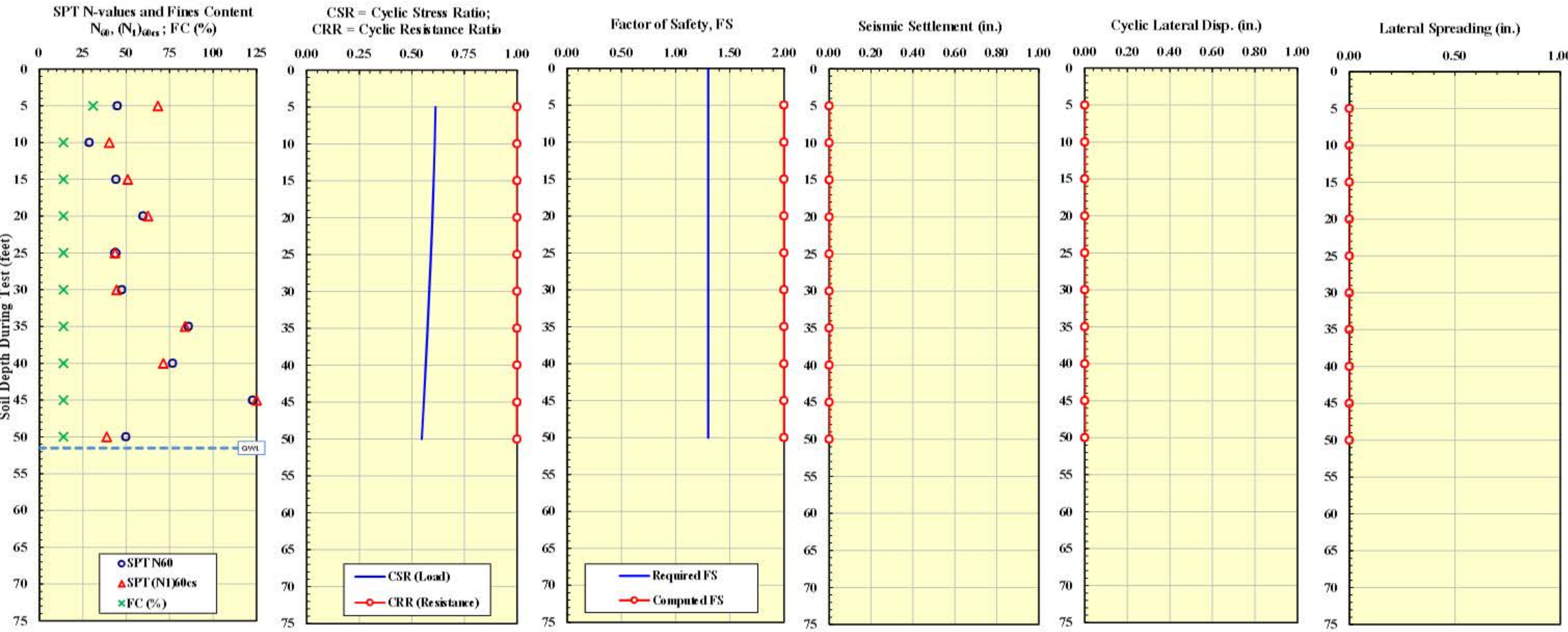
PROJECT INFORMATION	
Project Name	Steeplechase and Kalmia Booster Pump Station (BPS) Replacement
Project No.	20-81-256-02
Project Location	City of Moreno Valley, Riverside County, California
Analyzed By	Syfur Rahman
Reviewed By	Md Zahangir Alam

TOPOGRAPHIC CONDITIONS		
Ground Slope, S	0.00 %	
Free Face (L/H) Ratio	N/A	H = 50.00 feet

GROUNDWATER DATA	
GWL Depth Measured During Test	51.50 feet
GWL Depth Used in Design	51.50 feet

BORING DATA	
Boring No.	BH-01
Ground Surface Elevation	1,937.00 feet
Proposed Grade Elevation	1,937.00 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	8.10
Peak Ground Acceleration, A_{max}	0.94 g
Factor of Safety Against Liquefaction, FS	1.30



Analysis Methods Used ==>>

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