# APPENDIX F

Geotechnical Investigation Report - Pipeline,
Converse Consultants

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

# GEOTECHNICAL INVESTIGATION REPORT

## APPROXIMATELY 2,400 LINEAR FEET OF PIPELINE
CITY OF MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

*CONVERSE PROJECT NO. 20-81-256-03*

*Prepared For:*

**GANNETT FLEMING, INC.**
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*Presented By:*

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March 2, 2022

March 2, 2022

Mr. Jerry Pascoe, PE, GE
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Gannett Fleming, Inc.
20 Pacifica, Suite 430
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**Subject:** **GEOTECHNICAL INVESTIGATION REPORT**
**Approximately 2,400 Linear Feet of Pipeline**
City of Moreno Valley, Riverside County, California
Converse Project No. 20-81-256-03

Dear Mr. Pascoe:

Converse Consultants (Converse) is pleased to submit this Geotechnical Investigation Report to assist with the design and construction of approximately 2,400 linear feet of pipeline project, located at Kalmia Avenue 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 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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## 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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## TABLE OF CONTENTS


**1.0 INTRODUCTION ---- 1**

**2.0 PROJECT DESCRIPTION ---- 1**

**3.0 ALIGNMENT CONDITION ---- 1**

**4.0 SCOPE OF WORK ---- 2**

4.1 Project Set-up ---- 2
4.2 Subsurface Exploration ---- 3
4.3 Laboratory Testing ---- 3
4.4 Analysis and Report Preparation ---- 3

**5.0 LABORATORY TEST RESULTS ---- 3**

5.1 Physical Testing ---- 3
5.2 Chemical Testing - Corrosivity Evaluation ---- 4

**6.0 SUBSURFACE CONDITIONS ---- 4**

6.1 Subsurface Profile ---- 5
6.2 Groundwater ---- 5
6.3 Excavatability ---- 5
6.4 Subsurface Variations ---- 6

**7.0 ENGINEERING GEOLOGY ---- 6**

7.1 Regional Geology ---- 6
7.2 Local Geology ---- 7

**8.0 CBC SEISMIC DESIGN PARAMETERS ---- 7**

**9.0 EARTHWORK RECOMMENDATIONS ---- 7**

9.1 General ---- 7
9.2 Pipeline Subgrade Preparation ---- 8
9.3 Pipe Bedding ---- 8
9.4 Backfill Materials ---- 9
9.5 Compacted Fill Placement ---- 9
9.6 Trench Zone Backfill ---- 10

**10.0 DESIGN RECOMMENDATIONS ---- 11**

10.1 General ---- 11
10.2 Resistance to Lateral Loads ---- 11
10.3 Soil Parameters for Pipe Design ---- 11
10.4 Bearing Pressure for Anchor and Thrust Blocks ---- 12
10.5 Soils Corrosivity ---- 12
10.6 Asphalt Concrete Pavement ---- 13
10.7 Pavement Repair ---- 14

**11.0 CONSTRUCTION RECOMMENDATIONS ---- 14**

11.1 General ---- 15

11.2 Temporary Sloped Excavations ---------- 15
11.3 Shoring Design ---------- 16

**12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION ---------- 18**

**13.0 CLOSURE ---------- 19**

**14.0 REFERENCES ---------- 20**

## FIGURES

**Following Page No.**

Figure No. 1, Approximate Alignment Location Map.......... 1
Figure No. 2, Approximate Boring Locations Map.......... 3
Figure No. 3, Lateral Earth Pressures for Temporary Braced Excavation.......... 17
Figure No. 4, Lateral Earth Pressures on Temporary Cantilever Wall.......... 17

## TABLES

**Page No.**

Table No. 1, CBC Seismic Design Parameters.......... 7
Table No. 2, Resistance to Lateral Loads.......... 11
Table No. 3, Soil Parameters for Pipe Design.......... 12
Table No. 4, Correlation Between Resistivity and Corrosion.......... 13
Table No. 5, Recommended Preliminary Pavement Sections.......... 14
Table No. 6, Slope Ratios for Temporary Excavations.......... 15
Table No. 7, Lateral Earth Pressures for Temporary Shoring.......... 16

## APPENDICES

Appendix A.......... *Field Exploration*
Appendix B.......... *Laboratory Testing Program*

# 1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for Approximately 2,400 Linear Feet of Pipeline project, located on Kalmia Avenue in the City of Moreno Valley, Riverside County, California. The project location is shown in Figure No. 1, *Approximate Alignment 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.

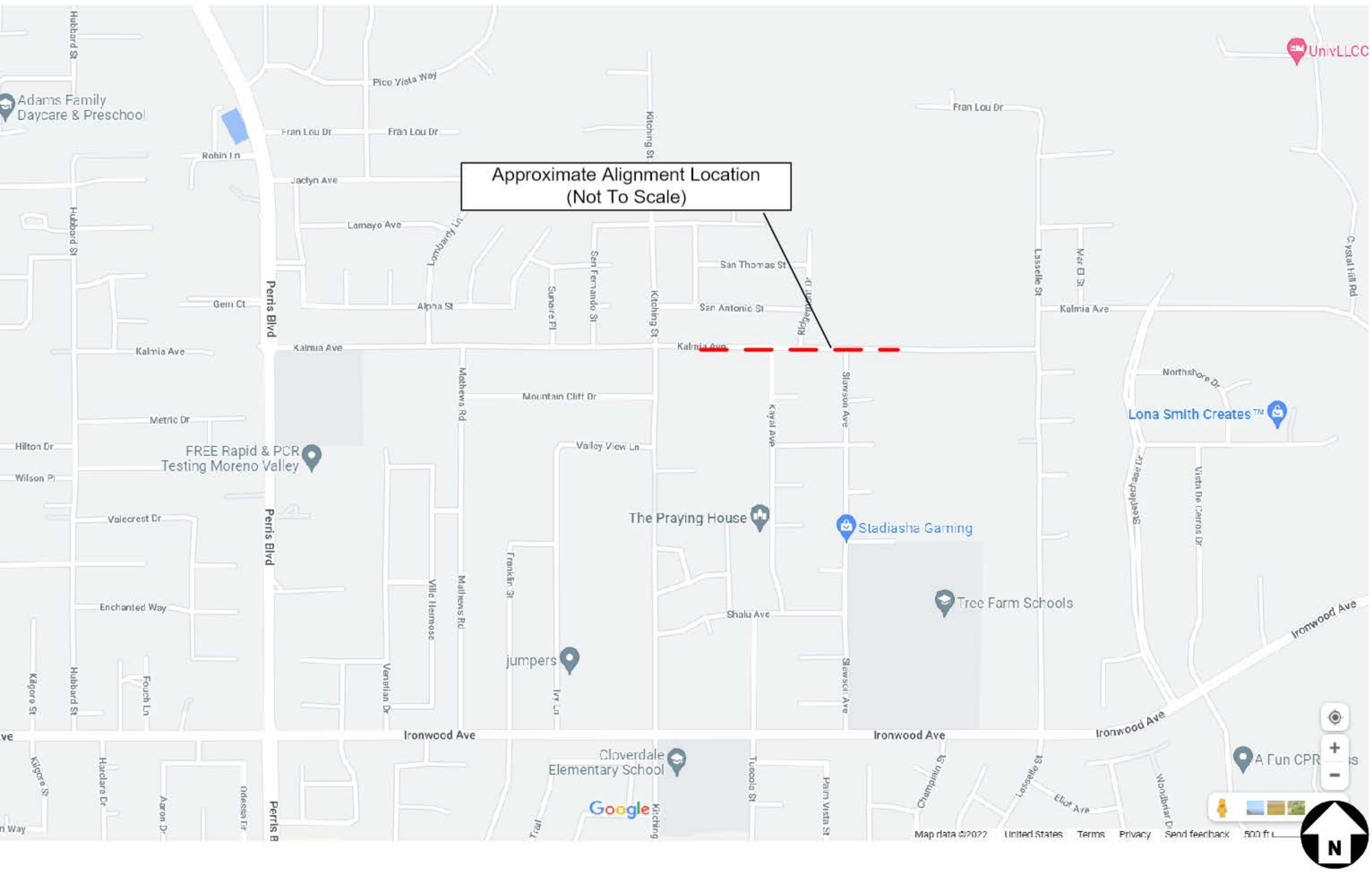
This report is prepared for the project described herein and is intended for use solely by Gannett Fleming, Inc. and their authorized agents for design purposes. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For 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 approximately 2,400 linear feet of 12-inch diameter pipe along Kalmia Avenue starting from the proposed Steeplechase and Kalmia Booster Pump Station (BPS) site to Dale Pressure Zone. We anticipate that the maximum depth to pipe invert for the pipeline will be about 10.0 feet below existing ground surface and it will be installed using open cut and cover technique.

# 3.0 ALIGNMENT CONDITION

Within the project limit, Kalmia Avenue is mostly bounded on the north and south by residential development. It is a paved road with single lane in each direction. No overhead utilities or overhanging streetlights is located within the project limit. Light traffic was observed outside school starting and ending times, heavy traffic was observed during school hours. Photograph No. 1 depicts the present alignment conditions.




oject: Approximately 2,400 Linear Feet of Pipeline

cation: City of Moreno Valley, Riverside County, California

r: Gannett Fleming, Inc.

**Approximate Alignment Location Map**

Project No. 20-81-256-03

Figure No. 1

**Converse Consultants**

*Photograph No. 1: Pipeline alignment along Kalmia Avenue, facing east.*

# 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 marked the boring 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.
- Conducted 3 site visits for utility meet and mark requirements.
- Engaged a California-licensed driller to drill exploratory borings.
- Engaged a California-licensed professional traffic control company.

### *4.2 Subsurface Exploration*

Two exploratory borings (BH-04 and BH-05) were drilled on January 27, 2022, to investigate the subsurface conditions. The borings were drilled using an 8-inch dimeter hollow stem auger to depth of 15.4 and 16.5 feet below existing 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 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 density (ASTM D2216 and ASTM D2937)
- Sand equivalent (ASTM D2419)
- Soil corrosivity (California Tests 422, 417, and 643)
- Grain size distribution (ASTM D6913)
- Maximum dry density and optimum moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

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

### *4.4 Analysis and Report Preparation*

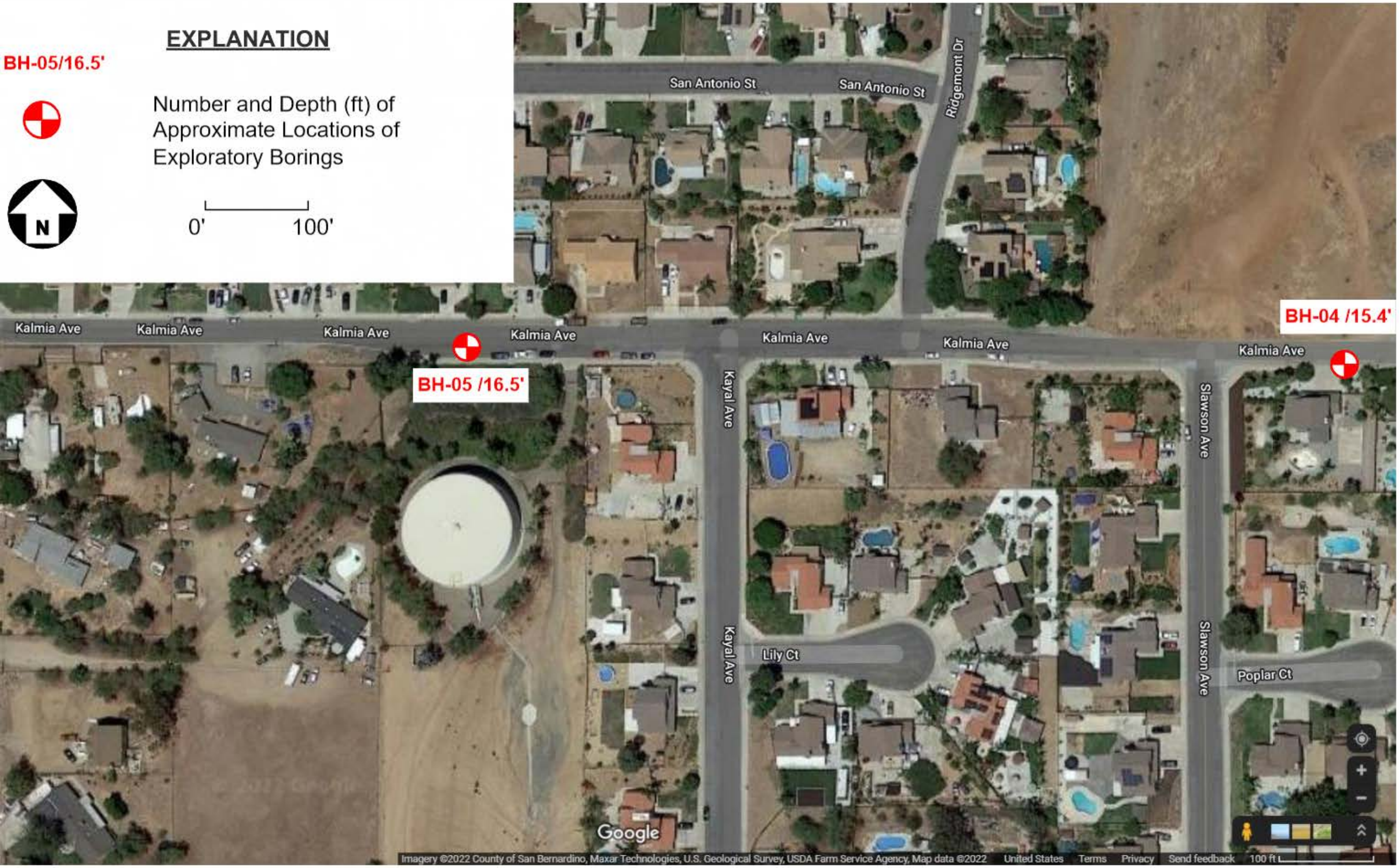
Data obtained from the field exploration and laboratory testing program was compiled and evaluated. Geotechnical analyses of the compiled data were performed, and this report was prepared to present our findings, conclusions, and recommendations for the project.

## 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 Log of Borings in Appendix A, *Field Exploration*. The results are also discussed in the following page.




ject: Approximately 2,400 Linear Feet of Pipeline

cation: City of Moreno Valley, Riverside County, California

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## Approximate Boring Locations Map

Project No.
20-81-256-03

**Converse Consultants**

Figure No.
2

- In-situ Moisture and Dry Density – *In-situ* dry densities and moisture contents of the alignment soils were determined in accordance with ASTM Standard D2216 and D2937. Dry densities of upper 10 feet of alluvium soils ranged from 110 to 131 pounds per cubic foot (pcf) with moisture contents of 4 to 10 percent.
- Sand Equivalent (SE) – Two representative bulk soil samples were tested to evaluate sand equivalent (SE) in accordance with the ASTM Standard D2419 test method. The measured sand equivalent test results were 19.
- Grain Size Analysis – Two representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship test was performed on a representative sample in accordance with ASTM D1557. 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 137.5 pcf and the optimum moisture content of 6.5 percent.
- Direct Shear – One direct shear test was performed on undisturbed representative ring samples under soaked moisture condition in accordance with ASTM Standard D3080. The results are presented in Drawings No. B-3, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

### *5.2 Chemical Testing - Corrosivity Evaluation*

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purposes of these tests were to determine the corrosion potential of alignment 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 measurements of the tested samples were 8.9 and 8.1.
- The sulfate contents of the tested samples were 0.0026 and 0.0032 percent by weight (26 and 32 ppm).
- The chloride concentrations of the tested samples were 19 and 21ppm.
- The minimum electrical resistivities when saturated were 8,031 and 8,423 ohm-cm.

## 6.0 SUBSURFACE 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 along the alignments consisted primarily of a mixture of sand, silt, trace clay and gravel up to 0.5 inches in maximum dimension.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 and A-3 *Logs of Borings,* in Appendix A, *Field Exploration.*

### *6.2 Groundwater*

Groundwater was not encountered in any of the borings to the maximum explored depth of 16.5 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 16.5 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 Excavatability*

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

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

### *6.4 Subsurface Variations*

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the alignments 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 pipeline alignment is discussed below.

### *7.1 Regional Geology*

The proposed alignment 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 proposed alignment 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 proposed alignment is 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.

## 8.0 CBC SEISMIC DESIGN PARAMETERS

Seismic parameters based on the 2019 California Building Code (CBSC, 2019) are provided in the following table. These parameters were determined using the generalized coordinates (33.953058N, 117.215849W) and the Seismic Design Maps ATC online tool.

**Table No. 1, CBC Seismic Design Parameters**

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

## 9.0 EARTHWORK RECOMMENDATIONS

Earthwork for the project will include trench excavation, pipe subgrade preparation, pipeline bedding placement, and trench backfill following the placement of the pipe.

### *9.1 General*

Prior to the start of construction, all underground existing utilities and appurtenances should be located along and adjacent to the proposed alignment. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing structures or utilities.

All debris, deleterious material and surficial soils containing roots and perishable materials (if any) should be stripped and removed from the alignment. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

Migration of fines from the surrounding native soils, in the case of water leaks from the pipe, must be considered in selecting the gradation of the materials placed within the trench, including bedding, pipe zone and trench zone backfill, as defined in the following sections. Such migration of fines may deteriorate pipe support and may result in settlement/ground loss at the surface.

### *9.2 Pipeline Subgrade Preparation*

The final subgrade surface should be level, firm, uniform, 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 3 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 sub-grade 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.

### *9.3 Pipe Bedding*

Bedding is defined as the material supporting and surrounding the pipe to one foot above the pipe. Pipe bedding should follow EMWD or City of Moreno Valley Standards, whichever is applicable. If additional recommendations, beyond EMWD or City of Moreno Valley 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. The sand equivalent of the site soils was found 19. 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.

Based on the ground water data, migration of fines from the surrounding native and/or fill soils may not be considered in selecting the gradation of any imported bedding material.

## *9.4 Backfill Materials*

No fill or aggregate base should be placed until excavation and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the project alignments, free of debris or organic matter are suitable as compacted fill after proper processing and removal of oversize materials to 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 should be 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 fill materials.

Imported soils, if used as fill, should be predominantly granular and meet the above criteria. Any imported fill should be tested and approved by geotechnical representative prior to delivery to the alignment.

## *9.5 Compacted Fill Placement*

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 and compacted to at least 90 percent of the laboratory maximum dry density. 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.

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

## *9.6 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 on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Trench zone backfill should follow EMWD or City of Moreno Valley Standards, whichever is applicable. If additional recommendations beyond EMWD or City of Moreno Valley Standards are needed, the following specifications can be used for trench backfills.

- 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.
- 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

General design recommendations, resistance to lateral loads, pipe design parameters, bearing pressures, and soil corrosivity are discussed in the following subsections.

## *10.1 General*

Where pipes connect to rigid structures and 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 pipelines where they exit the structures. Consideration can be given to flexible connections, concrete slurry support beneath the pipes where they exit the structures, overlaying the pipes with a few inches of compressible material, (i.e., Styrofoam, or other materials), or other techniques.

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

## *10.2 Resistance to Lateral Loads*

Resistance to lateral loads can be assumed to be provided by passive earth pressures and friction between construction materials and native soils. The resistance to lateral loads were estimated by using on-site native soils strength parameters obtained from laboratory testing. The resistance to lateral loads recommended for use in design of the thrust blocks are presented in the following table.

**Table No. 2, Resistance to Lateral Loads**

| Soil Parameters | Value |
|---|---|
| Passive earth pressure (psf per foot of depth) | 300 |
| Maximum allowable bearing pressure against native soils (psf) | 2,500 |
| *Coefficient of friction between CML&C steel pipe and native soils, fs | 0.25 |
| Note: * Pipe material is not known at this time | |

## *10.3 Soil Parameters for Pipe Design*

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

**Table No. 3, Soil Parameters for Pipe Design**

| Soil Parameters | Value |
|---|---|
| Average compacted fill total unit weight (assuming 92 percent of relative compaction), $\gamma$ (pcf) | 135 |
| Soil cohesion, c (psf) | 0 |
| Angle of internal friction of soils, $\phi$ | 32 |
| Coefficient of friction between concrete and native soils, fs | 0.35 |
| *Coefficient of friction between CML&C steel 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 |
| Note: * Pipe material is not known at this time | |

## *10.4 Bearing Pressure for Anchor and Thrust Blocks*

An allowable net bearing pressure presented in Table No. 3, *Soil Parameters for Pipe Design* may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 18 inches wide.

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

## *10.5 Soils Corrosivity*

Two representative soil samples 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 design recommendations pertaining to soil corrosivity are presented below.

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

We anticipate that concrete structures such as footings, slab, and pavement will be exposed to moisture from precipitation and irrigation. Based on the alignment locations and the results of chloride testing of the alignment 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. 4, Correlation Between Resistivity and Corrosion**

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

The measured values of the minimum electrical resistivities of the samples when saturated were 8.031 and 8,423 ohm-cm for the alignment. This indicates that the soil tested is moderately corrosive to ferrous metals in contact with the soil. 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 alignment soils.

## *10.6 Asphalt Concrete Pavement*

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

Based on the above information, asphalt concrete and aggregate base thickness was calculated 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 table below.

**Table No. 5, Recommended Preliminary 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 | 9.5 | 8.5 |
| | 8 | 5.0 | 11.0 | 10 |

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 Pavement Repair*

Pavement repairs due to the installation of pipeline should be based on the City of Moreno Valley Standards or Table No. 5 *Recommended Preliminary Pavement Sections* whichever is applicable.

## 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 alignment. 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. 6, Slope Ratios for Temporary Excavations**

| Soil Type | OSHA Soil Type | Depth of Cut (feet) | Recommended Maximum Slope (Horizontal: Vertical)[1] |
|---|---|---|---|
| Silty Sand (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, excavation 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.

### *11.3 Shoring Design*

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

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

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

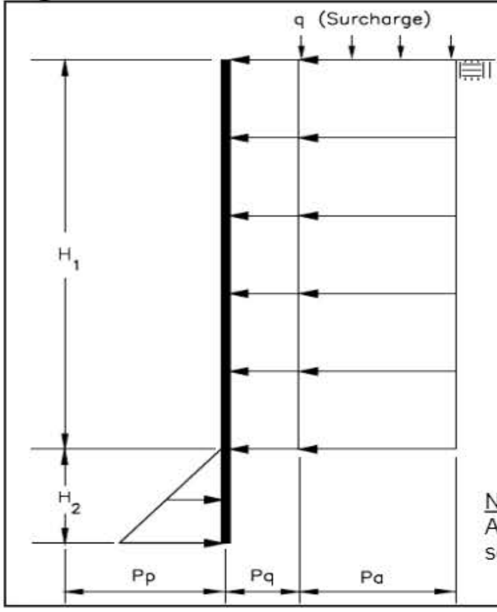
**Table No. 7, Lateral Earth Pressures for Temporary Shoring**

| Lateral Resistance Soil Parameters* | Value |
|---|---|
| Active Earth Pressure (Braced Shoring) (psf) (A) | 26 |
| Active Earth Pressure (Cantilever Shoring) (psf) (B) | 40 |
| At-Rest Earth Pressure (Cantilever Shoring) (psf) (C) | 60 |
| Passive earth pressure (psf per foot of depth) (D) | 300 |
| Maximum allowable bearing pressure against native soils (psf) (E) | 2,500 |
| Coefficient of friction between sheet pile and native soils, fs (F) | 0.25 |

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

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

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




Total Earth Pressure, P

**P = Pq + Pa**

**Pq = 0.5q** - incremental surcharge pressure

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

Lateral Pressure Resistance

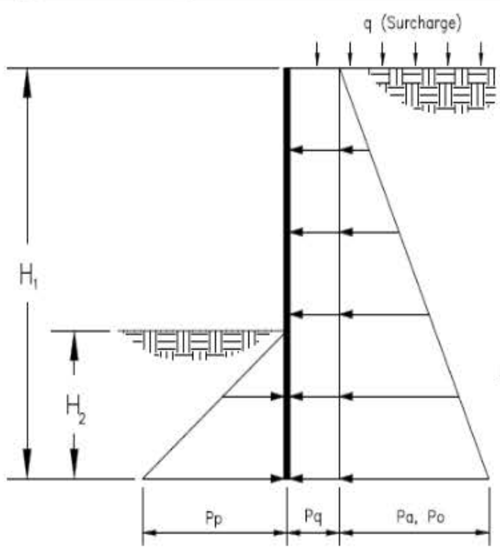
**Pp = (D) $H_2$ ≤ (E) psf** - passive earth pressure (on native soils)

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

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

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

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




Total Earth Pressure, P

**P = Pq + Pa, Po**

**Pq = 0.5q** - incremental surcharge pressure

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

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

Lateral Pressure Resistance

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

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

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

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

Passive resistance includes a safety factor of 1.5. The upper 1 foot for passive resistance should be ignored unless the surface is confined by a pavement or slab.

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

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

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

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

# 12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

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

# 13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Gannett Fleming, Inc. 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 alignment 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

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

AMERICAN SOCIETY OF CIVIL ENGINEERS (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, SEI/ASCE Standard No. 7-16, dated January 1, 2022.

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CALIFORNIA DEPARTMENT OF WATER RESOURCES (DWR), 2020, Water Data Library (http://wdl.water.ca.gov/waterdatalibrary/), accessed in May 2021.

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

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

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# Appendix A

## Field Exploration

# APPENDIX A

## FIELD EXPLORATION

Our field investigation included an alignment reconnaissance and a subsurface exploration program consisting of drilling soil borings. During the alignment reconnaissance, the surface conditions were noted, and the borings were marked at the locations selected by Ms. Carolina Cubides with Gannett Fleming, Inc. The approximate boring locations were established in the field with reference to existing street centerlines and other visible features. The locations should be considered accurate only to the degree implied by the method used.

Boring BH-01 through BH-03 were drilled within the pump station site. Two exploratory borings (BH-04 and BH-05) were drilled on January 27, 2022, to investigate the subsurface conditions along the pipe alignment. The borings were drilled using an 8-inch dimeter hollow stem auger to depth of 15.4 and 16.5 feet below existing 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.

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

Following the completion of logging and sampling, the borings were backfilled with soil cuttings mixed with cement, compacted by pushing down with an auger using the drill rig weight and surface patched with black dyed cement. If construction is delayed, the surface may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 and A-3, *Logs of Borings*.

# SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

## FIELD AND LABORATORY TESTS

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

## BORING LOG SYMBOLS

SAMPLE TYPE

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

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

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

### DRILLING METHOD SYMBOLS

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

## UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Approximately 2,400 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. 20-81-256-03

Drawing No. A-1a

## 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-04

Dates Drilled: 1/27/2022 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi,

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

Ground Surface Elevation (ft): 1950 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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" ASPHALT CONCRETE/ 4" AGGREGATE BASE** | | | | | | |
| | | **VERY OLD ALLUVIAL FAN DEPOSITS** | | | | | | SE, CR, PA, CP |
| | | **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 0.5 inch in maximum dimension, trace clay, medium dense, moist, reddish brown to brown. | | | 7/10/23 | 8 | 131 | |
| 5 | | - trace caliche, very dense, orangish brown | | | 16/46/50-4" | 10 | 119 | |
| | | | | | 14/27/36 | 4 | 123 | |
| 10 | | - dense | | | 12/23/24 | 5 | 115 | |
| 15 | | - very dense | | | 24/50-2" | 8 | 117 | |
| | | End of boring at 15.4 feet bgs.<br>No groundwater was encountered.<br>Borehole backfilled with soil cuttings mixed with cement, compacted by pushing down with an auger using the drill rig weight, and suface patched with black dyed cement concrete on 1/27/2022. | | | | | | |

Converse Consultants

Approximately 2,400 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. 20-81-256-03

Drawing No. A-2

# Log of Boring No. BH-05

Dates Drilled: 1/27/2022 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi,

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

Ground Surface Elevation (ft): 1951 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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" ASPHALT CONCRETE/ 4" AGGREGATE BASE** | | | | | | |
| | | **VERY OLD ALLUVIAL FAN DEPOSITS** | | | | | | SE, CR |
| | | **SILTY SAND (SM):** fine to coarse-grained, trace clay, very dense, moist, dark brown. | | | 6/14/50-2" | 7 | 129 | |
| 5 | | - large rock on north side of hole, loose | | | 1/push | 6 | 112 | DS |
| | | - medium dense | | | 11/16/16 | 6 | 124 | PA |
| 10 | | - loose | | | 4/4/6 | 6 | 111 | |
| 15 | | - medium dense | | | 5/8/8 | 5 | 116 | |

End of boring at 16.5 feet bgs.
No groundwater was encountered.
Borehole backfilled with soil cuttings mixed with cement, compacted by pushing down with an auger using the drill rig weight, and suface patched with black dyed cement concrete on 1/27/2022.

# Appendix B

## Laboratory Testing Program

# APPENDIX B

# LABORATORY TESTING PROGRAM

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

## ***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 alignment soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration.*

## **Sand Equivalent**

Two representative soil samples were tested in accordance with the ASTM D2419 test method to determine the sand equivalent. The test results are presented in the following table.

**Table No. B-1, Sand Equivalent Test Results**

| Boring No. | Depth (feet) | Soil Description | Sand Equivalent |
|---|---|---|---|
| BH-04 | 1-5 | Silty Sand (SM) | 19 |
| BH-05 | 1-5 | Silty Sand (SM) | 19 |

## **Soil Corrosivity**

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of the 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-2, Summary of Soil Corrosivity Test Results**

| Boring No. | Depth (feet) | pH | Soluble Sulfates (CA 417) (% by weight) | Soluble Chlorides (CA 422) (ppm) | Min. Resistivity (CA 643) (Ohm-cm) |
|---|---|---|---|---|---|
| BH-04 | 1-5 | 8.9 | 0.0026 | 19 | 8,031 |
| BH-05 | 1-5 | 8.1 | 0.0032 | 21 | 8,423 |

## Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analyses were performed on two 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-04 | 1-5 | Silty Sand (SM) | 6.0 | 72.0 | 22.0 | |
| BH-05 | 5-10 | Silty Sandy (SM) | 10.0 | 67.0 | 23.0 | |

## Maximum Dry Density and Optimum Moisture Content

Laboratory maximum dry density-optimum moisture content relationship test was performed on one representative bulk sample. The test was conducted in accordance with the ASTM Standard D1557 test method. The test 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 Results**

| Boring No. | Depth (feet) | Soil Description | Optimum Moisture (%) | Maximum Density (lb/cft) |
|---|---|---|---|---|
| BH-04 | 1-5 | Silty Sand (SM), Reddish Brown | 6.5 | 137.5 |

## Direct Shear

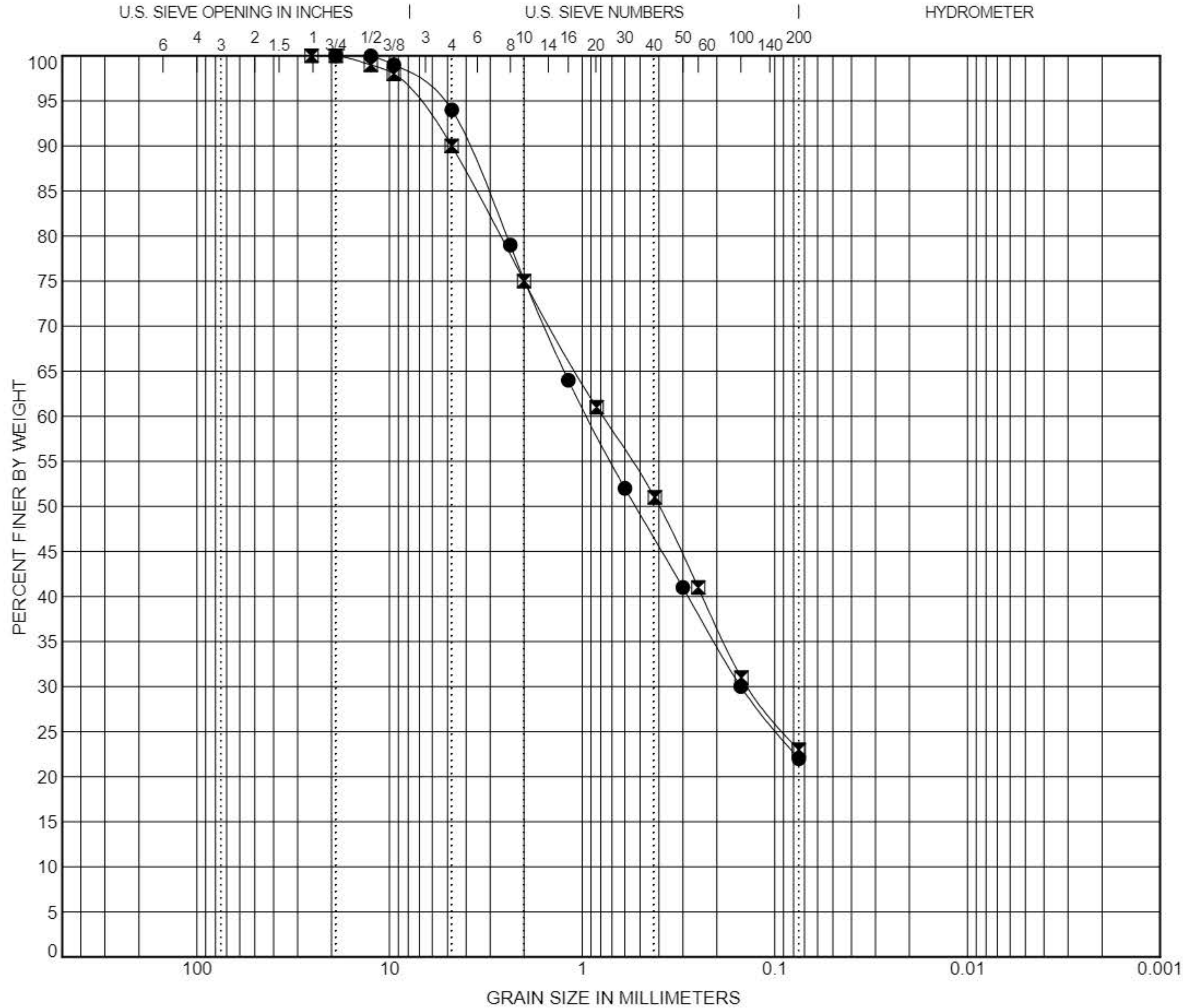
One direct shear test was performed on relatively undisturbed representative ring samples under soaked moisture condition in accordance with the ASTM D3080 procedure. For the test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected 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 Drawings 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-05 | 5.0-6.5 | Silty Sand (SM) | 32 | 60 |

## Sample Storage

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




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Boring No. | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **BH-04** | **1-5** | **SILTY SAND (SM)** | | | | | |
| ⧗ | **BH-05** | **5-10** | **SILTY SAND (SM)** | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| | Boring No. | Depth (ft) | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **BH-04** | **1-5** | **19** | **0.942** | **0.15** | | **6.0** | **72.0** | **22.0** | |
| ⧗ | **BH-05** | **5-10** | **25.4** | **0.784** | **0.137** | | **10.0** | **67.0** | **23.0** | |
| | | | | | | | | | | |
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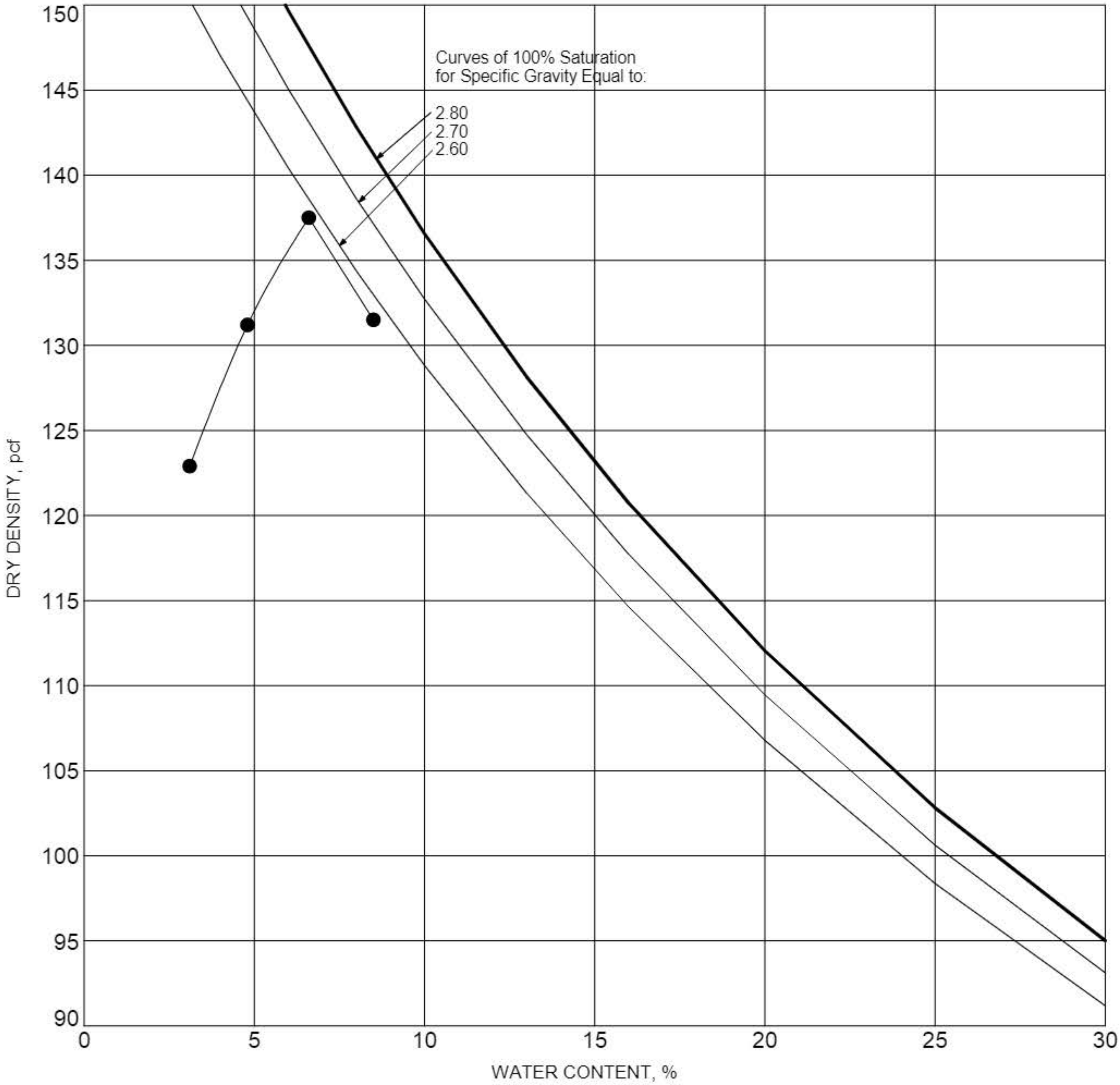
# GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Approximately 2,400 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. **20-81-256-03**

Drawing No. **B-1**




| SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf |
|---|---|---|---|---|---|---|
| ● | BH-04 | 1-5 | SILTY SAND (SM) | D1557 - A | 6.5 | 137.5 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

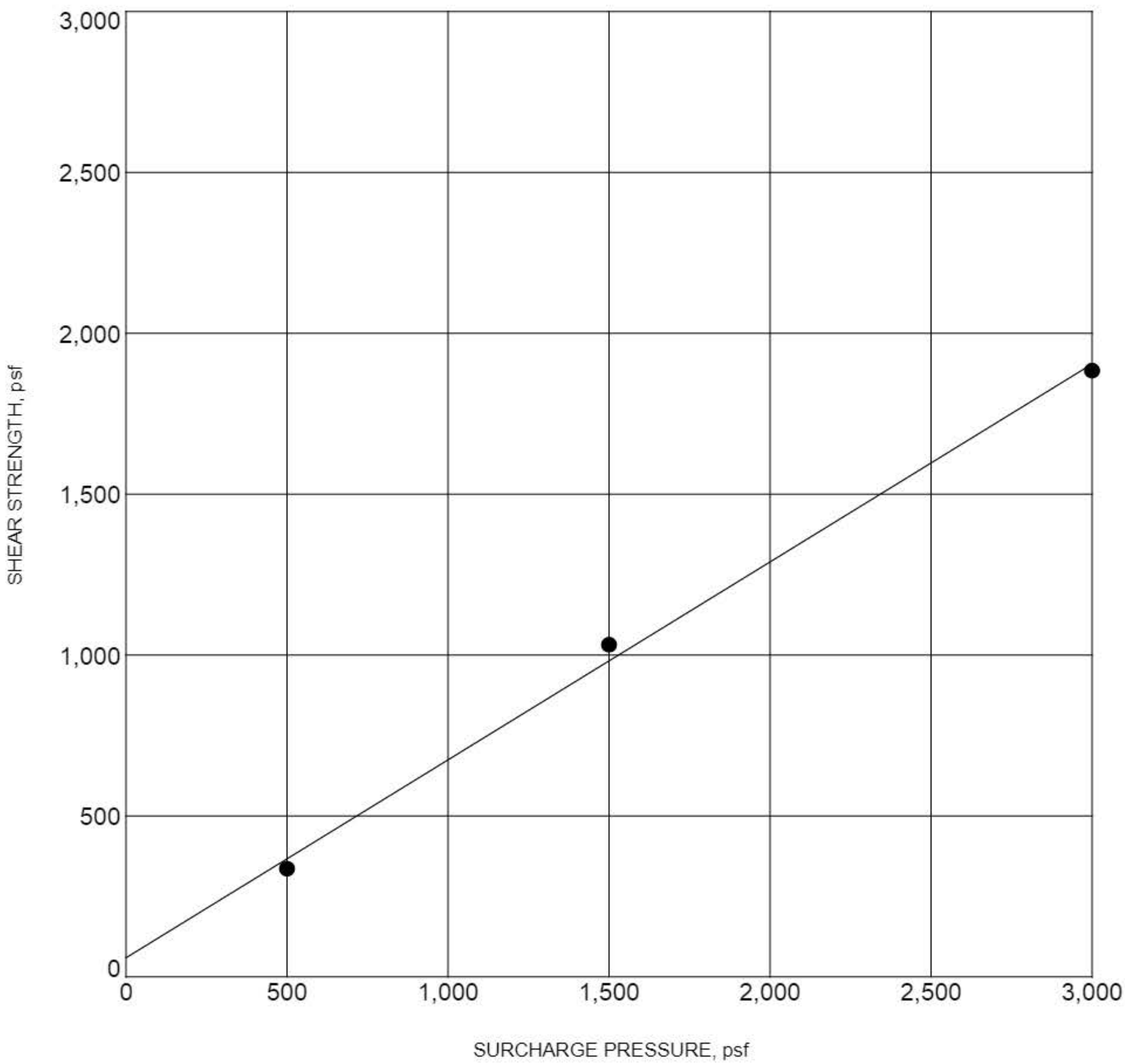
# MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Approximately 2,400 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No.
20-81-256-03

Drawing No.
B-2




| | | | |
|---|---|---|---|
| BORING NO. : | **BH-05** | DEPTH (ft) : | **5.0-6.5** |
| DESCRIPTION : | **SILTY SAND (SM)** | | |
| COHESION (psf) : | **60** | FRICTION ANGLE (degrees): | **32** |
| MOISTURE CONTENT (%) : | **6.0** | DRY DENSITY (pcf) : | **112.4** |

NOTE: Ultimate Strength.

# DIRECT SHEAR TEST RESULTS

Converse Consultants

Approximately 2,400 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Gannett Fleming, Inc.

Project No. **20-81-256-03**

Drawing No. **B-3**