APPENDIX B

Geotechnical Report
B1 - Pettit Regulated Pressure Zone Project
July 25, 2019

Mr. Jorge D. Anaya, PE
Civil Engineer/Engineering Department
Eastern Municipal Water District (EMWD)
2277 Trumble Road
Perris, CA 92572

Subject: GEOTECHNICAL INVESTIGATION REPORT
Pettit Regulated Pressure Zone Project
City of Moreno Valley, Riverside County, California
Converse Project No. 18-81-312-01

Dear Mr. Anaya:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of the Pettit Regulated Pressure Zone Project located within the City of Moreno Valley, Riverside County, California. This report was prepared in accordance with our revised proposal dated January 10, 2019, and your Agreement No. 116781 dated January 28, 2019.

The Geotechnical Data Report (Converse, 2019) prepared for the Cactus II Feeder project has been utilized by Converse for the design of PRS No. 2 and 3 as well as the pipeline segments on Perris Boulevard and Lasselle Street and pipeline jumper connections west of Indian Street along Cactus Avenue.

Based upon our field investigation, laboratory data, and analyses, the proposed project is considered feasible from a geotechnical standpoint, provided that 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 EMWD. Should you have any questions, please do not hesitate to contact us at 909-796-0544.

CONVERSE CONSULTANTS

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

Dist: 3/Addresssee
HSQ/JB/ZA/kvg
PROFESSIONAL CERTIFICATION

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

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

Zahangir Alam, PhD, EIT
Senior Staff Engineer

Jay Burnham, PG
Project Geologist

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

The following is a summary of our geotechnical investigation, conclusions, and recommendations, as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed Pettit Regulated Pressure Zone (RPZ) Project is located within the City of Moreno Valley, Riverside County, California. The Pettit RPZ Project consists of 4 pressure reducing stations [Pressure Reducing Stations (PRSs) 1 through 4; each location has been described in Section 3.0 Site Description]. The approximate size of PRSs 1 through 4 will be 9.5'x17.5', 9'x17', 6'x17.5' and 5.5'x17.5', respectively. The pressure reducing station enclosures will be 8"x8"x16" CMU walls (retaining walls) or wrought iron with removable roof. The enclosures will be founded on shallow foundations and 8" thick concrete slabs-on-grade.

- The project includes water pipeline segments and appurtenances along Perris Boulevard (approximately 637 linear feet) and Lasselle Street (approximately 658 linear feet); and 12 pipeline jumper connections. These jumper connections are located east and west of Indian Street along Cactus Avenue. The proposed water pipelines will be constructed of 12" diameter PVC C900. The jumper connections will consist of 8" and 12" PVC C900 piping with gate valves connecting to existing pipelines. The invert depth of pipelines along Perris Blvd. and Lasselle St. will be approximately within 5 to 7 feet bgs. The invert depth of pipelines at PRS sites will be between approximately 4 and 10 feet bgs. All pipelines and connection pipes will be installed using cut and cover technique.

- The Geotechnical Data Report (Converse, 2019) prepared for the Cactus II Feeder project has been utilized for the design of PRS No. 2 and 3 as well as the pipeline segments on Perris Boulevard and Lasselle Street and pipeline jumper connections west of Indian Street along Cactus Avenue. The relevant investigation data and recommendations have been included and incorporated with this report.

- Our scope of work included project set-up, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report.

- Eight exploratory borings (BH-1 through BH-8) were drilled between March 5 and 6, 2019, at locations approved by EMWD. The borings were drilled to depths ranging from approximately 11.5 to 51.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations. Due to proximity of existing underground utilities, hand auger was utilized for most of the borings up to 5 feet bgs during drilling.
Thirty-three exploratory borings (BH-01 through BH-35) were drilled between February 15 and February 22, 2017 and two additional borings were drilled on June 21, 2018 at locations approved by Black & Veatch to investigate the subsurface conditions for the Cactus II Feeder project. Relevant borings (Cactus II: BH-01 through BH-04, BH-07, BH-16, BH-17 and BH-30) which are in close proximity to the Pettit RPZ project location have been utilized in this project by Converse.

The subsurface soils at the project areas consist primarily of a mixture of sand, silt, clay and gravel. Gravel up to 1.5 inches in largest dimension was encountered in several borings (Pettit: BH-02 through BH-04 and BH-08; Cactus II: BH-02, BH-03 and BH-07) at various depths. All borings were able to penetrate the subsurface profile to the planned depths during current investigation. On the Cactus II Feeder project, granitic bedrock (excavated as sand) was encountered in borings BH-16 and BH-17 from depths of 2.5 feet to the maximum explored depth. Excavation difficulty is presented later.

Groundwater was not encountered in any borings during the current investigation. Groundwater was encountered in boring Cactus II: BH-02 at depth 23 feet bgs during Cactus II Feeder project investigation. Based on the data reviewed, the historical high groundwater level is estimated to be approximately 14 feet bgs and the current depth is expected to be approximately 21 feet bgs. Groundwater may be encountered if groundwater rises to historic levels. Considering the depths of Pettit RPZ PRSs and pipelines, dewatering is not expected to be required during the construction of the project. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

The project is not located within a currently designated State of California or Riverside County Earthquake Fault Zone. The nearest active fault is the San Jacinto fault, located approximately 4.5 miles northeast of the east end of the project.

The potential impact to the project from surface fault rupture, land sliding lateral spreading, tsunamis, and earth-quake-induced flooding is considered to be low.

Liquefaction potential is considered low for the entire project for current ground water condition.

In the current investigation (Pettit RPZ project), the expansion index (EI) of the samples tested were 1, 11 and 27, corresponding to very low to low expansion potential. The measured sand equivalents were 9, 14, 32 and 64. Generally, Sand equivalent greater than 30 is suitable for pipe bedding material.
In the Cactus II Feeder project, the expansion index (EI) of the samples tested was 0, corresponding to very low expansion potential. The measured sand equivalents were between 13 and 15. Generally, Sand equivalent greater than 30 is suitable for pipe bedding material.

The sulfate contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations. No concrete type restrictions are specified for exposure category S0. A minimum compressive strength of 2,500 psi is recommended. The chloride contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category C2 (concrete is exposed to moisture and external sources of chlorides). For exposure category C2, ACI provides concrete compressive strength of at least 5,000 psi, maximum water cement ratio of 0.4 and maximum chloride content of 0.15 percent.

The measured values of the minimum electrical resistivity when saturated ranged from 2,570 to 17,720 Ohm-cm. This indicates that the tested soils are mildly to moderately corrosive for ferrous metals in contact with the soils.

According to the Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive if the pH is 5.5 or less, or chloride content is 500 parts per million (ppm) or greater, or sulfate content is 1,500 ppm or greater, or resistivity less than 2000 ohm-cm. Based on the tested results, the soils within the project areas are not considered corrosive (resistivities are greater than 2000 ohm-cm). Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures, if necessary, for any ferrous metals in contact with the sites and alignments soils.

Prior to the start of construction, all existing underground utilities should be located within the project limits. 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.

The surface and subsurface soil materials for the proposed project are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. Difficult excavation will occur where high concentration of gravel is encountered. Based on results of the seismic refraction survey conducted for the Cactus II Feeder project, excavation will be difficult due to the presence of bedrock at the PRS Site 3 and the pipeline along Lasselle Street at depth below 2.5 feet bgs.

Excavated on-site earth materials cleared of deleterious matter can be moisture conditioned and re-used as compacted fill.
The footings, slabs-on-grade and pavement should be overexcavated based on Table No. 5, *Overexcavation Depths*. The overexcavation below the footings and slabs should be uniform. The overexcavation should extend to at least 2 feet beyond the footprint of the footings and slabs and at least 1 foot beyond the edge of the pavements.

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

Earthwork for the pipeline includes pipe trench excavation, pipe subgrade preparation, and backfilling of the trench following the placement of the pipe. Excavated on-site soils free of particles larger than 3 inches and deleterious matter may be used for backfill. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse grained soil and between optimum and above 2 percent of optimum for fine grained soil, then placed in horizontal layers not exceeding loose lifts of 8 inches. All backfill material should be compacted to a minimum of 90 percent of the laboratory maximum dry density. The upper 1 foot of backfill beneath the pavement sections should be compacted to at least 95 percent of the laboratory maximum dry density.

Footings should be at least 18 inches in width and embedded to at least 18 inches below the lowest adjacent grade. The footing dimensions and reinforcement should be based on structural design. Continuous and isolated footings can be designed based on an allowable net bearing capacity of 2,500 psf.

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or horizontal distance of the span of the structures.

Based on our analysis, negligible dry seismic settlement (up to 0.25 inches) was observed. Differential settlement is estimated as half of the total settlement over a horizontal distance of the span of the structures.

Allowable net bearing capacities, lateral earth pressures, PRSs and pipelines design parameters are presented in the text of this report.

Pavement design recommendations are presented in the Section 10.10 *Asphalt Concrete Pavement* of this report.
- Slope ratios for temporary excavations are provided in the Section 11.2 *Temporary Sloped Excavations* and shoring recommendations are provided in the Section 11.3 *Shoring Design* of this report.

Based on our investigation, it is our professional opinion that the project is suitable for construction provided the findings and conclusions presented in this geotechnical investigation report are considered in the planning, design and construction of the project.
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1.0 INTRODUCTION

This report contains the findings of the geotechnical investigation performed by Converse for the Pettit Regulated Pressure Zone (RPZ) Project located within the City of Moreno Valley, Riverside County, California. The owner of this project is Eastern Municipal Water District (EMWD). The approximate project vicinity is shown in Figure No. 1, Approximate Project Vicinity Map. The approximate location of the proposed PRSs and pipeline alignments is shown on Figure No. 2, Approximate PRSs and Pipeline Alignments Locations Map.

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

This report was prepared for the project described herein and is intended for use solely by EMWD and its authorized agents. This report may be made available to the prospective bidders for bidding purposes. However, the bidders are responsible for their own interpretation of the project conditions between and beyond the boring locations, based on factual data contained in this report. This report may not contain sufficient information for use by others and/or other purposes.

2.0 PROJECT BACKGROUND/DESCRIPTION

Under a separate contract with Black and Veatch, Converse Consultants prepared a Geotechnical Data Report for the Cactus II Feeder Project (Converse, 2019), which consists of approximately 29,700 linear feet of pipeline, to be installed along the Cactus Avenue, Indian Street, Alessandro Boulevard, and Perris Boulevard in the City of Moreno Valley, Riverside County, California. The Pettit Regulated Pressure Zone project is in close proximity between north of Cactus Avenue, south of Cottonwood Avenue and between Heacock Avenue and Lasselle Street in the City of Moreno Valley.

The Pettit RPZ Project consists of 4 pressure reducing stations [Pressure Reducing Stations (PRSs) 1 through 4; each location has been described in Section 3.0 Site Description]. The approximate size of PRSs 1 through 4 will be 9.5’x17.5’, 9’x17’, 6’x17.5’ and 5.5’x17.5’, respectively. The pressure reducing station enclosures will be 8”x8”x16” CMU walls (retaining walls) or wrought iron with removable roof. The enclosures will be founded on shallow foundations and 8” thick concrete slabs-on-grade.

The project includes water pipeline segments and appurtenances along Perris Boulevard (approximately 637 linear feet) and Lasselle Street (approximately 658 linear feet); and 12 pipeline jumper connections. These jumper connections are located east and west of Indian Street along Cactus Avenue. The proposed water pipelines will be constructed of 12” diameter PVC C900. The jumper connections will consist of 8” and 12” PVC C900 piping with gate valves connecting to existing pipelines. The invert depth
Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Approximate Project Vicinity Map

Project No. 18-81-312-01

Converse Consultants
Approximate PRSs and Pipeline Alignments Locations Map

Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Not to Scale
of pipelines along Perris Blvd. and Lasselle St. will be approximately within 5 to 7 feet bgs. The invert depth of pipelines at PRS sites will be approximately between 4 and 10 feet bgs. All pipelines and connection pipes will be installed using cut and cover technique.

The Geotechnical Data Report (Converse, 2019) prepared for the Cactus II Feeder project has been utilized for the design of PRS No. 2 and 3 as well as the pipeline segments on Perris Boulevard and Lasselle Street and pipeline jumper connections west of Indian Street along Cactus Avenue. The relevant investigation data and recommendations have been included and incorporated with this report.

3.0 SITE DESCRIPTION

PRS sites and the alignments conditions are described below.

**PRS Site 1**
- Located east of Heacock Street, approximately 400 feet south from the intersection of Heacock Street and Alessandro Boulevard.
- Just beside shoulder on the dirt area.
- Overhead utilities were observed.

**PRS Site 2**
- Located north of Alessandro Boulevard, approximately 125 feet east from the intersection of Indian Street and Alessandro Boulevard.
- Just beside shoulder on the landscaping area.
- No overhead utilities were observed.

**PRS Site 3**
- Located north of Alessandro Boulevard, approximately 125 feet west from the intersection of Lasselle Street and Alessandro Boulevard.
- No shoulder
- On the dirt area.
- Overhead utilities were observed.

**PRS Site 4**
- Located north of Cactus Avenue, approximately 90 feet west from the intersection of Lasselle Street and Cactus Avenue.
- Just beside shoulder on the dirt area.
- No overhead utilities were observed.

**Pipeline along Perris Boulevard**
- Station No. 35+68 to 42+05
- Approximately 367 linear feet of 12-inch-diameter PVC C900 water pipeline.
- Three lanes in each direction with shoulders, median and intermittent turn lanes.
- Bounded by commercial places.
- Overhead utilities were observed on west side.
Medium to heavy traffic was observed throughout day.

**Pipeline along Lasselle Street**
- Station No. 50+00 to 56+58
- Approximately 658 linear feet of 12-inch-diameter PVC C900 water pipeline.
- One lane in each direction with intermittent turn lanes and no shoulders and median.
- Bounded by open land.
- No overhead utilities were observed along the street except at the intersection with Alessandro Blvd. where an overhead exists just north of the intersection.
- Light traffic was observed throughout day.

**Pipeline Jumper Connections**
All connection pipes, 8 and 12 inches in diameter PVC C900, are located at the Intersection of Cactus Avenue with Sylvester Drive, Chantry Drive, Kitching Street, Rio Hondo Drive, Agave Street, Parkwood Court, Victor Drive, Perris Boulevard, Joshua Tree Avenue, Unit Court and in the intersection of Perris Boulevard and Brodiaea Avenue.

### 4.0 SCOPE OF WORK

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

#### 4.1 Document Review

We reviewed the previous geotechnical data report for the Cactus II Feeder project (Converse, 2019). We also reviewed geohazard and groundwater maps to evaluate any impact on the design and construction of the proposed project.

Besides, pertinent information was used to understand the subsurface conditions and plan the investigation for this project.

#### 4.2 Project Set-up

The project set-up consisted of the following tasks.

- Conducted field reconnaissance to mark the boring locations so drill rig access to all the locations is available.
- Obtained encroachment permit from the City of Moreno Valley.
- 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.
4.3 Subsurface Exploration

Our subsurface exploration included soil borings. Additional soil boring and seismic refraction survey data collected for the Cactus II Feeder project is also included.

4.3.1 Soil Borings

Eight exploratory borings (BH-01 through BH-08) for Pettit RPZ project were drilled between March 5 and 6, 2019, at locations approved by EMWD. The borings were drilled to depths ranging from approximately 11.5 to 51.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations. Due to proximity of existing underground utilities, hand auger was utilized for most of the borings up to 5 feet bgs during drilling.

The approximate locations of the borings are shown on Figures No. 3a through 3d, Approximate Boring Locations Map. A detailed discussion of the subsurface exploration is presented in Appendix A, Field Exploration. Approximate locations and depths of the borings are presented in the following table.

Table No. 1, Boring Locations Details

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Date Drilled</th>
<th>Location, Pipeline Station</th>
<th>Depth (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>3/5/19</td>
<td>PRS Site 1</td>
<td>21.5</td>
</tr>
<tr>
<td>BH-02</td>
<td>3/6/19</td>
<td>Intersection of Cactus Avenue &amp; Victor Drive</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-03</td>
<td>3/6/19</td>
<td>Intersection of Cactus Avenue &amp; Perris Blvd</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-04</td>
<td>3/5/19</td>
<td>Intersection of Cactus Avenue &amp; Agave Street</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-05</td>
<td>3/5/19</td>
<td>Intersection of Cactus Avenue &amp; Parkwood Court</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-06</td>
<td>3/6/19</td>
<td>Intersection of Cactus Avenue &amp; Rio Hondo Drive</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-07</td>
<td>3/6/19</td>
<td>Intersection of Cactus Avenue &amp; Sylvester Drive</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-08</td>
<td>3/5/19</td>
<td>PRS Site 4</td>
<td>51.5</td>
</tr>
</tbody>
</table>

Thirty-three exploratory borings (BH-01 through BH-35) were drilled between February 15 and February 22, 2017 and two additional borings were drilled on June 21, 2018 at locations approved by Black & Veatch to investigate the subsurface conditions for the Cactus II Feeder project. Relevant borings which are in close proximity to current Pettit RPZ project location have been utilized in this project by Converse and are presented in the following table.

Table No. 2, Relevant Boring Locations Details (Cactus II Feeder)

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Date Drilled</th>
<th>Location, Pipeline Station</th>
<th>Depth (feet)</th>
<th>Pettit RPZ Covered Items</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>2/21/17</td>
<td>Turnout Facility No. 1, 0+00</td>
<td>26.5</td>
<td>Connection pipes</td>
</tr>
</tbody>
</table>
Approximate Boring Locations Map

Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Project No. 18-81-312-01

Figure No. 3a
Approximate Boring Locations Map

Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Project No. 18-81-312-01

Figure No. 3b
Approximate Boring Locations Map

Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Project No. 18-81-312-01

LEGEND
BH-04
Number and Approximate Location of Exploratory Boring

Converse Consultants
Approximate Boring Locations Map

Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Converse Consultants

Project No. 18-81-312-01

Figure No. 3d
Relevant Boring Locations (Cactus II Feeder Project)
LEGEND:

- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST TELEPHONE
- EXIST GAS
- EXIST ELECTRIC

NOTES:

1. FOR TURNOUT FACILITY ENLARGED PLAN, SEE FIGURE NO. 5 IN APPENDIX D.

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.
LEGEND:

- **PROPOSED CML&C STEEL WATER PIPE**
- **EXIST WATER**
- **R/W** - RIGHT-OF-WAY
- **EXIST SEWER**
- **EXIST TELEPHONE**
- **EXIST GAS**
- **EXIST ELECTRIC**
- **EXIST STORM DRAIN**

**SOIL CUTTNGS AND COLD PATCH. BOREHOLE MUST BE LOCATED WITHIN PIPELINE TRENCH WIDTH.**

**FIGURE 1"=40'**

**MATCHLINE SEE FIGURE 2**

**MATCHLINE SEE FIGURE 4**

**CACTUS II FEEDER PIPELINE ALIGNMENT**

**EASTERN MUNICIPAL WATER DISTRICT**

**PROJECT 190120**

**EMWD EASTERN MUNICIPAL WATER DISTRICT**
MATCHLINE SEE FIGURE 3

MATCHLINE SEE FIGURE 5

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST TELEPHONE
- EXIST ELECTRIC

15°

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.
MATCHLINE SEE FIGURE 8

MATCHLINE SEE FIGURE 10

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- R/W — RIGHT-OF-WAY
- EXIST SEWER
- EXIST TELEPHONE
- EXIST GAS
- EXIST ELECTRIC
LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST GAS
- EXIST ELECTRIC
- EXIST TELEPHONE

FIGURE 1" = 40'
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

FIGURE 1"=40'

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- EXIST SEWER
- EXIST GAS
- EXIST TELEPHONE
- R/W
- RIGHT-OF-WAY
- EXIST ELECTRIC

NOTES:
1. FOR TURNOUT FACILITY ENLARGED PLAN, SEE FIGURE NO. 6 IN APPENDIX D.

EASTERN MUNICIPAL WATER DISTRICT
CACTUS II FEEDER
PIPELINE ALIGNMENT

FIGURE 46
### Table No. 2, Relevant Boring Locations Details (Cactus II Feeder) continued

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Date Drilled</th>
<th>Location, Pipeline Station</th>
<th>Depth (feet)</th>
<th>Pettit RPZ Covered Items</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-02</td>
<td>2/15/17</td>
<td>Cactus Avenue, 2+50</td>
<td>51.5</td>
<td>along Cactus Avenue between Heacock St. and Indian St.</td>
</tr>
<tr>
<td>BH-03</td>
<td>2/15/17</td>
<td>Cactus Avenue, 11+00</td>
<td>16.5</td>
<td></td>
</tr>
<tr>
<td>BH-04</td>
<td>2/15/17</td>
<td>Cactus Avenue, 21+50</td>
<td>16.5</td>
<td></td>
</tr>
<tr>
<td>BH-07</td>
<td>2/16/17</td>
<td>Indian Street, 50+00</td>
<td>16.5</td>
<td>PRS Site 2</td>
</tr>
<tr>
<td>BH-16</td>
<td>2/20/17</td>
<td>Alessandro Boulevard, 127+00</td>
<td>15.5</td>
<td>PRS Site 3 and pipeline along Lasselle St.</td>
</tr>
<tr>
<td>BH-17</td>
<td>2/20/17</td>
<td>Alessandro Boulevard, 137+00</td>
<td>15.5</td>
<td></td>
</tr>
<tr>
<td>BH-30</td>
<td>2/22/17</td>
<td>Prior Turnout Facility No. 2 Location, Perris Boulevard, 407+00</td>
<td>26.5</td>
<td>Pipeline along Perris Blvd.</td>
</tr>
</tbody>
</table>

(The approximate locations of the borings are attached after Figure No. 3d.)

All boring locations (Pettit RPZ and Cactus II Feeder projects) are presented in Figure No. 4, *Approximate Boring Locations Key Map*.

#### 4.3.2 Seismic Refraction Survey

Southwest Geophysics, Inc. was retained to conduct a seismic refraction survey for the Cactus II Feeder project consisting of seven seismic lines in areas of suspected hard bedrock. The purpose of the survey was to obtain a shear wave velocity profile of the subsurface materials and to assist in evaluation of the excavatability of the bedrock. The seismic refraction survey report is presented in Appendix E, *Seismic Refraction Survey*.

#### 4.4 Laboratory Testing

Representative samples of soils within the project limits were tested in the laboratory to aid in the soils’ classification, and to evaluate their relevant engineering properties. These tests included the following.

- In situ moisture contents and dry densities (ASTM D2216 and D7263)
- Expansion Index Test (ASTM D4829)
- Sand equivalent (ASTM D2419)
- Soil corrosivity tests (California Test Methods 643, 422, and 417)
- Swell/Collapse tests (ASTM D4546)
- R-value (California Test Method 301)
- Grain size analysis (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)
- Consolidation (ASTM D2435)

For in-situ moisture and dry density data, see the logs of borings in Appendix A, *Field Exploration* and Appendix A-1, *Relevant Logs of Borings for Cactus II Feeder*. For a
Approximate Boring Locations Key Map

LEGEND
Number and Approximate Location of Exploratory Boring (Pettit RPZ Project)
Number and Approximate Location of Exploratory Boring (Cactus II Feeder Project)

Project: Pettit Regulated Pressure Zone
Location: City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Converse Consultants
Project No. 18-81-312-01
Figure No. 4
description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.

### 4.5 Analysis and Report Preparation

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

### 5.0 SITE CONDITIONS

The subsurface conditions within the project limits are discussed in the subsections below.

#### 5.1 Existing Pavement Section

The encountered pavement thicknesses at the boring locations are presented in the following table.

**Table No. 3, Existing Pavement Thicknesses**

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Project</th>
<th>Date Drilled</th>
<th>Asphalt Concrete Thickness (inches)</th>
<th>Aggregate Base Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>Pettit RPZ</td>
<td>3/5/19</td>
<td>6.5</td>
<td>8.0</td>
</tr>
<tr>
<td>BH-02</td>
<td>Pettit RPZ</td>
<td>3/6/19</td>
<td>5.0</td>
<td>9.0</td>
</tr>
<tr>
<td>BH-03</td>
<td>Pettit RPZ</td>
<td>3/6/19</td>
<td>8.5</td>
<td>14.0</td>
</tr>
<tr>
<td>BH-04</td>
<td>Pettit RPZ</td>
<td>3/5/19</td>
<td>5.5</td>
<td>4.0</td>
</tr>
<tr>
<td>BH-05</td>
<td>Pettit RPZ</td>
<td>3/5/19</td>
<td>5.5</td>
<td>4.0</td>
</tr>
<tr>
<td>BH-06</td>
<td>Pettit RPZ</td>
<td>3/6/19</td>
<td>6.5</td>
<td>9.5</td>
</tr>
<tr>
<td>BH-07</td>
<td>Pettit RPZ</td>
<td>3/6/19</td>
<td>5.5</td>
<td>14.0</td>
</tr>
<tr>
<td>BH-08</td>
<td>Pettit RPZ</td>
<td>3/5/19</td>
<td>4.5</td>
<td>9.0</td>
</tr>
<tr>
<td>BH-01*</td>
<td>Cactus II Feeder</td>
<td>2/21/17</td>
<td>N/E</td>
<td>N/E</td>
</tr>
<tr>
<td>BH-02</td>
<td>Cactus II Feeder</td>
<td>2/15/17</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>BH-03</td>
<td>Cactus II Feeder</td>
<td>2/15/17</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>BH-04</td>
<td>Cactus II Feeder</td>
<td>2/15/17</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>BH-07</td>
<td>Cactus II Feeder</td>
<td>2/16/17</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>BH-16*</td>
<td>Cactus II Feeder</td>
<td>2/20/17</td>
<td>N/E</td>
<td>N/E</td>
</tr>
<tr>
<td>BH-17*</td>
<td>Cactus II Feeder</td>
<td>2/20/17</td>
<td>N/E</td>
<td>N/E</td>
</tr>
<tr>
<td>BH-30*</td>
<td>Cactus II Feeder</td>
<td>2/22/17</td>
<td>N/E</td>
<td>N/E</td>
</tr>
</tbody>
</table>

(*drilled on dirt area. N/E = not encountered.*)
5.2 Subsurface Profile

Based on the exploratory borings and laboratory test results, the subsurface soils at the project areas consist primarily of a mixture of sand, silt, clay and gravel. Gravel up to 1.5 inches in largest dimension was encountered in several borings (Pettit: BH-02 through BH-04 and BH-08; Cactus II: BH-02, BH-03 and BH-07) at various depths. All borings were able to penetrate the subsurface profile to the planned depths during current investigation. On Cactus II Feeder project, granitic bedrock (excavated as sand) was encountered in borings BH-16 and BH-17 at depth from 2.5 feet to the maximum explored depth.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-9, Logs of Borings, in Appendix A, Field Exploration. Relevant logs of borings from Cactus II Feeder project are also attached in Appendix A, Field Exploration as Appendix A-1, Relevant Logs of Borings for Cactus II Feeder.

5.3 Groundwater

Groundwater was not encountered in any borings during the current investigation. Groundwater was encountered in boring Cactus II: BH-02 at depth 23 feet bgs during Cactus II Feeder project investigation.

Regional groundwater data from the GeoTracker database (SWRCB, 2019) for well locations within close proximity to the project was reviewed to evaluate the current and historical groundwater levels.

- Circle K #1775 (ID No. T0606502742) located at the intersection of Heacock Street and John F Kennedy Drive reported groundwater depths ranging from approximately 14 to 20 feet bgs between 2009 and 2013.
- Mobil #18-A3E (ID No. T0606599291), located at the intersection of Alessandro Boulevard and Indian Street reported groundwater at depths ranging from approximately 35 to 49 feet bgs between 2001 and 2011.
- Arco #5208 (ID No. T0606562779) located at the intersection of Alessandro Boulevard and Perris Boulevard reported groundwater at depths ranging from approximately 27 to 43 feet bgs between 2005 and 2008.
- Tosco / 76 Station #6962 (ID No. T0606500535) located at the intersection of Alessandro Boulevard and Perris Boulevard reported groundwater at depths ranging from approximately 25 to 35 feet bgs between 2005 and 2010.

No further groundwater data within one mile of the project limits, east of Perris Boulevard was available from the GeoTracker database. The National Water Information System (USGS, 2019a) was also reviewed for available groundwater data. No wells were located within close proximity to the project.

Recent groundwater measurements have been taken at the intersection of Cactus Avenue and Heacock Street as well as Alessandro Boulevard and Kitcning Street in
wells installed to monitor groundwater for the Cactus II Feeder project. In both monitoring wells, groundwater was measured at approximately 21 feet BGS over the course of 3 consecutive months.

Based on the data reviewed, the historical high groundwater level in the vicinity of the western portion of the project (generally west of Kitching Street) is estimated to be approximately 14 feet bgs and the current depth is expected to be approximately 21 feet bgs. Groundwater may be encountered if groundwater rises to historic levels. The historical high groundwater depth for the eastern half of the project limits is not known with certainty but is estimated to be deeper than 25 feet bgs. Considering the depths of PRSs and pipelines, dewatering is not expected to be required during the construction of the Pettit RPZ project.

It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the project vicinity. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

5.4 Excavatability

The surface and subsurface soil materials for the Pettit RPZ project (except The PRS site 3 and the pipeline excavations along Lasselle Street) are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. Difficult excavation will occur where high concentration of gravel is encountered.

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.

The PRS site 3 and the pipeline excavations along Lasselle Street will encounter granitic bedrock of varying degrees of weathering at depths as shallow as 2.5 feet. Where encountered in the hollow-stem auger borings (Cactus II: BH-16 and BH-17) the bedrock was penetrated to the planned depth, indicating the bedrock is generally excavatable in those immediate areas. However, excavation will be difficult due to the presence of bedrock.

The use of specialized equipment or techniques, such as hydraulic hammers (“breakers”), jackhammers, blasting, or non-explosive rock reduction methods should be anticipated. Appropriate excavation equipment should be selected by an experienced earthwork contractor. Determination of the appropriate equipment may require test excavations in representative areas. Rock reduction techniques may be required to excavate in this area.
5.5 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 should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations.

6.0 GEOLOGIC SETTING

The regional and local geology are discussed in the following subsections.

6.1 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.

6.2 Local Geology

Regional mapping (Morton and Matti, 1996) indicates that the subsurface along the alignments and beneath the pressure reducing stations is primarily comprised of a combination of very old alluvial deposits with intermittent young alluvial deposits. The younger (Holocene to late Pleistocene-aged) alluvial-fan deposits consist of unconsolidated to moderately consolidated silt, sand, cobbles, and boulders. The older (early Pleistocene-aged) alluvial-fan deposits consist of mostly well-dissected, well-
indurated, reddish-brown sand deposits, containing minor gravel. The younger alluvial deposits are expected to be encountered at the PRS-2 location as well as at the connection pipe locations along Cactus Avenue from approximately 0.3 miles east of Heacock Street to 0.3 miles west of Kitching Street.

7.0 FAULTING AND SEISMICITY

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

7.1 Faulting

The project is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 1974; Riverside County, 2019). The nearest active fault is the San Jacinto fault, located approximately 4.5 miles northeast of the east end of the project.

7.2 CBC Seismic Design Parameters

Seismic parameters based on the California Building Code (CBSC, 2016) were determined at each PRS site using the Seismic Design Maps application (OSHPD, 2019) and are provided in the following table. These parameters will be applicable for pipeline segments (close to PRS sites).

Table No. 4, CBC 2016 Seismic Design Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Facility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PRS Site 1</td>
</tr>
<tr>
<td>Coordinates</td>
<td>33.9161°N, 117.2435°W</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_s$</td>
<td>1.506g</td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
<td>0.634g</td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(1)), $F_a$</td>
<td>1.0</td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(2)), $F_v$</td>
<td>1.5</td>
</tr>
<tr>
<td>MCE 0.2-sec period Spectral Response Acceleration, $S_{ms}$</td>
<td>1.506g</td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{m1}$</td>
<td>0.950g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for short period $S_{2s}$</td>
<td>1.004g</td>
</tr>
</tbody>
</table>
### Secondary Effects of Seismic Activity

Generally, in addition to ground shaking, effects of seismic activity on a project may include surface fault rupture, soil liquefaction, and settlement due to earthquake shaking, landslides, lateral spreading, tsunamis, seiches, 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:** The project is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 1974; Riverside County, 2019). The potential for surface rupture resulting from the movement of faults is not known with certainty but is considered low.

**Liquefaction:** 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 to 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

Riverside County has designated portions of the Pettit RPZ project as having low or moderate susceptibility to liquefaction, generally corresponding to the older and younger alluvial-fan units, respectively. Boring locations Pettit: BH-05 through BH-08 are in areas designated as having low susceptibility to liquefaction while boring locations Pettit: BH-01 through BH-04 are in an area designated as having moderate susceptibility to liquefaction.

No groundwater was encountered at any boring locations except at boring Cactus II: BH-02 at a depth of 23 feet bgs which will be utilized for the design of jumper pipe connection. Therefore, liquefaction potential is considered low for the entire project for current ground water condition.
**Dry Seismic Settlement:** Seismically induced settlement occurs in unsaturated, unconsolidated, granular sediments during ground shaking associated with earthquakes. Based on blow counts and pipe invert depths, the potential of dry seismic settlement along pipelines is considered low. Seismic settlement is considered low for PRS site 3 due to the presence of granitic bedrock. Dry seismic induced settlement analysis was performed based on soil data gathered from the boring Pettit: BH-08 (Cactus II: BH-02 was not analyzed as this boring will be utilized only for connection pipe), as presented in Appendix C, *Dry Seismic Settlement Analysis*. Based on analysis, negligible dry seismic settlement (up to 0.25 inches) was observed.

**Landslides:** Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The slopes along the Kitching Street flood control channel is concrete-lined and are not considered at risk for landsliding. The remainder of the project is not adjacent to any slopes at risk for landsliding.

**Lateral Spreading:** Seismically induced lateral spreading involves primarily lateral movement of earth materials over deeper layers which have liquefied due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved.

The slopes along the Kitching Street flood control channel slopes are concrete-lined and are not considered at risk for lateral spreading. Due to the relatively flat topography of the land within the project vicinity, the potential for lateral spreading is low.

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

**Seiches:** Seiches are large waves generated in enclosed bodies of water in response to ground shaking. The project crosses a flood control channel that runs parallel to Kitching Street. There is a potential for waves to overtop the channel during a large seismic event if moderate levels of water are present in the channel during the event.

**Earthquake-Induced Flooding:** Dams or other water-retaining structures may fail as a result of large earthquakes, resulting in flooding. At the closest point, the project is located approximately 3.8 miles from the northern tip of the Perris Lake Dam. Based on the position of surrounding hills and relative elevations, water would be conveyed to the south-southwest away from the project in the event of an earthquake-induced breach of the dam. The risk of earthquake-induced flooding is considered low.
8.0 LABORATORY TEST RESULTS

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

8.1 Physical Testing

- **In-situ Moisture and Dry Density** – *In-situ* dry density and moisture content of the sites and alignments soils were determined in accordance to ASTM D2216 and D7263. Dry densities of upper 15 feet soils within the project ranged from 99 to 125pcf with moisture contents of 2 to 12 percent. Results are presented in the log of borings in Appendix A, *Field Exploration*.

- **Expansion Index** – Three representative bulk soil samples were tested to evaluate expansion potential of soils in accordance with the ASTM D4829 test method. The measured expansion indices of the soil samples were 1, 11 and 27, indicating very low to low expansion potential.

- **Sand Equivalent** – Four representative bulk soil samples were tested to evaluate sand equivalent (SE) in accordance with the ASTM D2419 test method. The measured SE of the soil samples were 9, 14, 32 and 64. Sand equivalent greater than 30 is considered suitable for pipe bedding material.

- **Collapse Potential** – The collapse potential of two relatively undisturbed samples were tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The results showed a collapse of 0.3 and 1.3 percent, indicating a slight collapse potential.

- **R-Value** – Three representative bulk samples were tested to evaluate the R-value in accordance with California Test 301. The test results indicated R-values of 33, 36 and 65.

- **Grain Size Analysis** – Five representative samples were tested to determine their relative grain size distributions in accordance with the ASTM Standard D6913. Test results are graphically presented in Drawings No. B-1a, *Grain Size Distribution Result, in Appendix B, Laboratory Testing Program*.

- **Maximum Dry Density and Optimum Moisture Content** – Typical moisture-density relationship test of three representative soil samples were conducted in accordance with ASTM D1557. The test results are presented in Drawing No. B-2a, *Moisture-Density Relationship Results, in Appendix B, Laboratory Testing Program*. The laboratory maximum dry densities ranged from 133.0 to 134.0 pounds per cubic feet (pcf), with optimum moisture contents between 7.0 and 8.5 percent.

- **Direct Shear** – Three direct shear tests were performed on relatively undisturbed representative soil samples at soaked moisture conditions per ASTM D3080 methods. Results of the direct shear tests are presented in Drawings No. B-3
through B-5, Direct Shear Test Results in Appendix B, Laboratory Testing Program.

- Consolidation Test – Two consolidation tests were performed on relatively undisturbed samples of the project soils (PRS sites 1 and 4) in accordance to ASTM Standard D2435. The test results are shown on Drawings No. B-8 and B-9, Consolidation Test Results, in Appendix B, Laboratory Testing Program.

### 8.2 Chemical Testing - Corrosivity Evaluation

Four 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 project soils when placed in contact with common pipe materials. These tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Test Methods 643, 422, and 417. The test results are discussed below and are presented in Appendix B, Laboratory Testing Program.

- The pH measurements of the samples ranged from 7.9 to 9.2.
- The sulfate contents of the samples tested ranged from 0.0035 to 0.0052 percent by weight.
- The chloride concentrations of the samples tested ranged from 34 to 52 ppm.
- The minimum electrical resistivities when saturated ranged from 2,570 to 17,720 Ohm-cm.

### 9.0 EARTHWORK RECOMMENDATIONS

This section contains our general recommendations regarding earthwork and grading for the proposed pipelines and PRSs. 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. Recommendations for earthwork associated with the project are presented in the following subsections.

#### 9.1 General

The proposed PRSs will be founded on shallow foundations. The earthwork for the structures will include excavation of soil and fill placement.

Earthwork for the pipeline will include trench excavation, pipe subgrade preparation, pipeline bedding placement, and trench backfill following the placement of the pipe segment. Earthwork for the PRSs will also include excavation of soils for structural fill placement.
Prior to the start of construction, all underground existing utilities and appurtenances should be located at the project. 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, surface vegetation, deleterious material, existing fill, and surficial soils containing roots and perishable materials should be stripped and removed from the project. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

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

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 PRSs and Pavements Earthwork Recommendations

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

<table>
<thead>
<tr>
<th>Structure/Pavement</th>
<th>Minimum Excavation Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footings</td>
<td>24 inches below footings bottom or 5 feet below existing ground surface, whichever is deeper</td>
</tr>
<tr>
<td>Slabs-on-grade</td>
<td>18 inches below slab</td>
</tr>
<tr>
<td>Pavements</td>
<td>12 inches below finish grade</td>
</tr>
</tbody>
</table>

The overexcavation below the footings, slabs-on-grade and pavement should be uniform. The overexcavation should extend to at least 2 feet beyond the footprint of the footings and slabs and at least 1 foot beyond the edge of the pavement. The overexcavation bottom should be scarified and compacted as described in Section 9.4, *Compacted Fill Placement*.

Variations in the depths and the lateral extent of overexcavation are based on observations by the geotechnical consultant during grading and should be anticipated.
The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill or structures. If isolated pockets of very soft, loose, or pumping subgrade are encountered, the overexcavation should be locally deepened, as needed, to expose undisturbed, firm, and unyielding soils.

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

### 9.3 Fill Materials

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the project 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. Screening may be required to remove oversized particles from some on-site soils. On-site soils used as fill should meet the following criteria.

- Predominantly granular.
- 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 30 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Any imported fill should be tested and approved by geotechnical representative prior to delivery to the project. Imported materials, if required, should meet the above criteria prior to being used as structural backfill or engineered 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 project should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method, unless a higher compaction is specified herein. At least the upper 12 inches of subgrade soils below finish grade underneath pavement should be compacted to at least 95 percent of the laboratory maximum dry density.

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

To reduce differential settlement, variations in the soil type, degree of compaction and thickness of the compacted fill placed underneath the foundations should be kept to a minimum.

The project geotechnical consultant should observe the placement of fill and conduct in-place field density tests to check for adequate moisture content and relative compaction as required by the project specifications. Where less than the required relative compaction is indicated, additional compactive efforts should be applied and the soil moisture conditioned as necessary, until the required relative compaction is attained.

### 9.5 Site Drainage

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. The structure slabs should have a gradient of at least 2 percent towards drainage facilities. A desirable drainage gradient is 1 percent for paved areas and 2 percent in landscaped areas (if any). Surface drainage should be directed to suitable non-erosive devices.

### 9.6 Pipeline Recommendations

Earthwork associated with construction of underground utilities will include pipeline sub-grade preparation, pipe bedding placement, and trench backfill. Recommendations for these activities are provided in the following sections.

#### 9.6.1 Pipeline Subgrade Preparation

The final subgrade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles, larger than 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.

### 9.6.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to one foot above the pipe. Pipe bedding should follow the guideline of the City of Moreno Valley Modified Standard Plan MVSI–132E-1 *Water Line (up to 12" Dia.) Trench Backfill and Roadway Repair*, MVSI-132F-1 *Water Line (Larger than 12’ Dia.) Trench Backfill and Roadway Repair* or EMWD Standard Drawing B-286B (attached in Appendix D). Besides, additional information for pipe bedding are provided as below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate, or crushed rock may be used as pipe bedding material. The sand equivalents of the tested soils vary from 9 to 64. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

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

Bedding materials should be vibrated in-place to achieve compaction. Care should be taken to densify the bedding material below the springline of the pipe. Prior to placing the pipe bedding material, the pipe subgrade should be uniform and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

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

\[
\begin{align*}
\text{i. } \frac{D_{15}(F)}{D_{85}(B)} & \leq 5 \\
\text{ii. } \frac{D_{50}(F)}{D_{50}(B)} & < 25 \\
\text{iii. } \text{Bedding Materials must have less than 5 percent minus 75 µm (No. 200) sieve to avoid internal movement of fines.}
\end{align*}
\]

Where,

\[F = \text{Bedding Material}\]
9.6.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Trench backfill should follow the guideline of the City of Moreno Valley Modified Standard Plan MVSI–132E-1 Water Line (up to 12” Dia.) Trench Backfill and Roadway Repair, MVSI-132F-1 Water Line (Larger than 12’ Dia.) Trench Backfill and Roadway Repair or EMWD Standard Drawing B-286B (attached in Appendix D). Besides, additional trench backfill recommendations are presented below.

- Trench backfill should be compacted by mechanical methods, such as sheepfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein.
- 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.
- 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.

9.7 Backfill Recommendations Behind Subterranean Wall

Compaction of backfill adjacent to structural walls can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing...
the walls. The compaction of wall backfill should be conducted procedure described in section 9.4 *Compaction Fill Placement.*

**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 pipelines 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. Automatic shutoffs should be installed to limit the potential leakage in the event of damage in a seismic event.

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

**10.2 Shallow Footing Design Parameters**

The proposed PRS structures will be supported on continuous (strip) and/or isolated spread footings. Similar types of soils were observed in all PRS sites. The design of the shallow continuous or isolated spread footings should be based on the recommended parameters presented in the following table.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum continuous strip footing width</td>
<td>18 inches</td>
</tr>
<tr>
<td>Minimum isolated footing width</td>
<td>18 inches</td>
</tr>
<tr>
<td>Minimum continuous or isolated footing depth of embedment below lowest adjacent grade</td>
<td>18 inches</td>
</tr>
<tr>
<td>Allowable net bearing capacity</td>
<td>2,500 psf</td>
</tr>
</tbody>
</table>

The allowable net bearing capacity is defined as the maximum allowable net bearing pressure on the ground. It is obtained by dividing the net ultimate bearing capacity by a safety factor. The ultimate bearing capacity is the bearing stress at which ground fails by shear or experiences a limiting amount of settlement at the foundation. The net ultimate bearing capacity was obtained by subtracting the total overburden pressure on a horizontal plane at the foundation level from the ultimate bearing capacity.
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.

10.3 Lateral Earth Pressures and Resistance to Lateral Loads

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

10.3.1 Active Earth Pressures

The active earth pressure behind any buried wall or foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall or foundation inclination, surcharges, and any hydrostatic pressures. The recommended lateral earth pressures for the project are presented in the following table.

<table>
<thead>
<tr>
<th>Loading Conditions</th>
<th>Lateral Earth Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active earth conditions (wall is free to deflect at least 0.001 radian)</td>
<td>40</td>
</tr>
<tr>
<td>At-rest (wall is restrained)</td>
<td>60</td>
</tr>
</tbody>
</table>

These pressures assume a level ground surface behind the walls for a distance greater than the walls height, 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.

10.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 240 psf per foot of depth may be used for the sides of footings poured against recompacted soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,500 psf for compacted fill.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.
Due to the low overburden stress of the soil at shallow depth, the upper 1 foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

### 10.3.3 Seismic Earth Pressures

In addition to static lateral earth pressures, the walls should be designed for seismic lateral earth pressures during seismic shaking. For cantilever or flexible walls, we recommend the inverted triangle seismic earth pressure distribution of $28H$ in accordance with the Seed and Whitman (1970) procedure, where $H$ is the height of the backfill behind the wall. These lateral earth pressures assume a level ground surface around the structure for a distance greater than the structure height and no surcharge.

### 10.4 Soil Parameters for Pipe Design

Structural design of pipelines 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. 8, *Soil Parameters for Pipe Design*.

#### Table No. 8, Soil Parameters for Pipe Design

<table>
<thead>
<tr>
<th>Soil Parameters</th>
<th>Perris Blvd.</th>
<th>Lasselle St.</th>
<th>Pipeline Jumper Connections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average compacted fill total unit weight, $\gamma$ (pcf) (assume 92% relative compaction)</td>
<td>132</td>
<td>133</td>
<td>133</td>
</tr>
<tr>
<td>Angle of internal friction of soils, $\phi$</td>
<td>29</td>
<td>30</td>
<td>32</td>
</tr>
<tr>
<td>Soil cohesion, $c$ (psf)</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Coefficient of friction between formed concrete and native soils, $f_s$ (degree)</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Coefficient of friction between PVC pipe and native soils, $f_s$ (degree)</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Bearing pressure against native soils (psf)</td>
<td>2,500</td>
<td>2,500</td>
<td>2,500</td>
</tr>
<tr>
<td>Coefficient of passive earth pressure, $K_p$</td>
<td>2.88</td>
<td>3.00</td>
<td>3.25</td>
</tr>
<tr>
<td>Coefficient of active earth pressure, $K_a$</td>
<td>0.35</td>
<td>0.33</td>
<td>0.31</td>
</tr>
<tr>
<td>Modulus of Soil Reaction $E'$ (psi)</td>
<td>1,500</td>
<td>1,500</td>
<td>1,500</td>
</tr>
</tbody>
</table>
10.5 Bearing Pressure for Anchor and Thrust Blocks

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

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

10.6 Slabs-on-Grade

Slabs-on-grade should be supported on properly compacted fill. Compacted fill used to support slabs-on-grade should be placed and compacted in accordance with Section 9.4 Compacted Fill Placement.

Slabs-on-grade should have a minimum thickness of 8 inches for support of nominal live loads. Minimum top and bottom reinforcement for slabs-on-grade should be No. 5 reinforcing bars, spaced at 12-inches on-center each way. Structural design elements of slabs-on-grade, including but not limited to thickness, reinforcement, joint spacing of more heavily loaded slabs will be dependent upon the anticipated loading conditions and the modulus of subgrade reaction of the supporting materials and should be designed by a structural engineer.

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

The upper 12 inches of soil subgrade under slabs-on-grade should be moisture conditioned to 0 to 3 percent above optimum moisture content within 12 hours prior to placement of concrete.

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

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

Concrete should be cured by protecting it against loss of moisture and rapid temperature change for at least 7 days after placement. Moist curing, waterproof paper, white polyethylene sheeting, white liquid membrane compound, or a combination thereof may be used after finishing operations have been completed. The edges of
10.7 Soil Corrosivity

The results of chemical testing of four representative samples from the project were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, Laboratory Testing Program, and are discussed below.

The sulfate contents of the majority of the soils within the project limits correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-11, Table 4.2.1). ACI recommends a minimum compressive strength of 2,500 psi for exposure category S0 in ACI 318-11, Table 4.3.1. No concrete type restrictions are specified.

We anticipate that concrete structures such as vaults and equipment pads 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 318-11, Table 4.2.1). ACI provides concrete design recommendations in ACI 318-11, Table 4.3.1, including a minimum compressive strength of 2,500 psi, and a maximum chloride content of 0.3 percent.

The measured values of the minimum electrical resistivity when saturated ranged from 2,570 to 17,720 Ohm-cm. This indicates that the tested soils are mildly to moderately corrosive for ferrous metals in contact with the soil (Romanoff, 1957).

According to the Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive if the pH is 5.5 or less, or chloride content is 500 parts per million (ppm) or greater, or sulfate content is 1,500 ppm or greater, or resistivity less than 2000 ohm-cm. Based on the tested results, the soils within the project areas are not considered corrosive.

Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures, if necessary, for any ferrous metals in contact with the project areas soils.

10.8 Soil Expansion

Foundations and slabs at each PRS site (PRS 1 through 4), and all pipelines (except pipeline connection at PRS site 4) can be designed for very low expansive soil conditions (EI ≤ 20).
10.9 Settlement

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of the span of the structures.

Our analysis of the potential dry seismic settlement is presented in Appendix D, *Dry Seismic Settlement Analysis*. Based on our analysis, negligible dry seismic settlement (up to 0.25 inches) was observed. Differential settlement is estimated as half of the total settlement over a horizontal distance of the span of the structures.

The static and dynamic settlement estimates should not be combined for design purposes. The maximum combined static and dynamic settlement is not anticipated to exceed the maximum anticipated dynamic settlement.

10.10 Asphalt Concrete Pavement

Three representative soil samples were tested to determine the R-value of the subgrade soils. Based on laboratory testing, R-values were 33, 36 and 65. For pavement design, we have utilized an R-value of 30 and design Traffic Indices (TIs) ranging from 6.0 to 10.0

Based on the above information, asphalt concrete and aggregate base thickness results are determined using the Caltrans Highway Design Manual (Caltrans, 2017), 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.

<table>
<thead>
<tr>
<th>R-value</th>
<th>Traffic Index (TI)</th>
<th>Pavement Section</th>
<th>Option 1</th>
<th>Option 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Asphalt Concrete (inches)</td>
<td>Aggregate Base (inches)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>5.0</td>
<td>5.0</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>5.0</td>
<td>8.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>5.0</td>
<td>11.0</td>
<td>12.0</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>6.0</td>
<td>12.0</td>
<td>14.0</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>6.5</td>
<td>14.0</td>
<td>16.0</td>
</tr>
</tbody>
</table>

Pavement section should be based on the City of Moreno Valley Modified Standard Plan MVSI-132E-1 *Water Line (up to 12” Dia.) Trench Backfill and Roadway Repair*, MVSI-132F-1 *Water Line (Larger than 12” Dia.) Trench Backfill and Roadway Repair* or Table No. 9, *Recommended Preliminary Pavement Sections*, whichever is applicable. 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 Standards of the City of Moreno Valley or EMWD or Section 200-2.2,”Crushed Aggregate Base,” of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2018) and should be placed in accordance with Section 301-2 of the SSPWC.

Asphaltic concrete materials should conform to the Standard of the City of Moreno Valley or EMWD or Section 203 of the SSPWC and should be placed in accordance with Section 302-5 of the SSPWC.

11.0 CONSTRUCTION RECOMMENDATIONS

11.1 General

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

Vertical braced excavations are feasible for this project. Sloped excavations may not be feasible in locations adjacent to existing utilities or structures, pavements, utilities, channels, or other improvements. Recommendations pertaining to temporary excavations are presented in this section.

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, current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the owner’s representative. 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. The final determination of temporary slope
gradients should be based on review of the encountered soils by a competent person employed by the contractor, in accordance with Section 1541 of the OSHA Construction Safety Orders.

Table No. 10, Slope Ratios for Temporary Excavations

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>OSHA Soil Type</th>
<th>Depth of Cut (feet)</th>
<th>Recommended Maximum Slope (Horizontal:Vertical)¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (SP), Silty Sand (SM), Sand with Silt (SP-SM)</td>
<td>C</td>
<td>0-10</td>
<td>1.5:1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10-20</td>
<td>2:1</td>
</tr>
<tr>
<td>Sandy Silt (ML) and Sandy Clay (CL)</td>
<td>B</td>
<td>0-10</td>
<td>1:1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10-20</td>
<td>1.5:1</td>
</tr>
</tbody>
</table>

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

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 as necessary to protect the workers in the excavation. Surfaces exposed in sloped 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. 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. 11, Lateral Earth Pressures for Temporary Shoring

<table>
<thead>
<tr>
<th>Lateral Resistance Soil Parameters*</th>
<th>Perris Blvd.</th>
<th>Lasselle St.</th>
<th>Pipeline Jumper Connections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Earth Pressure (Braced Shoring) (psf) (A)</td>
<td>28</td>
<td>26</td>
<td>25</td>
</tr>
<tr>
<td>Active Earth Pressure (Cantilever Shoring) (psf) (B)</td>
<td>46</td>
<td>44</td>
<td>42</td>
</tr>
<tr>
<td>At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)</td>
<td>68</td>
<td>66</td>
<td>62</td>
</tr>
<tr>
<td>Passive earth pressure (psf per foot of depth) (D)</td>
<td>240</td>
<td>250</td>
<td>270</td>
</tr>
<tr>
<td>Maximum allowable bearing pressure against native soils (psf) (E)</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
</tr>
<tr>
<td>Coefficient of friction between sheet pile and native soils, fs (degree) (F)</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
</tbody>
</table>

*Parameters A through F are used in Figures No. 3 and 4 on the next page.

Restrained (braced) shoring systems should be designed based on Figure No. 5, *Lateral Earth Pressure for Temporary Braced Excavation* to support a uniform rectangular lateral earth pressure.

Figure No. 5, Lateral Earth Pressures for Temporary Braced Excavation

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. 6, *Lateral Earth Pressures on Temporary Cantilever Wall*.
Earth 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 of time and in alternating sections.

12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Converse 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.

Converse should be present to observe conditions during construction. Testing should be performed to determine density and moisture of the soils during pipeline and PRS 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 KRIEGER & STEWART, EMWD and its authorized agents, to assist in the design and construction of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

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

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

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

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


CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2016, California Building Code (CBC).


CONVERSE CONSULTANTS, 2019, Final Geotechnical Data Report, Cactus II Feeder Project, Riverside County, California, dated February 27, 2019.


Appendix A

Field Exploration
APPENDIX A

FIELD EXPLORATION

Our field investigation included reconnaissance of the project areas and a subsurface exploration program consisting of drilling soil borings. Proposed boring locations were reviewed and approved by EMWD. An encroachment permit was obtained from the City of Moreno Valley prior to commencement of drilling. During the reconnaissance, the surface conditions were noted, and the boring locations were marked along the street using nearby landmarks as a guide. The boring locations should be considered accurate only to the degree implied by the method used to identify them in the field.

Eight exploratory borings (Pettit: BH-1 through BH-8) were drilled between March 5 and 6, 2019, at locations approved by EMWD. The borings were drilled to depths ranging from approximately 11.5 to 51.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations. Due to proximity of existing underground utilities, hand auger was utilized for most of the borings up to 5 feet bgs during drilling.

Thirty-three exploratory borings (Cactus II: BH-01 through BH-35) were drilled between February 15 and February 22, 2017 and two additional borings were drilled on June 21, 2018 at locations approved by Black & Veatch to investigate the subsurface conditions for the Cactus II feeder project. Relevant borings (Cactus II: BH-01 through BH-04, BH-07, BH-16, BH-17 and BH-30) which are in close proximity to current project location have been utilized in this project by Converse.

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soil sampling. Encountered earth materials were continuously logged by a Converse geologist and visually classified in the field in accordance with the Unified Soil Classification System. Where appropriate, 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 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 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory.

Standard Penetration Tests (SPTs) were performed in borings Pettit: BH-08 and Cactus II: BH-02 at selected depths 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 each 6 inches of sampler penetration are shown on the boring logs. The standard penetration tests were performed in accordance with the ASTM Standard...
D1586-84 test method. Bulk samples retrieved inside the SPT sampler were collected in zip-lock plastic bags for delivery to our laboratory.

In addition to drive samples, representative bulk samples were collected from selected depths within the borings. Bulk samples were obtained from drill cuttings and placed in large plastic bags for delivery to our laboratory.

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 driven samples are indicated in the log at the top of the next drive sample.

Following the completion of logging and sampling, all borings were backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete, where necessary. 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 (if possible).

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. Logs of the exploratory borings are presented in Drawings No. A-2 through A-9, *Logs of Borings*.

Relevant logs of borings from Cactus II Feeder project are attached in Appendix A-1, *Relevant Logs of Borings of Cactus II Feeder*. 
SOIL CLASSIFICATION CHART

MAJOR DIVISIONS

COARSE GRAINED SOILS

MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE

GRAVEL AND GRAVELLY SOILS

MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE

GRAVELS WITH FINES

( аппречирабл амовант оф финэс)

SAND AND SANDY SOILS

MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE

SANDS WITH FINES

( аппречирабл амовант оф финэс)

FINE GRAINED SOILS

MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE

SILTS AND CLAYS

LIQUID LIMIT LESS THAN 50

SILTS AND CLAYS

LIQUID LIMIT GREATER THAN 50

HIGHLY ORGANIC SOILS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST

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

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE

(Results shown in Appendix B)

STRENGTH

Pocket Penetrometer p

Direct Shear ds

Direct Shear (single point) ds'

Unconfined Compression uc

Triaxial Compression tx

Consolidation c

Collapse Test col

Resilience (R) Value r

Chemical Analysis ca

Electrical Resistivity or

Permeability perm

Soil Cement sc

CONSISTENCY

Very Soft Soft Medium Stiff Very Stiff Hard

SPT (N) < 2 2 - 6 7 - 12 13 - 25 > 26 > 30

CA Sampler < 3 3 - 8 9 - 15 16 - 30 > 30
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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>MOISTURE</th>
<th>BULK</th>
<th>OTHER</th>
</tr>
</thead>
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<td></td>
<td></td>
<td>4/8/13</td>
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<td>115</td>
<td>ei,ca,er ma,max</td>
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<td></td>
<td>9/13/18</td>
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<tr>
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<td></td>
<td>4/10/16</td>
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<td>117</td>
<td>c</td>
</tr>
<tr>
<td>15-20</td>
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<td></td>
<td>15/32/50</td>
<td>12</td>
<td>117</td>
<td>c</td>
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<td>20-25</td>
<td></td>
<td></td>
<td>10/16/21</td>
<td>20</td>
<td>107</td>
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6.5" ASPHALT CONCRETE / 8" AGGREGATE BASE

ALLUVIUM
SANDY CLAY (CL): fine to medium-grained sand, brown.

SILTY SAND (SM): fine to coarse-grained, trace clay, reddish-brown.

SANDY CLAY (CL): fine to coarse-grained sand, brown.

End of boring at 21.5 feet bgs.
No groundwater encountered.
**Log of Boring No. BH-02**

Dates Drilled: 3/6/2019  
Logged by: William Buckley  
Checked By: James Burnham

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

Ground Surface Elevation (ft): 1547  
Depth to Water (ft): NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
<th>SAMPLES</th>
</tr>
</thead>
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<td></td>
<td></td>
<td></td>
<td>DRIVE</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>5&quot; ASPHALT CONCRETE / 9&quot; AGGREGATE BASE</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>ALLUVIUM</td>
<td>4/6/9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SILTY SAND (SM): fine to coarse-grained, some gravel up to 1.5&quot; in largest dimension, brown.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- scattered gravel up to 1&quot; in largest dimension</td>
<td></td>
</tr>
</tbody>
</table>
| 10        |             | End of boring at 11.5 feet bgs.  
No groundwater encountered.  

Pettit Regulated Pressure Zone  
City of Moreno Valley, Riverside County, California  
For: Eastern Municipal Water District

Converse Consultants  
Project No: 18-81-312-01  
Drawing No: A-3
Log of Boring No. BH-03

Dates Drilled: 3/6/2019  Logged by: William Buckley  Checked By: James Burnham

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

Ground Surface Elevation (ft): 1553  Depth to Water (ft): NOT ENCONTRATED

<table>
<thead>
<tr>
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<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
</tr>
</thead>
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<td>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.</td>
</tr>
</tbody>
</table>

8.5" ASPHALT CONCRETE / 14" AGGREGATE BASE

ALLUVIUM
Silty Sand (SM): fine to medium-grained, scattered gravel up to 1.5" in largest dimension, brown.

Silty Sand (SM): fine to coarse-grained, brown.

End of boring at 11.5 feet bgs.
No groundwater encountered.
Log of Boring No. BH-04

Dates Drilled: 3/5/2019
Logged by: William Buckley
Checked By: James Burnham

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

Ground Surface Elevation (ft): 1550
Depth to Water (ft): NOT ENCOUNTERED

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.

5.5” ASPHALT CONCRETE / 4” AGGREGATE BASE

ALLUVIUM
Silty Sand (SM): fine to medium-grained, trace clay, brown.

- fine to coarse-grained, scattered gravel up to 0.5” in largest dimension

End of boring at 11.5 feet bgs.
No groundwater encountered.

Converse Consultants
City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District

Project No. 18-81-312-01
Drawing No. A-5
### Log of Boring No. BH-05

**Dates Drilled:** 3/5/2019  
Logged by: William Buckley  
Checked By: James Burnham  
Equipment: 8" HOLLOW STEM AUGER  
Driving Weight and Drop: 140 lbs / 30 in  
Ground Surface Elevation (ft): 1547  
Depth to Water (ft): NOT ENCOUNTERED

<table>
<thead>
<tr>
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<tr>
<td>5.5&quot; ASPHALT CONCRETE / 4&quot; AGGREGATE BASE</td>
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</tr>
<tr>
<td>ALLUVIUM</td>
<td></td>
</tr>
<tr>
<td>SILTY SAND (SM): fine to medium-grained, trace clay, brown.</td>
<td></td>
</tr>
<tr>
<td>- fine to coarse-grained, reddish-brown</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SAMPLES</th>
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</thead>
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</tr>
<tr>
<td>BLOWS</td>
</tr>
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<td>MOISTURE</td>
</tr>
<tr>
<td>DRY UNIT WT. (pcf)</td>
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<tr>
<td>OTHER</td>
</tr>
<tr>
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</tr>
<tr>
<td>6/9/12</td>
</tr>
<tr>
<td>9/20/34</td>
</tr>
</tbody>
</table>

End of boring at 11.5 feet bgs.  
No groundwater encountered.  
End of boring at 11.5 feet bgs.
No groundwater encountered.
### 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.

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#### 5.5" ASPHALT CONCRETE / 14" AGGREGATE BASE

**ALLUVIUM Silty Sand (SM):** fine to medium-grained, trace clay, brown.

<table>
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<tr>
<th>Date</th>
<th>Drive</th>
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<th>Moisture</th>
<th>Dry Unit Wt.</th>
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**Information:**

- Dates Drilled: 3/6/2019
- Logged by: William Buckley
- Checked By: James Burnham
- Equipment: 8" Hollow Stem Auger
- Driving Weight and Drop: 140 lbs / 30 in
- Ground Surface Elevation (ft): 1543
- Depth to Water (ft): NOT ENCOUNTERED
**Log of Boring No. BH-08 (PRS Site 4)**

**Dates Drilled:** 3/5/2019  
**Logged by:** William Buckley  
**Checked By:** James Burnham

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

**Ground Surface Elevation (ft):** 1540  
**Depth to Water (ft):** NOT ENCOUNTERED

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### SUMMARY OF SUBSURFACE CONDITIONS

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#### 4.5" ASPHALT CONCRETE OVER 9" BASE

- **ALLUVIUM**
  - **SANDY CLAY (CL):** fine to medium-grained sand, dark brown.
  - **SAND (SP):** fine to coarse-grained, few gravel up to 1.5" in largest dimension, light brown.

- **SAND with SILT (SP-SM):** fine to coarse-grained, scattered gravel up to 1.5" in largest dimension, light brown.

- **Silty Sand (SM):** fine to coarse-grained, trace clay, reddish-brown.

#### SAMPLES

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<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
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**Petit Regulated Pressure Zone**  
City of Moreno Valley, Riverside County, California  
For: Eastern Municipal Water District

**Project ID:** 18-81-312-01.GPJ; **Template:** LOG

**Converse Consultants**  
Project No. 18-81-312-01  
Drawing No. A-9a
**Log of Boring No. BH-08 (PRS Site 4)**

Dates Drilled: 3/5/2019  
Logged by: William Buckley  
Checked By: James Burnham

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

Ground Surface Elevation (ft): 1540  
Depth to Water (ft): NOT ENCOUNTERED

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<table>
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**SUMMARY OF SUBSURFACE CONDITIONS**

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**ALLUVIUM**

**Silty Sand (SM):** fine to coarse-grained, scattered gravel up to 1" in largest dimension, light brown.

**Sandy Silt (ML):** fine-grained sand, trace clay, brown.

- Fine to coarse-grained sand

**Silt (SM):** fine to coarse-grained, light brown.

End of boring at 51.5 feet bgs.  
No groundwater encountered.  

**Samples**

- **DRIVE**
- **BULK**
- **MOISTURE**
- **DIEY UNIT WT. (pcf)**
- **OTHER**

<table>
<thead>
<tr>
<th>Sample</th>
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<th>DWT.</th>
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**Converse Consultants**

City of Moreno Valley, Riverside County, California

For: Eastern Municipal Water District

Project No. 18-81-312-01  
Drawing No. A-9b
Appendix A-1

Relevant Logs of Borings for Cactus Feeder II*

(*Utilized for the design of Pettit RPZ PRS No. 2 and 3 as well as the pipeline segments on Perris Boulevard and Lasselle Street and pipeline jumper connections west of Indian Street along Cactus Avenue)
**Log of Boring No. BH-01 (Turnout Facility No. 1)**

- **Dates Drilled:** 2/21/2017
- **Logged by:** William Buckley
- **Checked By:** Scot Mathis
- **Equipment:** 8" HOLLOW STEM AUGER
- **Driving Weight and Drop:** 140 lbs / 30 in
- **Ground Surface Elevation (ft):** 1549
- **Depth to Water (ft):** NOT ENCLOSED

**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.

<table>
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<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>Reddish-brown</td>
</tr>
<tr>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>15</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>0</td>
<td>-</td>
</tr>
</tbody>
</table>

**ALLUVIUM:**

- **Silty Sand (SM):** fine to medium-grained, reddish-brown.
- **Sandy Silt (ML):** fine to medium-grained sand, brown.

**Drives**

<table>
<thead>
<tr>
<th>Date</th>
<th>Number</th>
<th>Moisture</th>
<th>Bulk</th>
<th>Blows</th>
<th>Unit WT.</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/18/24</td>
<td>5</td>
<td>107</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4/50-5&quot;</td>
<td>5</td>
<td>105</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28/24/18</td>
<td></td>
<td></td>
<td></td>
<td>se, ca, er</td>
<td></td>
</tr>
<tr>
<td>6/9/13</td>
<td>3</td>
<td>93</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25/50-5&quot;</td>
<td>13</td>
<td>108</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6/7/7</td>
<td>15</td>
<td>117</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10/17/18</td>
<td>15</td>
<td>116</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**End of boring at 26.5 feet bgs.**

No groundwater encountered.

Borehole backfilled with soil cuttings and tamped on 2/21/17.
### Log of Boring No. BH-02

**Dates Drilled:** 2/15/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1548  
**Depth to Water (ft):** 23

#### 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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>5&quot; ASPHALT CONCRETE / 4&quot; AGGREGATE BASE</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
</table>
|           | 5"         | ALLUVIUM:  
Silty Sand (SM): fine to medium-grained, few gravel up to 1.5" in largest dimension, brown. | | | | | | | |
|           | 5"         | Sandy Silt (ML): fine to coarse-grained sand, reddish-brown. | | | | | | | |
|           | 10"        | - cuttings become moist  
Silty Sand (SM): fine to medium-grained, trace clay, brown. | | | | | | | |
|           | 15"        | Sandy Silt (ML): fine-grained sand, brown. | | | | | | | |
|           | 20"        | 6/5/4 | | | | | | | |
|           | 25"        | 12/24/45 | | | | | | | |
|           | 30"        | 5/9/14 | | | | | | | |

**MDGS:** Converse Consultants  
**Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities**  
**City of Moreno Valley, Riverside County, California**  
**For: Black & Veatch**

**Project No.:** 15-81-158-01  
**Drawing No.:** A1-3a
Log of Boring No. BH-02

Dates Drilled: 2/15/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1548  
Depth to Water (ft): 23

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>SILTY SAND (SM):</strong> fine to medium-grained, brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- fine to coarse-grained</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 51.5 feet bgs.  
Groundwater was encountered at 23 feet bgs.  
Borehole backfilled with soil cuttings, tamped and patched with cold asphalt concrete on 2/15/17.
SUMMARY OF SUBSURFACE CONDITIONS

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5" ASPHALT CONCRETE / 5" AGGREGATE BASE

ALLUVIUM:

SILTY SAND (SM): fine to medium-grained, few gravel up to 1" in largest dimension, brown.

SANDY SILT (ML): fine to medium-grained sand, reddish-brown.

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and patched with cold asphalt concrete on 2/15/17.

Ground Surface Elevation (ft): 1551
Depth to Water (ft): NOT ENCOUNTERED

5/5/3
3/3/4
4/8/11
6/11/42
5" ASPHALT CONCRETE / 5" AGGREGATE BASE
### Log of Boring No. BH-04

#### Dates Drilled: 2/15/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1555  
Depth to Water (ft): NOT ENCONTRUED

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#### SUMMARY OF SUBSURFACE CONDITIONS

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#### 5" ASPHALT CONCRETE / 5" AGGREGATE BASE

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Alluvium: Silty Sand (SM):</th>
<th>Sample</th>
<th>moisture</th>
<th>dry unit wt. (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>fine to medium-grained, few gravel up to 1&quot; in largest dimension, brown.</td>
<td>3/4/9</td>
<td>3</td>
<td>124</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Sand with Silt (SP-SM):</th>
<th>Sample</th>
<th>moisture</th>
<th>dry unit wt. (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td></td>
<td>fine to coarse-grained, light brown to brown.</td>
<td>6/6/8</td>
<td>2</td>
<td>100</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>8/13/20</td>
<td>2</td>
<td>dist.</td>
</tr>
<tr>
<td>15.5</td>
<td></td>
<td></td>
<td>14/17/25</td>
<td>13</td>
<td>118</td>
</tr>
</tbody>
</table>

End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings, tamped and patched with cold asphalt concrete on 2/15/17.
### 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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>DRIVE</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>3/8/8</td>
</tr>
<tr>
<td>10</td>
<td>3/2/4</td>
</tr>
<tr>
<td>15</td>
<td>3/5/12</td>
</tr>
<tr>
<td>16.5</td>
<td>7/16/19</td>
</tr>
</tbody>
</table>

**5" ASPHALT CONCRETE / 4" AGGREGATE BASE**

**ALLUVIUM:**
- **Silty Sand (SM):** fine-grained, few gravel up to 1.5" in largest dimension, brown.

**Sandy Silt (ML):** fine-grained sand, brown.

**Silty Sand (SM):** fine to medium-grained, gravel up to 1" in largest dimension, brown.

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and patched with cold asphalt concrete on 2/16/17.
Log of Boring No. BH-16

Dates Drilled: 2/20/2017
Logged by: William Buckley
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1581  Depth to Water (ft): NOT ENCOUNTERED

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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>ALLUVIUM: SILTY SAND (SM): fine to medium-grained, brown.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>- fine to coarse-grained, light brown</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GRANITIC BEDROCK: Excavates as SAND (SP): fine to coarse-grained, orangish-brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- olive-brown</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>End of boring at 15.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped on 2/20/17.</td>
</tr>
</tbody>
</table>

Note: The table includes sample data for moisture and dry unit weight. The exact values are not visible in the image.
Log of Boring No. BH-17

Dates Drilled: 2/20/2017  Logged by: William Buckley  Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1595  Depth to Water (ft): NOT ENCOUNTERED

---

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.

### ALLUVIUM:

**Silty Sand (SM):** fine to coarse-grained, light brown to brown.

### GRANITIC BEDROCK:

**Excavates as Sand (SP):** fine to coarse-grained, orangish-brown.

- light brown to brown

- no recovery

End of boring at 15.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 2/20/17.
**Log of Boring No. BH-30**

Dates Drilled: 2/22/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1608  
Depth to Water (ft): NOT ENCOUNTERED

---

**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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td></td>
</tr>
<tr>
<td>10-15</td>
<td></td>
</tr>
<tr>
<td>15-20</td>
<td></td>
</tr>
<tr>
<td>20-25</td>
<td></td>
</tr>
<tr>
<td>25-30</td>
<td></td>
</tr>
</tbody>
</table>

**SAMPLES**

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>DRIVER</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/5/6</td>
<td>8</td>
<td>111</td>
<td></td>
<td></td>
<td>ma, max, ei</td>
</tr>
<tr>
<td>5/8/9</td>
<td>4</td>
<td>106</td>
<td></td>
<td></td>
<td>se, ca, er</td>
</tr>
<tr>
<td>4/9/15</td>
<td>10</td>
<td>120</td>
<td></td>
<td></td>
<td>col, ds</td>
</tr>
<tr>
<td>4/17/19</td>
<td>6</td>
<td>109</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6/15/26</td>
<td>2</td>
<td>116</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**ALLUVIUM:**

**Silty Sand (SM):** fine to medium-grained, brown.

**CLAYEY SAND (SC):** fine to medium-grained, brown. - reddish-brown

**Silty Sand (SM):** fine to coarse-grained, trace clay, reddish-brown.

End of boring at 26.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 2/22/17.
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 the Pettit RPZ project. Relevant laboratory test results of Cactus II Feeder project are also included.

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

In-situ dry density and moisture content tests were performed on relatively undisturbed ring samples, in accordance to ASTM Standard D2216 and ASTM D7263 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the project soils. For test results, see the Logs of Borings in Appendix A, Field Exploration.

**Expansion Index**

Five representative bulk samples (Pettit: 3; Cactus II: 2) were tested to evaluate the expansion potential of the materials encountered. The tests were conducted in accordance with ASTM Standard D4829. Test results are presented in the following table.

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Expansion Index</th>
<th>Expansion Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>11</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-04/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>1</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-08/Pettit</td>
<td>15-20</td>
<td>Silty Sand (SM), trace Clay</td>
<td>27</td>
<td>Low</td>
</tr>
<tr>
<td>BH-01/Cactus II</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-30/Cactus II</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
</tbody>
</table>

**Sand Equivalent**

Seven representative soil samples (Pettit: 4; Cactus II: 3) 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-2, Sand Equivalent Test Results

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Sand Equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-02/Pettit</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>32</td>
</tr>
<tr>
<td>BH-04/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>14</td>
</tr>
<tr>
<td>BH-06/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM), Trace Clay</td>
<td>9</td>
</tr>
<tr>
<td>BH-08/Pettit</td>
<td>5-10</td>
<td>Sand (SP)</td>
<td>64</td>
</tr>
<tr>
<td>BH-01/Cactus II</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>15</td>
</tr>
<tr>
<td>BH-16/Cactus II</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>15</td>
</tr>
<tr>
<td>BH-30/Cactus II</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>13</td>
</tr>
</tbody>
</table>

**Soil Corrosivity**

Seven representative soil samples (Pettit: 4; Cactus II: 3) 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 sites and alignments soils when placed in contact with common construction materials. These tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance to Caltrans Test Methods 643, 422 and 417. Test results are presented in the following table.

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

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>pH</th>
<th>Soluble Sulfates (CA 417) (% by weight)</th>
<th>Soluble Chlorides (CA 422) (ppm)</th>
<th>Min. Resistivity (CA 643) (Ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01/Pettit</td>
<td>5-10</td>
<td>9.2</td>
<td>0.0036</td>
<td>37</td>
<td>3.597</td>
</tr>
<tr>
<td>BH-03/Pettit</td>
<td>5-10</td>
<td>7.9</td>
<td>0.0035</td>
<td>34</td>
<td>17,720</td>
</tr>
<tr>
<td>BH-05/Pettit</td>
<td>1-5</td>
<td>9.2</td>
<td>0.0047</td>
<td>34</td>
<td>7,788</td>
</tr>
<tr>
<td>BH-08/Pettit</td>
<td>15-20</td>
<td>8.9</td>
<td>0.0052</td>
<td>52</td>
<td>2,570</td>
</tr>
<tr>
<td>BH-01/Cactus II</td>
<td>5-10</td>
<td>8.4</td>
<td>0.0010</td>
<td>235</td>
<td>1,800</td>
</tr>
<tr>
<td>BH-16/Cactus II</td>
<td>5-10</td>
<td>7.9</td>
<td>0.0010</td>
<td>175</td>
<td>5,300</td>
</tr>
<tr>
<td>BH-01/Cactus II</td>
<td>5-10</td>
<td>8.5</td>
<td>0.0010</td>
<td>125</td>
<td>5,200</td>
</tr>
</tbody>
</table>

**Collapse**

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, four collapse tests (Pettit: 2; Cactus II: 2) were performed in accordance with the ASTM Standard D4546 laboratory procedure. Each sample was loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test results including collapse from consolidation tests are presented in the following table.
### Table No. B-4, Collapse Test Results

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>Soil Classification</th>
<th>Percent Swell (+)</th>
<th>Percent Collapse (-)</th>
<th>Collapse Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-03/Pettit</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>-1.3</td>
<td></td>
<td>Slight</td>
</tr>
<tr>
<td>BH-06/Pettit</td>
<td>7.5-9.0</td>
<td>Silty Sand (SM)</td>
<td>-0.5</td>
<td></td>
<td>Slight</td>
</tr>
<tr>
<td>BH-01*/Pettit</td>
<td>15.0-16.5</td>
<td>Silty Sand (SM)</td>
<td>-0.2</td>
<td></td>
<td>Slight</td>
</tr>
<tr>
<td>BH-08*/Pettit</td>
<td>15.0-16.5</td>
<td>Silty Sand (SM)</td>
<td>-1.0</td>
<td></td>
<td>Slight</td>
</tr>
<tr>
<td>BH-01/Cactus II</td>
<td>7.5-9.0</td>
<td>Silty Sand (SM)</td>
<td>-2.9</td>
<td></td>
<td>Moderate</td>
</tr>
<tr>
<td>BH-30/Cactus II</td>
<td>7.5-9.0</td>
<td>Silty Sand (SM)</td>
<td>-2.1</td>
<td></td>
<td>Moderate</td>
</tr>
</tbody>
</table>

(*from consolidation test)

### R-value

Four representative bulk soil samples (Pettit: 3; Cactus II: 1) were tested for resistance value (R-value) in accordance with the Caltrans Test Method 301. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table.

### Table No. B-5, R-Value Test Results

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>Soil Classification</th>
<th>Measured R-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-02/Pettit</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>65</td>
</tr>
<tr>
<td>BH-05/Pettit</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>33</td>
</tr>
<tr>
<td>BH-07/Pettit</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>36</td>
</tr>
<tr>
<td>BH-02/Cactus II</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>35</td>
</tr>
</tbody>
</table>

### Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analyses were performed on nine select samples (Pettit: 5; Cactus II: 4) in accordance with the ASTM Standard D6913 test method. Grain-size curves are shown in Drawings No. B-1a and B-1b, *Grain Size Distribution Results*, and in the following table.

### Table No. B-6, Grain Size Distribution Test Results

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (ft)</th>
<th>Soil Classification</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>2.0</td>
<td>54.0</td>
<td>44.0</td>
<td></td>
</tr>
<tr>
<td>BH-02/Pettit</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>9.0</td>
<td>73.0</td>
<td>18.0</td>
<td></td>
</tr>
<tr>
<td>BH-04/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>1.0</td>
<td>60.0</td>
<td>39.0</td>
<td></td>
</tr>
<tr>
<td>BH-06/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>1.0</td>
<td>50.0</td>
<td>49.0</td>
<td></td>
</tr>
<tr>
<td>BH-08/Pettit</td>
<td>5-10</td>
<td>Sand (SP)</td>
<td>7.0</td>
<td>88.4</td>
<td>4.6</td>
<td></td>
</tr>
</tbody>
</table>
### Maximum Density and Optimum Moisture Content

Four laboratory maximum dry density and optimum moisture content relationship tests (Pettit: 3; Cactus II: 1) were performed on representative bulk samples. The tests were conducted in accordance with the ASTM Standard D1557 test method. Test results are presented on Drawing Nos. B-2a and 2b, *Moisture-Density Relationship Results*, and in the following table.

**Table No B-7, Summary of Moisture-Density Relationship Results**

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Optimum Moisture (%)</th>
<th>Maximum Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM), reddish-brown</td>
<td>8.0</td>
<td>133.0</td>
</tr>
<tr>
<td>BH-03/Pettit</td>
<td>5-10</td>
<td>Silty Sand (SM), brown</td>
<td>7.0</td>
<td>133.0</td>
</tr>
<tr>
<td>BH-07/Pettit</td>
<td>1-5</td>
<td>Silty Sand (SM), brown</td>
<td>8.5</td>
<td>134.0</td>
</tr>
<tr>
<td>BH-30/Cactus II</td>
<td>0-5</td>
<td>Silty Sand (SM), brown</td>
<td>8.2</td>
<td>134.5</td>
</tr>
</tbody>
</table>

### Direct Shear Tests

Five direct shear tests (Pettit: 3; Cactus II: 2) were performed on a relatively undisturbed representative soil sample under soaked moisture conditions, in accordance with the ASTM D3080 method. For the test, three samples contained in a brass sampler ring were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 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 results, including sample density and moisture content, see Drawings No. B-3 through B-7, *Direct Shear Test Results*, and in the following table.

**Table No. B-8, Summary of Direct Shear Test Results**

<table>
<thead>
<tr>
<th>Boring No./Project</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Ultimate Strength Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Friction Angle</td>
</tr>
<tr>
<td>BH-01/Pettit</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM)</td>
<td>34</td>
</tr>
<tr>
<td>BH-05/Pettit</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>35</td>
</tr>
</tbody>
</table>
Consolidation Tests

Two tests (PRS site 1 and 4) were conducted in accordance with ASTM Standard D2435 method. Data obtained from these tests performed on relatively undisturbed ring samples were 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 Drawings No. B-8 and B-9, Consolidation Test Results.

Sample Storage

Soil samples currently stored in our laboratory will be discarded thirty days after the date of the final report, unless this office receives a specific request to retain the samples for a longer period.
### GRAIN SIZE DISTRIBUTION RESULTS

#### Project ID: 15-81-158-01.GPJ; Template: GRAIN SIZE

**Cactus II Feeder**
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>Description</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Cc</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-04</td>
<td>1-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-16</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-30</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>1.0-5.0</td>
<td>12.5</td>
<td>0.449</td>
<td>0.449</td>
<td>3.0</td>
<td>63.0</td>
<td>34.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-04</td>
<td>1-5</td>
<td>4.75</td>
<td>0.434</td>
<td>0.434</td>
<td>0.0</td>
<td>64.0</td>
<td>33.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-16</td>
<td>0-5</td>
<td>9.5</td>
<td>0.358</td>
<td>0.358</td>
<td>2.3</td>
<td>65.6</td>
<td>31.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-30</td>
<td>0-5</td>
<td>9.5</td>
<td>0.334</td>
<td>0.334</td>
<td>1.0</td>
<td>62.4</td>
<td>36.6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Converse Consultants**

Project No. 15-81-158-01
Drawing No. B-1b
### MOISTURE-DENSITY RELATIONSHIP RESULTS

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>BORING NO.</th>
<th>DEPTH (ft)</th>
<th>DESCRIPTION</th>
<th>ASTM TEST METHOD</th>
<th>OPTIMUM WATER, %</th>
<th>MAXIMUM DRY DENSITY, pcf</th>
</tr>
</thead>
<tbody>
<tr>
<td>●</td>
<td>BH-01</td>
<td>5-10</td>
<td>SILTY SAND (SM), REDDISH-BROWN</td>
<td>D1557 - B</td>
<td>8.0</td>
<td>133.0</td>
</tr>
<tr>
<td>■</td>
<td>BH-03</td>
<td>5-10</td>
<td>SILTY SAND (SM), BROWN</td>
<td>D1557 - B</td>
<td>7.0</td>
<td>133.0</td>
</tr>
<tr>
<td>▲</td>
<td>BH-07</td>
<td>1-5</td>
<td>SILTY SAND (SM), BROWN</td>
<td>D1557 - B</td>
<td>8.5</td>
<td>134.0</td>
</tr>
</tbody>
</table>
### MOISTURE-DENSITY RELATIONSHIP RESULTS

- **Project No.:** 15-81-158-01
- **Drawing No.:** B-2b

#### SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf
--- | --- | --- | --- | --- | --- | ---
● | BH-30 | 0-5 | SILTY SAND (SM), BROWN | D1557 - B | 8.2 | 134.5

Curves of 100% Saturation for Specific Gravity Equal to:
- 2.80
- 2.70
- 2.60

#### WATER CONTENT, %

- 0 5 10 15 20 25 30

#### DRY DENSITY, pcf

- 90 95 100 105 110 115 120 125 130 135 140 145 150
Project No. 18-81-312-01

Converse Consultants
Pettit Regulated Pressure Zone
City of Moreno Valley, Riverside County, California
For: Eastern Municipal Water District
Project ID: 18-81-312-01.GPJ; Template: DIRECT SHEAR

NOTE: Ultimate Strength.

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DESCRIPTION</th>
<th>DEPTH (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>SILTY SAND (SM)</td>
<td>10.0-11.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>COHESION (psf)</th>
<th>FRICTION ANGLE (degrees)</th>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>140</td>
<td>34</td>
<td>12.0</td>
<td>117.3</td>
</tr>
</tbody>
</table>
DESCRIPTION:

SURCHARGE PRESSURE, psf

Project No. 18-81-312-01

5.0-6.5

DRY DENSITY (pcf)

DIRECT SHEAR TEST RESULTS

BORING NO. : BH-05

DEPTH (ft) : 5.0-6.5

DESCRIPTION : SILTY SAND (SM)

COHESION (psf) : 60

FRICTION ANGLE (degrees): 35

MOISTURE CONTENT (%) : 9.2

DRY DENSITY (pcf) : 124.6

NOTE: Ultimate Strength.
**DIRECT SHEAR TEST RESULTS**

---

**Pettit Regulated Pressure Zone**  
City of Moreno Valley, Riverside County, California  
For: Eastern Municipal Water District

Project No. 18-81-312-01  
Drawing No. B-5

---

**SURCHARGE PRESSURE, psf**

---

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-08</th>
<th>DEPTH (ft)</th>
<th>10.0-11.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESCRIPTION</td>
<td>SAND WITH SILT (SP-SM)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>COHESION (psf)</td>
<td>10</td>
<td>FRICTION ANGLE (degrees):</td>
<td>37</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>3.4</td>
<td>DRY DENSITY (pcf) :</td>
<td>118.1</td>
</tr>
</tbody>
</table>

**NOTE:** Ultimate Strength.

---

**CONVERSE CONSULTANTS**  
18-81-312-01.GPJ; Template: DIRECT SHEAR
**DIRECT SHEAR TEST RESULTS**

Cactus II Feeder  
Eastern Municipal Water District  
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-01</th>
<th>DEPTH (ft)</th>
<th>2.5-4.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESCRIPTION</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>COHESION (psf)</td>
<td>260</td>
<td>FRICTION ANGLE (degrees):</td>
<td>37</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>5.3</td>
<td>DRY DENSITY (pcf) :</td>
<td>107.3</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
### DIRECT SHEAR TEST RESULTS

**Cactus II Feeder**  
Eastern Municipal Water District  
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch  

**Project No.:** 15-81-158-01  
**Drawing No.:** B-7  

#### BORING NO.: BH-30  
**DEPTH (ft):** 7.5-9.0  
**DESCRIPTION:** SILTY SAND (SM)  
**COHESION (psf):** 110  
**FRICION ANGLE (degrees):** 31  
**MOISTURE CONTENT (%):** 8.0  
**DRY DENSITY (pcf):** 111.0  

**NOTE:** Ultimate Strength.
### CONSOLIDATION TEST RESULTS

**Project No.:** 18-81-312-01  
**Drawing No.:** B-8

**Description:** Silty Sand (SM)

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>BH-01</th>
<th>Depth (ft)</th>
<th>15.0-16.5</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Silty Sand (SM)</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Percent Saturation</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>12</td>
<td>117</td>
<td>76</td>
</tr>
<tr>
<td>Final</td>
<td>16</td>
<td>116</td>
<td>100</td>
</tr>
</tbody>
</table>

**Note:** Solid circles indicate readings after addition of water.
NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

BORING NO.: BH-08
DEPTH (ft): 15.0-16.5

DESCRIPTION: SILTY SAND (SM)

<table>
<thead>
<tr>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>PERCENT SATURATION</th>
<th>VOID RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL</td>
<td>9</td>
<td>120</td>
<td>63</td>
</tr>
<tr>
<td>FINAL</td>
<td>15</td>
<td>116</td>
<td>100</td>
</tr>
</tbody>
</table>
Appendix C

Dry Seismic Settlement Analysis
APPENDIX C

DRY SEISMIC SETTLEMENT ANALYSIS

The subsurface data obtained from the boring BH-08 (PRS site 4) during the current field investigation was used to evaluate the dry seismic settlement due to potential densification of relatively loose sediments subjected to ground shaking during earthquakes.

The analysis was performed using the program SPTLIQ (InfraGEO Software, 2018). An earthquake magnitude of M7.3 and a peak ground acceleration (PGA) of 0.604g, where g is the acceleration due to gravity, were selected for this analysis. The PGA was based on the CBC seismic design parameters presented in Section 7.2, CBC Seismic Design Parameters. An analysis considering current groundwater condition was performed for the boring.

The results of our analyses are presented on Plate C-1 and summarized in the following table.

Table C-1, Estimated Dry Seismic Settlement

<table>
<thead>
<tr>
<th>Location</th>
<th>Groundwater Conditions (feet bgs)</th>
<th>Dynamic Settlement (inches)</th>
<th>Differential Dynamic Settlement (inch/40 linear feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-08</td>
<td>&gt;50</td>
<td>0.25</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Based on our analysis, the site has the potential for up to 0.25 inches of dry seismic settlement. Differential settlement is estimated as half of the total settlement over a horizontal distance of the span of the structures.
Simplified liquefaction hazards assessment using standard penetration test (SPT) data

**PROJECT INFORMATION**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Name</td>
<td>Petit Regulated Pressure Zone</td>
</tr>
<tr>
<td>Project No.</td>
<td>18-81-312-01</td>
</tr>
<tr>
<td>Project Location</td>
<td>City of Moreno Valley</td>
</tr>
<tr>
<td>Analyzed By</td>
<td>Zahangir Alam</td>
</tr>
<tr>
<td>Reviewed By</td>
<td></td>
</tr>
</tbody>
</table>

**TOPOGRAPHIC CONDITIONS**

| Ground Slope, S     | 0.00 %                         |
| Free Face (L/H) Ratio| N/A H = 15 feet                |

**GROUNDWATER LEVEL DATA**

| GWL Depth Measured During Test | 55.00 feet |
| GWL Depth Used in Design      | 55.00 feet |

**SEISMIC DESIGN PARAMETERS**

| Earthquake Moment Magnitude, Mw | 7.30 |
| Peak Ground Acceleration, Amax  | 0.64 g |
| Required Factor of Safety, FS   | 1.20 |

**SEISMIC DESIGN PARAMETERS**

- Borehole Diameter: 8.00 inches
- Hammer Weight: 140.00 pounds
- Hammer Drop: 30.00 inches
- Hammer Energy Efficiency Ratio, ER: 80.0 %
- Hammer Distance to Ground Surface: 5.00 feet

**BORING DATA**

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>BH-08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Surface Elevation</td>
<td>1,540.00 feet</td>
</tr>
<tr>
<td>Proposed Grade Elevation</td>
<td>1,540.00 feet</td>
</tr>
<tr>
<td>Borehole Diameter</td>
<td>8.00 inches</td>
</tr>
<tr>
<td>Hammer Weight</td>
<td>140.00 pounds</td>
</tr>
<tr>
<td>Hammer Drop</td>
<td>30.00 inches</td>
</tr>
<tr>
<td>Hammer Energy Efficiency Ratio, ER</td>
<td>80.0 %</td>
</tr>
<tr>
<td>Hammer Distance to Ground Surface</td>
<td>5.00 feet</td>
</tr>
</tbody>
</table>

**GROUNDWATER LEVEL DATA**

<table>
<thead>
<tr>
<th>Soil Depth During Test (feet)</th>
<th>0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75</th>
</tr>
</thead>
<tbody>
<tr>
<td>N60</td>
<td><em>SPT N60</em></td>
</tr>
<tr>
<td>(N1)60cs ; FC (%)</td>
<td><em>SPT (N1)60cs ; FC (%)</em></td>
</tr>
<tr>
<td>CSR = Cyclic Stress Ratio; CRR = Cyclic Resistance Ratio</td>
<td></td>
</tr>
<tr>
<td>Factor of Safety, FS</td>
<td>0.00 0.25 0.50 0.75 1.00 1.50 2.00</td>
</tr>
<tr>
<td>Seismic Settlement (in.)</td>
<td>0.00 0.05 0.10 0.15 0.20 0.25 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00</td>
</tr>
<tr>
<td>Cyclic Lateral Disp. (in.)</td>
<td>0.00 0.25 0.50 1.00</td>
</tr>
<tr>
<td>Lateral Spreading (in.)</td>
<td>0.00 0.50 1.00</td>
</tr>
</tbody>
</table>

**SPT N-values and Fines Content**

- SPT N60
- (N1)60cs ; FC (%)

**Analysis Methods Used**

- Liquefaction Triggering: Boulanger-Idriss (2014)
- Lateral Spreading: Zhang et al. (2004)
Appendix D

Pipe Bedding, Trench Backfill & Street Sections
UNSURFACED MEDIANs, ROADSIDe STRIPS, & EASEMENTS

BACKFILL COMPACTED TO 90% RELATIVE COMPACTION (MAXIMUM LIFT THICKNESS IS 6 INCHES).

BACKFILL COMPACTED TO 90% RELATIVE COMPACTION, MAXIMUM LIFT THICKNESS WHEN PONDING AND JETTING IS 2 FEET.

BACKFILL AND UTILITY BACKFILL PER UTILITY COMPANY OR MANUFACTURER'S SPECIFICATION.

CLEARANCE "A"

1. PIPE SIZES THROUGH 12"; "A" = 6" - 9"

2. PIPE SIZES OVER 12"; "A" = 1' - 0" MIN.

I. STRUCTURAL ZONE
II. INTERMEDIATE ZONE
III. PIPE AND UTILITY ZONE

SURFACING (EXCLUDING GRAVEL ROADS).

BASE MATERIAL AND SUB- GRADE, COMPACTED TO 95% RELATIVE COMPACTION.

REPLACE STRUCTURAL SECTION AS FOLLOWS:

SURFACING - EXISTING THICKNESS PLUS 1" OR 3" MINIMUM
BASE - CLASS I, THICKNESS SHALL EQUAL EXISTING Base MATERIAL, 4" MIN.
OR AS SPECIFIED

NOTE: WHEN A FIRM FOUNDATION IS NOT ENCOUNTERED, DUE TO SOFT, SPONGY, OR OTHER UNSUITABLE MATERIAL, SUCH MATERIAL SHALL BE REMOVED TO THE LIMITS DIRECTED BY THE ENGINEER, AND THE RESULTING EXCAVATION BACKFILLED WITH PIPE BEDDING MATERIAL COMPACTED TO 90% RELATIVE COMPACTION.
FINISH OVERLAY TO BE PLACED NO LATER THAN 15 DAYS AFTER BASE PAVING. APPLY TACK AND LEVELING COURSE TO BRING WITHIN 0.10' OF EXISTING GRADE. SURFACE COURSE SHALL BE TYPE III C3-PG64-10 AC. COLD MILL EXISTING PAVEMENT 1.5' DEEP.

FOR PARALLEL TRENCH FULL LANE WIDTH (TO NEAREST STRIPING) (SEE NOTE 6)
FOR PERPENDICULAR TRENCH, MIN WIDTH = 8 FEET
MATCH EXIST PAVEMENT + 1'
4" MINIMUM
1.5"

BASE COURSE SHALL BE TYPE III B2-PG64-10 AC

EXISTING PAVEMENT
MATCH EXIST AGGREGATE
BASE - 4" MIN IF EXIST IS 0" TO 4" (SEE NOTE 7)

TRENCH BACKFILL:
PROCESSED MISCELLANEOUS BASE BACKFILL OR 1-1/2 SACK CEMENT SLURRY BACKFILL SHALL BE PLACED PRIOR TO THE END OF EACH WORK DAY UNLESS OTHERWISE APPROVED BY THE CITY ENGINEER. ALL COMPACTION SHALL BE BY MECHANICAL METHODS ONLY. FLOODING OR PONDING WILL NOT BE ALLOWED UNLESS AUTHORIZED BY CITY ENGINEER. SEE SECTION 306.6.5.1, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, LATEST EDITION.

W = OD + 12" MIN (EQUAL DIST EA SIDE)
OD + 24" MAX
4" MIN (SEE NOTE NO 4)

BEDDING MATERIAL SHALL MEET OR EXCEED SECTION 306-6, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, LATEST EDITION. LOCALLY EXCAVATED NATIVE MATERIALS MAY BE BLENDED TO THE REQUIRED SAND EQUIVALENCY OF 30 OR GREATER.

NOTES:

1) SEE ADDITIONAL REQUIREMENTS, STD No MVSI-132C.

2) ALL TRENCHING AND BACKFILL SHALL BE DONE IN ACCORDANCE WITH SECTION 306, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, LATEST EDITION.

3) ALL TEMPORARY PAVING SHALL HAVE A MINIMUM 2" OF AC ON LOCAL STREETS AND 3" ON ALL OTHERS.

4) INCREASE BEDDING UNDER PIPE FROM 4" TO 6" FOR ROCK SUBGRADES.

5) USE THIS STANDARD PLAN FOR UP TO AND INCLUDING 12" DIAMETER WATER LINE WITH 36" COVER OVER PIPE.

6) LANE WIDTH REQUIREMENT MAY BE REDUCED AT DISCRETION OF CITY ENGINEER.

7) 1-1/2 SACK CEMENT SLURRY MAY BE USED IF USED FOR TRENCH BACKFILL.
NOTES:

1.) SEE ADDITIONAL REQUIREMENTS, STD No MVSI-132C.

2.) ALL TRENCHING AND BACKFILL SHALL BE DONE IN ACCORDANCE WITH SECTION 306, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, LATEST EDITION.

3.) ALL TEMPORARY PAVING SHALL HAVE A MINIMUM 2" OF AC ON LOCAL STREETS AND 3" ON ALL OTHERS.

4.) INCREASE BEDDING UNDER PIPE FROM 4" TO 6" FOR ROCK SUBGRADES.

5.) USE THIS STANDARD PLAN FOR WATER LINE OF GREATER THAN 12" DIAMETER. MINIMUM COVER OVER PIPE IS 48".

6.) LANE WIDTH REQUIREMENT MAY BE REDUCED AT DISCRETION OF CITY ENGINEER.

7.) 1-1/2 SACK CEMENT SLURRY MAY BE USED IF USED FOR TRENCH BACKFILL.

NOT TO SCALE
Appendix E

Seismic Refraction Survey
SEISMIC REFRACTION SURVEY
CACTUS II FEEDER
MORENO VALLEY, CALIFORNIA

PREPARED FOR:
Converse Consultants
10391 Corporate Drive
Redlands, CA 92373

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

March 17, 2017
Project No. 117078
March 17, 2017
Project No. 117078

Mr. James Burnham
Converse Consultants
10391 Corporate Drive
Redlands, CA  92373

Subject: Seismic Refraction Survey
Cactus II Feeder
Moreno Valley, California

Dear Mr. Burnham:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Cactus II Feeder project located in Moreno Valley, California. Specifically, our survey consisted of performing seven seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions please contact the undersigned at your convenience.

Sincerely,

SOUTHWEST GEOPHYSICS, INC.

Afrildo Iko Syahrial
Project Geologist/Geophysicist

AIS/HV/hv

Distribution: Addressee (electronic)
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Figure 4f – Seismic Profile, SL-6
Figure 4g – Seismic Profile, SL-7
1. **INTRODUCTION**

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Cactus II Feeder project located in Moreno Valley, California (Figure 1). Specifically, our survey consisted of performing seven seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

2. **SCOPE OF SERVICES**

Our scope of services included:

- Performance of seven seismic P-wave refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results, conclusions and recommendations.

3. **SITE DESCRIPTION**

The project site is located along Alessandro Boulevard roughly between Kitching Street and Darwin Drive in Moreno Valley, California (Figure 1). The seismic lines were located along the dirt shoulder adjacent to Alessandro Boulevard. Topography immediately adjacent to the roadway is generally flat; however, hills with exposures of granitic rock are nearby particularly in the area of lines SL-3 through SL-6. Figures 2, 3a and 3b depict the site conditions in the area of the seismic traverses.

Based on our discussions with you it is our understanding that the project involves the construction of a new water pipeline. Cuts up to 15 feet deep may be performed.

4. **SURVEY METHODOLOGY**

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the rippability characteristics of the subsurface materials and to develop subsurface velocity profiles of the areas surveyed. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves
generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Seven seismic lines (SL-1 through SL-7) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogeneous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assume that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock rippability. These characteristics may also vary with location and depth.
For trenching operations, the ripability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

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<th>Seismic P-wave Velocity</th>
<th>Rippability</th>
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<td>0 to 2,000 feet/second</td>
<td>Easy</td>
</tr>
<tr>
<td>2,000 to 4,000 feet/second</td>
<td>Moderate</td>
</tr>
<tr>
<td>4,000 to 5,500 feet/second</td>
<td>Difficult, Possible Blasting</td>
</tr>
<tr>
<td>5,500 to 7,000 feet/second</td>
<td>Very Difficult, Probable Blasting</td>
</tr>
<tr>
<td>Greater than 7,000 feet/second</td>
<td>Blasting Generally Required</td>
</tr>
</tbody>
</table>

It should be noted that the ripability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2011). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the ripability of the on-site materials prior to submitting their bids.

5. **ANALYSIS AND RESULTS**

As previously indicated, seven seismic traverses were conducted as part of our study. The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Figures 4a through 4g present the velocity models generated from our study. The approximate locations of the seismic refraction traverses are shown on the Line Location Map (Figure 2). In
general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

6. **CONCLUSIONS AND RECOMMENDATIONS**

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil overlying granitic bedrock with varying degrees of weathering. Distinct vertical and lateral velocity variations are evident in the models. These inhomogeneities are likely related to the presence of remnant boulders, intrusions and differential weathering of the bedrock materials. It is also evident in the tomography models that the depth to bedrock is highly variable across the site.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. Furthermore, blasting may be required depending on the excavation depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

7. **LIMITATIONS**

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophys-
ics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties’ sole risk.
8. SELECTED REFERENCES


Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

TOMOGRAPHY MODEL

SEISMIC PROFILE
SL-2
Cactus II Feeder
Moreno Valley, California

Note: Contour Interval = 1,000 feet per second

Figure 4b
TOMOGRAPHY MODEL

Note: Contour Interval = 1,000 feet per second
B2 - Cactus II Feeder Project
FINAL GEOTECHNICAL DATA REPORT

CACTUS II FEEDER PROJECT
EASTERN MUNICIPAL WATER DISTRICT
CITY OF MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 15-81-158-01

Prepared For:
BLACK & VEATCH
Mr. Jeremy Clemmons, PE
Engineering Manager
300 Ranchero Drive, Suite 250
San Marcos, CA 92069

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

February 27, 2019
February 27, 2019

Mr. Jeremy Clemmons, PE
Engineering Manager
Black & Veatch
300 Ranchero Drive, Suite 250
San Marcos, CA 92069

Subject: FINAL GEOTECHNICAL DATA REPORT
Cactus II Feeder Project
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
Converse Project No. 15-81-158-01

Dear Mr. Clemmons:

Converse Consultants (Converse) is pleased to submit this final geotechnical data report to assist with the design and construction of the Eastern Municipal Water District (EMWD) Cactus II Feeder pipeline and turnout facilities located within the City of Moreno Valley, Riverside County, California. The report was prepared to incorporate findings and recommendations for the Cactus II Feeder pipeline alignment and turnout facilities. This report was prepared in accordance with our revised proposal dated April 23, 2018 and your Amendment No. 3 dated June 5, 2018.

Based upon our field investigation, laboratory data, and analyses, the proposed pipeline and turnout facilities are considered feasible from a geotechnical standpoint, provided that 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 Black & Veatch and EMWD. Should you have any questions, please do not hesitate to contact us at 909-796-0544.

CONVERSE CONSULTANTS

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

Dist: 4/Addresssee
HSQ/SM/JB/ZA/kvg
PROFESSIONAL CERTIFICATION

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

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

Zahangir Alam, PhD, EIT
Senior Staff Engineer

James Burnham, PG
Project Geologist

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

Scot Mathis, PG, CEG
Senior Geologist
EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions, and recommendations, as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed Cactus II Feeder project consists of approximately 31,000 linear feet of pipeline, to be installed along Cactus Avenue, Indian Street, Alessandro Boulevard, Moreno Beach Drive, Perris Boulevard, and Nason Street in the City of Moreno Valley, Riverside County, California.

- The majority of the pipeline will be installed using the open cut-and-cover technique with a typical pipe cover of 4 to 10 feet. The pipe will consist of 42, 36, and 30-inch diameter cement mortar lined and coated welded steel pipe (CML/CMC).

- Bore-and-jack trenchless construction methods will be utilized to cross the Kitching Street Flood Control Channel. The jacking and receiving pits will be approximately 23.5 feet deep. Bore-and-jack trenchless construction is also being considered at the intersection of Alessandro Boulevard and Perris Boulevard; and at potentially large storm drain or sewer crossings. Potential crossings include the three 30-inch storm drains located on Alessandro Boulevard between Pearl Lane and Oliver Street, and the 78-inch storm drain and other utility crossings at the intersection of Alessandro Boulevard and Morrison Street.

- Four turnout facilities associated with the project will consist of at-grade concrete pads for the support of above ground pipe and equipment. The concrete pads are planned to be 12-inches-thick and rest on a 6-inch thick layer of ¾-inch crushed rock. The turnout facilities will have concrete masonry unit (CMU) wall along the property limit and a 24-foot access gate.

- Our scope of work included project set-up, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report.

- Thirty-three exploratory borings (BH-01 through BH-33) were drilled between February 15 and February 22, 2017, at locations approved by Black & Veatch to investigate the subsurface conditions. The borings were drilled to depths ranging from approximately 15.5 to 51.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations. Additional 2 exploratory borings (BH-34 and BH-35) were drilled
on June 21, 2018 to their maximum planned depths of 16.5 and 51.5 feet bgs due to the relocation of turnout facility no. 2 and connecting pipeline.

- A seismic refraction survey consisting of 7 lines was conducted by Southwest Geophysics, Inc. The results of the survey are presented in the text of the report and in Appendix C.

- Based on the exploratory borings, the subsurface materials primarily consisted of alluvial sediments consisting of primarily sand and silt mixtures with scattered gravel. Weathered granitic bedrock was encountered in borings BH-14 through BH-18 (Alessandro Street between Kitching Street and Darwin Road) at depths ranging from 2.5 to 25 feet bgs. The weathered granitic bedrock excavates as silty sand and sand. Borings BH-14 through BH-18 were able to penetrate the weathered bedrock to the planned depths.

- Bedrock is not anticipated in the excavations up to 20 feet bgs on Cactus Avenue, Indian Street, Alessandro Boulevard west of Kitching Street, Alessandro east of Darwin Drive, Nason Street, and Moreno Beach Drive. We anticipate that the sediments underlying this area will be readily excavatable with conventional trenching equipment, including excavators and trenching machines.

- Based on the seismic refraction and soil boring data, the depth at which difficult excavation will be encountered is expected to vary significantly along Alessandro Boulevard between Kitching Street and Darwin Drive, sometimes over short distances. In general, the bedrock is expected to be excavatable to depths of approximately 10 feet with easy to moderate difficulty between Kitching Street and 200 feet west of Lasselle Street. The bedrock along Alessandro Boulevard between 200 feet west of Lasselle Street and Darwin Drive is expected to be excavatable to 10 feet with difficulty ranging from moderate to extremely difficult (where blasting is generally required).

- Groundwater was encountered during the investigation in borings BH-02 and BH-35 at depths of 23 and 37 feet bgs, respectively. Groundwater was not encountered in any other boring during the investigation. The historical high groundwater level in the vicinity of the western half of the alignment (generally west of Kitching Street) is estimated to be approximately 14 feet bgs. Two groundwater monitoring wells were installed in September 2018, one west of Kitching Street (MW-1) and one in the Heacock/Pettit Booster Pump Station site (MW-2). Based on five months of monitoring, groundwater is at an approximate depth of 20.5 feet bgs and 19.8 feet bgs for MW-1 and MW-2, respectively. The historical high groundwater depth for the eastern half of the alignment is not known with certainty but is estimated to be deeper than 25 feet bgs. Groundwater may be encountered in the jacking and receiving pits. Groundwater
is not expected to be encountered elsewhere throughout the project. Perched groundwater may be present locally.

- The site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone. The nearest active fault is the San Jacinto fault, located approximately 2.5 miles northeast of the east end of the alignment. The alignment is located in a seismically active region. Ground shaking from earthquakes associated with nearby and distant faults may occur during the lifetime of the project. Seismic coefficients derived from the 2016 California Building Code are presented in the text of this report.

- The potential impact to the project alignment from surface fault rupture, land sliding lateral spreading, tsunamis, and earthquake-induced flooding is considered to be low.

- Riverside County has designated portions of the alignment as having low or moderate susceptibility to liquefaction, generally corresponding to the older and younger alluvial-fan units, respectively. Turnout Facilities No. 1, 2, and 3 are located in areas designated as having low susceptibility to liquefaction while Turnout Facility No. 4 is in an area designated as having moderate susceptibility to liquefaction.

- The maximum dynamic settlement at the turnout structures and along the alignment is estimated to be 3.3 inches, except for Alessandro Boulevard between Kitching Street and Darwin Drive, where the potential for dry seismic settlement is considered low due to the presence of shallow bedrock.

- The alignment crosses a flood control channel that runs parallel to Kitching Street. When moderate levels of water are present in the channel, there is a potential for waves to overtop the channel during a large seismic event.

- The expansion indices (EI) of soil samples ranged from 0 to 7, corresponding to very low expansion potential (except at location BH-11 where medium expansion potential (EI of 51) were observed). The collapse potentials of the tested soils ranged from 0.7 to 2.9 percent, indicating slight to moderate collapse potential.

- The sulfate contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations. No concrete type restrictions are specified for exposure category S0. A minimum compressive strength of 2,500 psi is recommended. The chloride contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category C1 (concrete is exposed to moisture, but not to external sources of chlorides). For exposure category C1, ACI provides concrete compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.
The measured values of the minimum electrical resistivity when saturated ranged from 1,200 to 17,000 Ohm-cm along the majority of the proposed alignment. This indicates that the majority of the soils along the proposed alignment are corrosive for ferrous metals in contact with the soil. A corrosion engineer should be consulted for corrosion mitigation measures for ferrous metals in contact with the soil.

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

Earthwork for the pipeline includes pipe trench excavation, pipe subgrade preparation, and backfilling of the trench following the placement of the pipe. Excavated site soils free of particles larger than 3 inches and deleterious matter may be used for backfill. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse grained soil and between optimum and above 2 percent of optimum for fine grained soil, then placed in horizontal layers not exceeding loose lifts of 8 inches. All backfill material should be compacted to a minimum of 90 percent of the laboratory maximum dry density. The upper one foot of backfill beneath the pavement sections should be compacted to at least 95 percent of the laboratory maximum dry density.

The proposed turnout facilities will include at-grade structures. The earthwork for the structures will include excavation of soil and fill placement. Structural footprints at the turnout structures should be overexcavated to at least 2 feet below the bottom of the deepest footing. The depth of overexcavation should be uniform across the entire structures. The overexcavation should extend laterally at least 5 feet beyond the at-grade footprints.

All areas to receive asphalt or concrete pavement should be overexcavated to a depth of 12 inches below subgrade. The overexcavation should extend to limits of the paving required by the City of Moreno Valley.

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 40 feet. We estimate that the turnout facilities have the potential for up to 3.3 inches of dynamic settlement during a large earthquake. Differential settlement can be estimated as half of the total settlement over a distance of 40 horizontal feet.

Anticipated soil conditions at each bore-and-jack crossing and recommendations for backfill of the jacking and receiving pits are provided in the text of this report.
Allowable net bearing capacities, lateral earth pressures, pipeline design parameters and asphalt concrete pavement sections are presented in the text of this report. Slope ratios for temporary excavations and shoring recommendations are also provided in the text of this report.

The results of our investigation indicate that the proposed project is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are considered and implemented in the design and construction of the pipeline and turnout facilities.
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1.0 INTRODUCTION

This report contains the findings of the geotechnical investigation performed by Converse for Cactus II Feeder project (pipeline and turnout facilities) located along several streets within the City of Moreno Valley, Riverside County, California. The vicinity of the project area is shown in Figure No. 1, Approximate Vicinity Map. The owner of the pipeline is Eastern Municipal Water District (EMWD). The approximate location of the proposed pipeline alignment and turnout facilities are shown in Figure No. 2, Approximate Alignment and Turnout Facilities Location Map.

The purpose of this investigation was to evaluate the nature and engineering properties of the subsurface soils and groundwater conditions, and to provide geotechnical recommendations for the design and construction of the proposed pipelines and turnout facilities.

This report was prepared for the project described herein and is intended for use solely by Black & Veatch, EMWD, and its authorized agents. This report may be made available to the prospective bidders for bidding purposes. However, the bidders are responsible for their own interpretation of the site conditions between and beyond the boring locations, based on factual data contained in this report. This report may not contain sufficient information for use by others and/or other purposes.

2.0 PROJECT DESCRIPTION

The Cactus II Feeder project will consist of approximately 31,000 linear feet of 30-inch to 42-inch diameter water pipeline. The pipeline construction will also include 4 turnout facilities. The description of the pipeline and the turnout facilities are summarized in the following sections.

Pipeline

The pipeline alignment and pipeline details are summarized in the Table No. 1, Pipeline Alignment Details. We understand that a majority of the pipeline will be installed using the open cut-and-cover technique and a typical depth to the top of the pipe of 4 to 10 feet below ground surface.

Bore-and-jack trenchless construction methods will be utilized to cross the Kitching Street Flood Control Channel. The 36-inch pipeline will be installed in a 54-inch diameter steel casing. The jacking and receiving pits will be approximately 23.5 feet deep.

Bore-and-jack trenchless construction is also being considered at the intersection of Alessandro Boulevard and Perris Boulevard; and at potentially large storm drain or sewer crossings. Potential crossings include the three 30-inch storm drains located on
Converse Consultants

Approximate Vicinity Map

Cactus II Feeder
Approximately 29,700 linear feet of Pipeline
City of Moreno Valley, Riverside County, California

Project No.
15-81-158-01

Figure No.
1
Explanation

Existing Cactus I Feeder

Planned Pipeline Alignment

OLIVER ST
MORENO BEACH DR
CACTUS AVE
QUINCY ST

Converse Consultants

Approximate Alignment Location Map

Cactus II Feeder
Approximately 29,700 linear feet of Pipeline
City of Moreno Valley, Riverside County, California

Figure No. 2

Project No. 15-B1-158-01
Alessandro Boulevard between Pearl Lane and Oliver Street, and the 78-inch storm drain and other utility crossings at the intersection of Alessandro and Morrison Street.

**Table No. 1, Pipeline Alignment Details**

<table>
<thead>
<tr>
<th>Roadway</th>
<th>From</th>
<th>To</th>
<th>Approximate Length (linear feet)</th>
<th>Pipe diameter (inch)</th>
<th>Pipe Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cactus Avenue</td>
<td>Heacock Street</td>
<td>Indian Street</td>
<td>2,500</td>
<td>42</td>
<td>Cement-Mortar Lined and Coated (CML&amp;C) Steel Pipe</td>
</tr>
<tr>
<td>Indian Street</td>
<td>Cactus Avenue</td>
<td>Alessandro Boulevard</td>
<td>2,550</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>Alessandro Boulevard</td>
<td>Indian Street</td>
<td>Moreno Beach Drive</td>
<td>18,720</td>
<td>42,36, and 30*</td>
<td></td>
</tr>
<tr>
<td>Perris Boulevard</td>
<td>Alessandro Boulevard</td>
<td>350 feet North of Bay Avenue</td>
<td>1,700</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>Nason Street</td>
<td>Alessandro Boulevard</td>
<td>Cottonwood Avenue</td>
<td>3,325</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Moreno Beach Drive</td>
<td>Alessandro Boulevard</td>
<td>Cactus Avenue</td>
<td>2,550</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

*The pipe diameter transitions 42 inches to 36 inches east of Perris Boulevard and from 36 inches to 30 inches east of Nason Street

**Turnout Facilities**

There are 4 turnout facilities associated with the proposed project. The location of each facility is summarized in the following table.

**Table No. 2, Turnout Facility Details**

<table>
<thead>
<tr>
<th>Turnout Facility No.</th>
<th>Location</th>
<th>Approximate Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turnout Facility No. 1</td>
<td>Southeast of Cactus Avenue and Heacock Street intersection</td>
<td>70’x80’</td>
</tr>
<tr>
<td>Turnout Facility No. 2</td>
<td>East of Perris Boulevard approximately 350 feet north of Bay Avenue</td>
<td>70’x80’</td>
</tr>
<tr>
<td>Turnout Facility No. 3</td>
<td>South of Alessandro Boulevard, approximately 500 feet east of the Alessandro Boulevard and Nason Street intersection</td>
<td>75’x75’</td>
</tr>
<tr>
<td>Turnout Facility No. 4</td>
<td>South of Alessandro Boulevard approximately 380 feet west of Moreno Beach Drive</td>
<td>75’x75’</td>
</tr>
</tbody>
</table>

The turnout facilities will consist of at-grade concrete pads for the support of above ground pipe and equipment. The concrete pads are planned to be 12-inches-thick and rest on a 6-inch thick layer of ¾-inch crushed rock. The turnout facilities will have an 8-foot to 11-foot-high concrete masonry unit (CMU) wall along the property limit and a 20-foot to 24-foot-wide access gate.
3.0 SITE DESCRIPTION

The alignment originates at the existing Cactus I Feeder transmission pipeline at the southeast corner of the Cactus Avenue and Heacock Street intersection. The alignment continues along Cactus Avenue, Indian Street, Alessandro Boulevard, Moreno Beach Drive, Perris Boulevard, and Nason Street. The alignment details are summarized in Table No. 1, Pipeline Alignment Details.

**Street Conditions**
Descriptions of street conditions along the alignment are described below.

**Cactus Avenue, Heacock Street to Indian Street**
- Two lanes each direction with a traversable median lane.
- No overhead utilities or other drilling obstructions.
- Light to medium traffic was observed mid-day.

**Indian Street, Cactus Avenue, to Alessandro Boulevard**
- Two lanes each direction with a traversable median lane and bicycle lanes.
- Overhead utilities are located adjacent to the southbound lane.
- Light to medium traffic was observed mid-day.

**Alessandro Boulevard, Indian Street to Perris Boulevard**
- Three lanes each direction with a non-traversable raised landscaped median.
- No overhead utilities and sparse trees.
- Medium traffic was observed mid-day.

**Alessandro Boulevard and Perris Boulevard Intersection**
- Highly trafficked intersection with 3 lanes each direction.

**Alessandro Boulevard, Perris Boulevard to Kitching Street**
- Three westbound lanes and two eastbound lanes with a wide shoulder and non-traversable raised landscaped median.
- No overhead utilities and sparse trees.
- Medium traffic was observed mid-day.

**Alessandro Boulevard and Kitching Street Intersection**
- Highly trafficked intersection with multiple lanes each direction.
- Kitching Street Flood Control Channel runs parallel and adjacent to the east side of Kitching street. The channel travels under Alessandro Boulevard at the intersection. Due to the channel, the pipeline will be constructed with bore-and-jack trenchless construction methods.
Alessandro Boulevard, Kitching Street to Lasselle Street
- One lane each direction with intermittent wide shoulders on eastbound and westbound lanes.
- No overhead drilling obstructions.
- Light traffic was observed mid-day.

Alessandro Boulevard, Lasselle Street to Moreno Beach Drive
- One lane each direction with a wide shoulder on the eastbound lane.
- Granitic bedrock is present along Alessandro Boulevard from approximately Lasselle Street to Darwin Drive. Distance from the street varies and is as close as 15 feet from the curb. The bedrock is discussed and pictured in the Geology section below.
- Bedrock may be present at shallow depths east of the outcrop along this section of the alignment.

Moreno Beach Drive, Alessandro Boulevard to Cactus Avenue
- One lane each direction with small shoulders on both sides of the street.
- Overhead utilities on southbound lane.

Perris Boulevard, Alessandro Boulevard to Approximately 350 feet North of Bay Avenue
- Two lanes each direction with non-traversable median and shoulders.
- Overhead utilities are located adjacent to the southbound lane.
- Light to medium traffic was observed mid-day.

4.0 SCOPE OF WORK

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

4.1 Project Set-up

The project set-up consisted of the following tasks.

- Conducted a site reconnaissance to mark the boring locations approved by Black & Veatch along the pipe alignment.
- Obtained encroachment permit from the City of Moreno Valley.
- 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 drilling, traffic control, and asphalt pavement repair and seismic refraction survey subcontractors.
4.2 Subsurface Exploration

Our subsurface exploration included soil borings and a seismic refraction survey, described in the following sections.

4.2.1 Soil Borings

Thirty-three exploratory borings (BH-01 through BH-33) were drilled between February 15 and February 22, 2017, at locations approved by Black & Veatch to investigate the subsurface conditions. The borings were drilled to depths ranging from approximately 15.5 to 51.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations. Additional 2 exploratory borings (BH-34 and BH-35) were drilled on June 21, 2018 to their maximum planned depths of 16.5 and 51.5 feet bgs.

The approximate locations of the borings are also shown on Figures No. 1 through 51 (prepared by Black and Veatch) and Figures No. 52 and 53 (prepared by Converse). The boring location figures are located at the back of the report. A detailed discussion of the subsurface exploration is presented in Appendix A, Field Exploration. Approximate locations and depths of the borings are presented in the following table.

Table No. 3, Boring Locations and Details

<table>
<thead>
<tr>
<th>Boring Number</th>
<th>Date Drilled</th>
<th>Location, Pipeline Station</th>
<th>Depth (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>2/21/17</td>
<td>Turnout Facility No. 1, 0+00</td>
<td>26.5</td>
</tr>
<tr>
<td>BH-02</td>
<td>2/15/17</td>
<td>Cactus Avenue, 2+50</td>
<td>51.5</td>
</tr>
<tr>
<td>BH-03</td>
<td>2/15/17</td>
<td>Cactus Avenue, 11+00</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-04</td>
<td>2/15/17</td>
<td>Cactus Avenue, 21+50</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-05</td>
<td>2/15/17</td>
<td>Indian Street, 31+00</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-06</td>
<td>2/16/17</td>
<td>Indian Street, 42+00</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-07</td>
<td>2/16/17</td>
<td>Indian Street, 50+00</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-08</td>
<td>2/16/17</td>
<td>Alessandro Boulevard, 60+00</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-09</td>
<td>2/20/17</td>
<td>Alessandro Boulevard, 68+00</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-10</td>
<td>2/16/17</td>
<td>Alessandro Boulevard west of Perris Boulevard, 77+00</td>
<td>26.5</td>
</tr>
<tr>
<td>BH-11</td>
<td>2/16/17</td>
<td>Alessandro Boulevard east of Perris Boulevard, 79+00</td>
<td>26.5</td>
</tr>
<tr>
<td>BH-12</td>
<td>2/22/17</td>
<td>Alessandro Boulevard, 91+50</td>
<td>16.5</td>
</tr>
<tr>
<td>BH-13</td>
<td>2/22/17</td>
<td>Alessandro Boulevard west of Kitching Channel (Bore and Jack Location), 103+50</td>
<td>25.5</td>
</tr>
</tbody>
</table>
4.2.2 Seismic Refraction Survey

Southwest Geophysics, Inc. was retained to conduct a seismic refraction survey consisting of seven seismic lines in areas of suspected hard bedrock. The purpose of the survey was to obtain a shear wave velocity profile of the subsurface materials and to assist in evaluation of the excavatability of the bedrock. The seismic refraction survey report is presented in Appendix C, *Seismic Refraction Survey*.
4.3 **Laboratory Testing**

Representative samples of soils along the alignments were tested in the laboratory to aid in the soils’ classification, and to evaluate their relevant engineering properties. These tests included the following.

- In situ moisture contents and dry densities (ASTM D2216 and ASTM 7263)
- Expansion index (ASTM D4829)
- Sand equivalent (ASTM D2419)
- R-value (California Test 301)
- Soil corrosivity tests (California Tests 643, 422, and 417)
- Collapse tests (ASTM D4546)
- Percent passing Sieve No. 200 (ASTM C117)
- Grain size analysis (ASTM C136)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)
- Consolidation (ASTM D2435)

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

4.4 **Analysis and Report Preparation**

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

5.0 **ALIGNMENT CONDITIONS**

The subsurface conditions along the alignments are discussed in the subsections below.

5.1 **Existing Pavement Section**

The thickness of the existing asphalt concrete and aggregate base, where encountered, are summarized in the following table.
Table No. 4, Existing Pavement Thicknesses

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Asphalt Concrete Thickness (inches)</th>
<th>Aggregate Base Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-02</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>BH-03</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>BH-04</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>BH-05</td>
<td>8.0</td>
<td>4.0</td>
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<td>BH-06</td>
<td>8.0</td>
<td>4.0</td>
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<td>BH-07</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>BH-08</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-09</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-10</td>
<td>8.0</td>
<td>12.0</td>
</tr>
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<td>BH-14</td>
<td>6.0</td>
<td>3.0</td>
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<tr>
<td>BH-15</td>
<td>N/A</td>
<td>N/A</td>
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<td>BH-16</td>
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<td>N/A</td>
</tr>
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<td>BH-17</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>BH-18</td>
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<td>N/A</td>
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<tr>
<td>BH-19</td>
<td>6.0</td>
<td>2.0</td>
</tr>
<tr>
<td>BH-20</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-21</td>
<td>N/A</td>
<td>N/A</td>
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<td>BH-22</td>
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<td>N/A</td>
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<td>BH-23</td>
<td>N/A</td>
<td>N/A</td>
</tr>
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<td>BH-24</td>
<td>N/A</td>
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<td>BH-25</td>
<td>6.0</td>
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<td>BH-26</td>
<td>N/A</td>
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<td>BH-27</td>
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<td>N/A</td>
</tr>
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<td>BH-29</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-30</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
## 5.2 Subsurface Profile

Based on the exploratory borings, the subsurface materials primarily consisted of alluvial sediments consisting of primarily sand and silt mixtures with scattered gravel. Weathered granitic bedrock was encountered in borings BH-14 through BH-18 (Alessandro Street between Kitching Street and Darwin Road) at depths ranging from 2.5 to 25 feet bgs. The weathered granitic bedrock excavates as silty sand and sand. Borings BH-14 through BH-18 were able to penetrate the weathered bedrock to the planned depths. Based on hammer blow counts, the upper 15 feet soils are generally loose to dense.


## 5.3 Groundwater

Groundwater was encountered during the investigation in boring BH-02 and BH-35 at depths of 23 and 37 feet bgs, respectively. Groundwater was not encountered in any other borings during the investigation.

Regional groundwater data from the GeoTracker database (SWRCB, 2018) for locations within close proximity to the alignment was reviewed to evaluate the current and historical groundwater levels.

- Circle K #1775 (ID No. T0606502742) located at the intersection of Heacock Street and John F Kennedy Drive reported groundwater depths ranging from approximately 14 to 20 feet bgs between 2009 and 2013.
- Mobil #18-A3E (ID No. T0606599291), located at the intersection of Alessandro Boulevard and Indian Street reported groundwater at depths ranging from approximately 35 to 49 feet bgs between 2001 and 2011.
- Arco #5208 (ID No. T0606562779) located at the intersection of Alessandro Boulevard and Perris Boulevard reported groundwater at depths ranging from approximately 27 to 43 feet bgs between 2005 and 2008.
- Tosco / 76 Station #6962 (ID No. T0606500535) located at the intersection of Alessandro Boulevard and Perris Boulevard reported groundwater at depths ranging from approximately 25 to 35 feet bgs between 2005 and 2010.
- Circle K #0872 (ID No. T0606547819) located at the intersection of Perris Boulevard and Dracaea Avenue reported groundwater depths ranging from approximately 43 to 52 feet bgs between 2006 and 2013.

No further groundwater data within one mile of the alignment east of Perris Boulevard was available from the GeoTracker database. The National Water Information System (USGS, 2018a) was also reviewed for available groundwater data. No wells were located within close proximity to the pipeline alignment.

Two groundwater monitoring wells were installed in September 2018, one west of Kitching Street (MW-1) and one in the Heacock/Pettit Booster Pump Station site (MW-2). Based on five months of monitoring, the groundwater level is at an approximate elevation of 20.5 feet and 19.8 feet bgs for MW-1 and MW-2, respectively. The Monitoring Well Installation Report (Converse, 2019) is included in Appendix F, Monitoring Well Installation Report.

Based on the data reviewed, the historical high groundwater level in the vicinity of the western portion of the alignment (generally west of Kitching Street) is estimated to be approximately 14 feet bgs, and the current depth is expected to be as shallow as 19.8 feet bgs. The bore-and-jack pits are located in the area of shallow historical groundwater and extend below the estimated historical high groundwater level. Groundwater may be encountered in the jacking and receiving pits. The historical high groundwater depth for the eastern half of the alignment is not known with certainty but is estimated to be deeper than 25 feet bgs. Dewatering is not expected to be required during the construction of the pipeline.

It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

### 5.4 Excavatability

Construction of the proposed pipeline will require excavation of bore-and-jack pits to depths of approximately 23.5 feet bgs and pipe trench to depths of up to approximately 19.5 feet bgs. Bedrock is not anticipated in the excavations up to 20 feet bgs on Cactus Avenue, Indian Street, Alessandro Boulevard west of Kitching Street, Alessandro east of Darwin Drive, Nason Street, and Moreno Beach Drive. We anticipate that the
The pipeline excavations along Alessandro Boulevard between Kitching Street and Darwin Drive will encounter granitic bedrock of varying degrees of weathering at depths as shallow as 2.5 feet. Where encountered in the hollow-stem auger borings (Borings BH-14 to BH-18) the bedrock was penetrated by the drilling auger to the planned depth, indicating the bedrock is generally excavatable in those immediate areas. However, the excavatability of the bedrock is expected to vary significantly based on the findings of the seismic refraction survey discussed below.

Southwest Geophysics, Inc. was retained to perform a seismic refraction survey for the purpose of evaluating bedrock rippability. Seven seismic refraction lines were conducted at selected locations along Alessandro Boulevard between Kitching Street and Darwin Drive. Their complete report, including seismic refraction profiles and maps of the seismic traverse locations, is presented in Appendix C, Seismic Refraction Survey.

Based on the seismic refraction and soil boring data, the depth at which difficult excavation will be encountered is expected to vary significantly along Alessandro Boulevard between Kitching Street and Darwin Drive, sometimes over short distances. In general, the bedrock is expected to be excavatable to depths of approximately 10 feet with easy to moderate difficulty between Kitching Street and 200 feet west of Lasselle Street. Excavations below 10 feet are expected to be excavatable with difficulty ranging from moderate to extremely difficult (where blasting is generally required). The difficulty of excavation will increase with depth as the degree of rock weathering decreases.

The bedrock along Alessandro Boulevard between 200 feet west of Lasselle Street and Darwin Drive is expected to be excavatable to 10 feet with difficulty ranging from moderate to extremely difficult (where blasting is generally required). The difficulty of excavation will increase with depth as the degree of rock weathering decreases. The use of specialized equipment or techniques, such as hydraulic hammers (“breakers”), jackhammers, blasting, or non-explosive rock reduction methods should be anticipated. Appropriate excavation equipment should be selected by an experienced earthwork contractor. Determination of the appropriate equipment may require test excavations in representative areas.

Based on our review of the seismic refraction data, and our experience with excavations in the site vicinity, we anticipate that corestones, or boulders of relatively unweathered rock, may be encountered embedded within the weathered bedrock, particularly in deeper excavations. Large or nested corestones may reduce excavation rates. Rock reduction techniques may be required to excavate and move very large corestones.
5.5  **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.

6.0  **GEOLOGIC SETTING**

The regional and local geology are discussed in the following subsections.

6.1  **Regional Geology**

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

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

6.2  **Local Geology**

Regional mapping (Morton and Miller, 2006) indicates that the subsurface along the alignment is mainly comprised of a combination of young and very old alluvial deposits with granitic bedrock outcropping east of Lasselle Street. The younger (Holocene to late Pleistocene-aged) alluvial-fan deposits consist of unconsolidated to moderately consolidated silt, sand, cobbles, and boulders. The older (middle to early Pleistocene-
aged) alluvial-fan deposits consist of moderately to well consolidated silt, sand, gravel, and conglomerate.

Granitic bedrock is exposed at the ground surface along the alignment to the east of Lasselle Street. The bedrock is Cretaceous in age and consists of undifferentiated tonalite. Tonalite is generally a coarse-grained granitic rock with a relatively high percentage of felsic minerals such as quartz, muscovite, and plagioclase feldspars. Bedrock was also encountered in boring BH-14 west of Lasselle at a depth of approximately 25 feet bgs.

The proposed alignment and turnout structures have been plotted on a portion of a geologic map in Figure No. 3, Alignment and Turnout Facilities Over Geologic Map following this page.

7.0 FAULTING AND SEISMICITY

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

7.1 Faulting

The site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 1974; Riverside County, 2018). The nearest active fault is the San Jacinto fault, located approximately 2.5 miles northeast of the end of the alignment.

7.2 CBC Seismic Design Parameters

Seismic parameters based on the California Building Code (CBSC, 2016) were determined at each turnout facility using the Seismic Design Maps application (USGS, 2018b) and are provided in the following table.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Facility/Pipeline Station</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Turnout Facility No. 1 / 0+00</td>
</tr>
<tr>
<td>Coordinates</td>
<td>33.9100°N, 117.2434°W</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
</tbody>
</table>
Explanation

<table>
<thead>
<tr>
<th>Geologic Unit</th>
<th>Liquefaction Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>Qyfa</td>
<td>Moderate</td>
</tr>
<tr>
<td>Qyva</td>
<td>Moderate</td>
</tr>
<tr>
<td>Qvofa</td>
<td>Low</td>
</tr>
<tr>
<td>Kt</td>
<td>None</td>
</tr>
</tbody>
</table>

Existing Cactus I Feeder

 Approximately 29,700 linear feet of Pipeline

City of Moreno Valley, Riverside County, California

Alignment Over Geologic Map


Converse Consultants
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Facility/Pipeline Station</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_s$</td>
<td>1.500g 1.615g 1.759g 1.943g</td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
<td>0.615g 0.701g 0.771g 0.865g</td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(1)), $F_a$</td>
<td>1.0 1.0 1.0 1.0</td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(2)), $F_v$</td>
<td>1.5 1.5 1.5 1.5</td>
</tr>
<tr>
<td>MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$</td>
<td>1.500g 1.615g 1.759g 1.943g</td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{M1}$</td>
<td>0.922g 1.052g 1.157g 1.298g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for short period $S_{DS}$</td>
<td>1.000g 1.077g 1.173g 1.295g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for 1-second period, $S_{D1}$</td>
<td>0.615g 0.676g 0.771g 0.865g</td>
</tr>
<tr>
<td>Maximum Peak Ground Acceleration, $PGA_M$</td>
<td>0.568g 0.636g 0.689g 0.756g</td>
</tr>
</tbody>
</table>

### 7.3 Secondary Effects of Seismic Activity

Generally, in addition to ground shaking, effects of seismic activity on a project site may include surface fault rupture, soil liquefaction, and settlement due to earthquake shaking, landslides, lateral spreading, tsunamis, seiches, 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:** The alignment is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 1974; Riverside County, 2018). The potential for surface rupture resulting from the movement of faults is not known with certainty but is considered 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.
Riverside County has designated portions of the alignment as having low or moderate susceptibility to liquefaction, generally corresponding to the older and younger alluvial-fan units, respectively. Turnout Facilities No. 1, 2, and 3 are located in areas designated as having low susceptibility to liquefaction while Turnout Facility No. 4 is in an area designated as having moderate susceptibility to liquefaction.

Relatively loose unsaturated sediments above the groundwater elevation may densify and settle when subjected to ground shaking during earthquakes. The site is underlain by loose to medium dense sediments, which may be susceptible to settlement during seismic shaking.

Dynamic settlement analyses were performed based on soil data gathered from Borings BH-02, BH-24 and BH-35, as presented in Appendix D, Settlement Analysis.

Based on the analysis, the dynamic settlement at the turnout structures and along the alignment is up to approximately 3.3 inches, except for Alessandro Boulevard between Kitching Street and Darwin Drive, where the potential for dry seismic settlement is considered low due to the presence of shallow bedrock.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The slopes along the Kitching Street flood control channel is concrete-lined and are not considered at risk for landsliding. The remainder of the alignment is not adjacent to any slopes at risk for landsliding.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over deeper layers which have liquefied due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved.

The slopes along the Kitching Street flood control channel slopes are concrete-lined and are not considered at risk for lateral spreading. Due to the relatively flat topography of the land along the remainder of the alignment, the potential for lateral spreading is low.

Tsunamis: Tsunamis are large waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location of the alignment, tsunamis do not pose a hazard to this site.
Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. The alignment crosses a flood control channel that runs parallel to Kitching Street. There is a potential for waves to overtop the channel during a large seismic event if moderate levels of water are present in the channel during the event.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes, resulting in flooding. At the closest point, the alignment is located approximately 3.8 miles from the northern tip of the Perris Lake Dam. Based on the position of surrounding hills and relative elevations, water would be conveyed to the south-southwest away from the alignment in the event of an earthquake-induced breach of the dam. The risk of earthquake-induced flooding is considered low.

8.0 LABORATORY TEST RESULTS

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

8.1 Physical Testing

- In-situ Moisture and Dry Density – *In-situ* dry density and moisture content of the site soils were determined in accordance to ASTM D2216 and ASTM 7263. Dry densities of the upper 15 feet soils along the proposed pipeline alignment ranged from 93 to 132 pcf with moisture contents of 1 to 20 percent. Results are presented in the log of borings in Appendix A, Field Exploration.
- Expansion Index – Seven representative bulk samples of the upper 20 feet soils were tested to evaluate expansion potential of soils in accordance with the ASTM D4829 test method. The test results indicated EI 0 to 51, corresponding to very low to medium expansion potential.
- Sand Equivalent – Eight representative bulk soil samples were tested to evaluate sand equivalent (SE) in accordance with the ASTM D2419 test method. The measured SE of the soil samples tested ranged from 12 to 23.
- Collapse Potential – Six relatively undisturbed representative samples collected from the upper 10 feet soils were tested in accordance with the ASTM Standard D4546 test method. The collapse potential was measured under a vertical stress of 2.0 kips per square foot (ksf). The test results showed collapse potential between 0.7 to 2.9 percent, indicating slight to moderate collapse potential.
- R-Value – Six representative bulk samples were tested to evaluate the R-value appropriate for pavement design. The test results indicated R-values of 26 to 58.
- Percent Passing Sieve #200 – A percent passing the #200 sieve test was performed on four representative soil samples in accordance to ASTM Standard D1140. The results, including data from the sieve analyses, indicate that the soils tested contained 22.2 to 49.8 percent silt or clay.
Grain Size Analysis – Fifteen representative soil samples were tested to determine their relative grain size distributions in accordance with the ASTM Standard C136. Test results are graphically presented in Drawings No. B-1a through B-1c, Grain Size Distribution Results.

Maximum Dry Density and Optimum Moisture Content – Results of seven typical moisture-density relationship tested in accordance with ASTM D1557 are presented in Drawings No. B-2a and B-2b, Moisture-Density Relationship Results, in Appendix B, Laboratory Testing Program. The laboratory maximum dry densities ranged from 130.0 to 135.4 pounds per cubic foot (pcf) and the optimum moisture contents ranged from 6.7 to 9.8 percent.

Direct Shear – Ten direct shear tests were performed on relatively undisturbed representative soil samples at soaked moisture conditions in accordance with ASTM D3080. Results of the direct shear tests are presented in Drawings No. B-3 through B-12, Direct Shear Test Results in Appendix B, Laboratory Testing Program.

Consolidation Test – One consolidation test was performed on relatively undisturbed soil sample, in accordance to ASTM Standard D2435. The test result is shown on Drawing No. B-13, Consolidation Test Result, in Appendix B, Laboratory Testing Program.

8.2 Chemical Testing - Corrosivity Evaluation

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

- The pH measurements of the samples ranged from 7.89 to 8.62.
- The sulfate contents of the samples tested ranged from 0.001 to 0.008 percent by weight.
- The chloride concentrations of the samples tested ranged from 32 to 245 ppm.
- The minimum electrical resistivities when saturated ranged from 1,200 to 17,000 Ohm-cm.

9.0 EARTHWORK RECOMMENDATIONS

This section contains our general recommendations regarding earthwork and grading for the proposed pipelines and turnout facilities. 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. Recommendations for earthwork associated with the project are presented in the following subsections.

9.1 General

The proposed turnout facilities will include at-grade structures. The earthwork for the structures will include excavation of soil and fill placement.

Earthwork for the pipeline will include trench excavation, pipe subgrade preparation, pipeline bedding placement, and trench backfill following the placement of the pipe, as well as excavation of jacking and receiving pits. Earthwork for the turnout structures will also include excavation of soils for structural fill placement.

Prior to the start of construction, all underground existing 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 structures or utilities.

All debris, surface vegetation, deleterious material, existing fill, and surficial soils containing roots and perishable materials should be stripped and removed from the site. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

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

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 Remedial Grading

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

**Turnout Facilities:** Structural footprints at the turnout structures should be overexcavated to at least 2 feet below the bottom of the deepest footing. The depth of overexcavation should be uniform across the entire structures. The overexcavation
should extend laterally at least 5 feet beyond the at-grade footprints. The overexcavation should be deepened as needed to remove any existing fill, and any very soft or saturated soil.

**Pavement:** All areas to receive asphalt or concrete pavement should be overexcavated to a depth of 12 inches below subgrade. The overexcavation should extend to limits of the paving required by the City of Moreno Valley.

Variations in the depths and lateral extent of overexcavation, based on observations by the geotechnical consultant during grading should be anticipated. The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill or structures. If isolated pockets of very soft, loose, or pumping subgrade are encountered, the overexcavation should be locally deepened, as needed, to expose undisturbed, firm, and unyielding soils.

### 9.3 Engineered Fill

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

- No particles larger than 3 inches in largest dimension.
- Rocks larger than one inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 30 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).

Imported materials, if required, should meet the following criteria prior to being used as compacted fill.

- Predominantly granular
- No particles larger than 3 inches in largest dimension.
- Free of organic material, loam, trash, or other deleterious material.
- Expansion index of 30 or less.
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Any imported fills should be tested and approved by geotechnical representative prior to delivery to the site.
9.4 **Compacted Fill Placement**

All surfaces to receive additional fill should be scarified to a depth of 6 inches. The scarified soil should be moisture conditioned to within ± 3 percent of optimum moisture for granular soils or 0 to 2 percent above optimum for fine soils. The scarified soil should be recompacted to at least 90 percent of the laboratory maximum dry density prior to the placement of any fill.

Fill soils should be evenly spread in horizontal, 8-inch-maximum, loose lifts. The fill materials should be thoroughly mixed and moisture conditioned to within 3 percent of optimum moisture content for granular soils and up to 2 percent above optimum moisture content for fine-grained soils.

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. The upper 12 inches of soil below asphalt and concrete pavement, should be compacted to at least 95 percent of 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.

To reduce differential settlement, variations in the soil type, degree of compaction and thickness of the compacted fill placed underneath the foundations should be kept to a minimum.

The project geotechnical consultant should observe the placement of fill and conduct in-place field density tests to check for adequate moisture content and relative compaction as required by the project specifications. Where less than the required relative compaction is indicated, additional compactive efforts should be applied and the soil moisture-conditioned as necessary, until the required relative compaction is attained.

9.5 **Site Drainage**

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. The turnout facility slabs should have a gradient of at least 2 percent towards drainage facilities. A desirable drainage gradient is 1 percent for paved areas and 2 percent in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.
9.6 **Pipeline Recommendations**

Earthwork associated with construction of underground utilities will include pipeline sub-grade preparation, pipe bedding placement, and trench backfill. Recommendations for these activities are provided in the following sections.

9.6.1 **Pipeline Subgrade Preparation**

The final subgrade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles, larger than 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.

9.6.2 **Pipe Bedding**

Bedding is defined as the material supporting and surrounding the pipe to one foot above the pipe. Pipe bedding should follow the City of Moreno Valley Modified Standard Plans MVSI-132E-1 and MVSI-132F-1, *Trench Backfill and Roadway Repair* (attached in Appendix E). Besides, additional information for pipe bedding are provided below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate, or crushed rock may be used as pipe bedding material. The sand equivalent of the tested soils varied from 12 to 23. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

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

Bedding materials should be vibrated in-place to achieve compaction. Care should be taken to densify the bedding material below the springline of the pipe. Prior to placing the pipe bedding material, the pipe subgrade should be uniform and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding.
material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

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

\[
\begin{align*}
&\text{i. } \frac{D_{15}(F)}{D_{85}(B)} \leq 5 \\
&\text{ii. } \frac{D_{50}(F)}{D_{50}(B)} < 25 \\
&\text{iii. Bedding Materials must have less than 5 percent minus 75 } \mu \text{m (No. 200) sieve to avoid internal movement of fines.}
\end{align*}
\]

Where,

\[
\begin{align*}
F &= \text{Bedding Material} \\
B &= \text{Surrounding Native and/or Fill Soils} \\
D_{15}(F) &= \text{Particle size through which 15% of bedding material will pass} \\
D_{85}(B) &= \text{Particle size through which 85% of surrounding soil will pass} \\
D_{50}(F) &= \text{Particle size through which 50% of bedding material will pass} \\
D_{50}(B) &= \text{Particle size through which 50% of surrounding soil will pass}
\end{align*}
\]

9.6.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Trench backfill should follow the City of Moreno Valley Modified Standard Plans MVSI-132E-1 and MVSI-132F-1, *Trench Backfill and Roadway Repair* (attached in Appendix E). Besides, additional trench backfill recommendations are presented below.

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.
- Trench zone backfill should be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method. At least the upper one 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 one 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 sheepfoot, 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 Standard D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) test methods or equivalent.
- Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.
- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.
- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project’s geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

9.7 Backfill of Jacking and Receiving Pits

We anticipate that the depths of the jacking and receiving pits will be approximately 23.5 feet below the existing grade. The pits should be backfilled following construction of the pipe crossings.

If dewatering is required, pumping activities should continue until the pits have been backfilled completely.

The pit bottoms should be free of trash, debris or other unsatisfactory materials at the time of backfill placement. The bottoms of the excavations should be scarified to a minimum depth of 12 inches below subgrade, moisture conditioned to within 3 percent of optimum moisture content, and recompacted to at least 90 percent of the laboratory maximum dry density.
The backfill soils should be well-blended and moisture conditioned to within 3 percent of optimum moisture content. The backfill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 90 percent of the laboratory maximum dry density per ASTM Standard D1557. If the ground surface is to be paved, the backfill within 12 inches of the pavement subgrade should be compacted to at least 95 percent of the laboratory maximum dry density.

The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, existing facilities, utilities, or completed work.

9.8 Backfill Recommendations Behind Subterranean Wall

Compaction of backfill adjacent to structural walls can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the walls. The compaction of wall backfill should be conducted procedure described in Section 9.4 Compacted Fill Placement.

10.0 PIPELINE 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

Based on our field exploration, laboratory testing and analyses of subsurface conditions within the project area, the proposed pipeline may be founded on native materials prepared as described in this report.

Where pipelines 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 bore-and-jack crossing and thrust blocks are presented in the following table.

Table No. 6, Resistance to Lateral Loads

<table>
<thead>
<tr>
<th>Lateral Resistance Soil Parameters</th>
<th>Cactus Avenue, Indian Street</th>
<th>Alessandro Boulevard from Indian Street to Kitching Street</th>
<th>Perris Boulevard from Alessandro Blvd. to 350 ft North of Bay Ave.</th>
<th>Alessandro Boulevard from Kitching Street to Darwin Drive to Moreno Beach Dr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passive earth pressure (psf per foot of depth)</td>
<td>260</td>
<td>250</td>
<td>240</td>
<td>250</td>
</tr>
<tr>
<td>Maximum allowable bearing pressure against native soils (psf)</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
<td>2,500</td>
</tr>
<tr>
<td>Coefficient of friction between formed concrete and native soils, fs (degree)</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Coefficient of friction between CML/CMC(^1) pipe and native soils, fs (degree)</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
</tr>
</tbody>
</table>

\(^1\) Concrete Mortar Lined (CML) Pipe, Cement Mortar Coating (CMC)

The passive earth pressure provided in the above table is used for resistance against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure.

Passive earth pressure values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above passive 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 Soil Parameters for Pipe Design

Structural design requires proper evaluation of all possible loads acting on pipes and structures. The stresses and strains induced on buried pipes 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 modulus of subgrade reaction was provided based on the soil type and compaction level of subgrade according to United States Bureau of Reclamation (USBR). The recommended values of the various soil parameters for design are provided in the following table.

Table No. 7, Soil Parameters for Pipe Design

<table>
<thead>
<tr>
<th>Soil Parameters</th>
<th>Cactus Avenue, Indian Street</th>
<th>Alessandro Boulevard from Indian Street to Kitching Street</th>
<th>Perris Boulevard from Alessandro Blvd. to 350 ft North of Bay Ave.</th>
<th>Alessandro Boulevard from Kitching Street to Darwin Drive</th>
<th>Nason St., Moreno Beach Dr., Alessandro Blvd. from Darwin Dr. to Moreno Beach Dr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average compacted fill total unit weight, $\gamma$ (pcf)</td>
<td>132</td>
<td>131</td>
<td>131</td>
<td>128</td>
<td>134</td>
</tr>
<tr>
<td>Angle of internal friction of soils, $\phi$</td>
<td>30</td>
<td>29</td>
<td>30</td>
<td>30</td>
<td>29</td>
</tr>
<tr>
<td>Soil cohesion, $c$ (psf)</td>
<td>60</td>
<td>90</td>
<td>10</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Coefficient of friction between formed concrete and native soils, $f_s$ (degree)</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Coefficient of friction between CML/CMC(^1) pipe and native soils, $f_s$ (degree)</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Bearing pressure against native soils (psf)</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
<td>2,500</td>
<td>2,000</td>
</tr>
<tr>
<td>Coefficient of passive earth pressure, $K_p$</td>
<td>3.00</td>
<td>2.88</td>
<td>3.12</td>
<td>3.00</td>
<td>2.88</td>
</tr>
<tr>
<td>Coefficient of active earth pressure, $K_a$</td>
<td>0.33</td>
<td>0.35</td>
<td>0.32</td>
<td>0.33</td>
<td>0.35</td>
</tr>
<tr>
<td>Modulus of Soil Reaction $E'$ (psi)</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
</tr>
</tbody>
</table>

\(^1\) Concrete Mortar Lined (CML) Pipe, Cement Mortar Coating (CMC)
10.4 Bearing Pressure for Anchor and Thrust Blocks

An allowable net bearing pressure presented in Table No. 7, *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 Jacking Force

The pipe jacking force is function of soil conditions, over burden pressure, pipe weight, size, annular space between pipe and soil, lubricant of the pipe, and installation time. The jacking force is equal to penetration resistance plus frictional resistance. Proper assessment of jacking force is required to design and select jacking pipes and thrust block.

The frictional resistance against the pipe during jacking is a function of the overburden pressure on the pipe, the friction angle between the pipe and the soil, the adhesion between the pipe and the soil, the surface area of the pipe, and the weight of the pipe.

The penetration resistance varies along the jack-and-bore depending on soil type and shape and steering action of the boring head.

Presence of boulders in the path of jack-and-bore operation can bring a sudden increase in the jacking force. Therefore, installation of pressure relief valves at the drive pit and indicators on the control panel is desirable to ensure that the allowable jacking force is not exceeded.

Design parameters presented Table No. 8, *Jacking System Design Parameters*, may be used to design jacking force system.

**Table No. 8, Jacking System Design Parameters**

<table>
<thead>
<tr>
<th>Locations</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-10 and BH-11</td>
<td>Bearing Pressure</td>
<td>2,000 psf</td>
</tr>
<tr>
<td></td>
<td>At-rest Lateral Earth Pressure</td>
<td>60 psf</td>
</tr>
<tr>
<td></td>
<td>Passive Earth Pressure</td>
<td>230 psf</td>
</tr>
<tr>
<td></td>
<td>Soil Unit Weight</td>
<td>120 pcf</td>
</tr>
<tr>
<td></td>
<td>Friction, between soil and concrete</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Friction, between soil and steel</td>
<td>0.25</td>
</tr>
</tbody>
</table>
We recommend that the ultimate compressive strength of the pipe should be at least 2.5 times the design jacking loads of the pipe.

The pipe designer should determine an appropriate factor of safety to be incorporated into the design of thrust block. The bore-and-jack contractor is responsible for selection of jacking force system and the final design of thrust blocks.

The jacking operations should be controlled at all times to minimize loss of ground. Steel casing sections should be jacked forward concurrently with the boring operation to provide continuous ground support.

A welded steel pipe casing is required to be installed at the crossing location. The annulus should be injected with cellular concrete or grout to fill any possible voids created by the crossing operation.

Ground distress potential along a crossing alignment depends on a number of factors, including type of soils, type of face support, internal pressure maintained to support the face, length of unlined zone, if any, and the amount of gap between the shield and the surrounding soils.

The potential of any significant ground distress at the surface can be minimized by selecting the proper equipment and construction method. The zone of influence of properly performed pipe crossing should be limited to a distance of about 2D above the crown of the shield, where D is the diameter of the shield. When depth of crown cover is about 2D or more, maximum ground surface settlement, if any, can be expected to be less. Higher ground settlement may occur for less depth of cover and inadequately supported pits can induce significant ground movement or even collapse.

It is the contractor’s responsibility to document the existing pre-construction conditions of any facilities and monitor deformations during construction. We recommend that ground surface above crossing operations be continuously monitored during

<table>
<thead>
<tr>
<th>Locations</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-13, BH-14, BH-19 and BH-25</td>
<td>Bearing Pressure</td>
<td>2,000 psf</td>
</tr>
<tr>
<td></td>
<td>At-rest Lateral Earth Pressure</td>
<td>62 psf</td>
</tr>
<tr>
<td></td>
<td>Passive Earth Pressure</td>
<td>240 psf</td>
</tr>
<tr>
<td></td>
<td>Soil Unit Weight</td>
<td>120 pcf</td>
</tr>
<tr>
<td></td>
<td>Effective Unit Weight</td>
<td>58 pcf</td>
</tr>
<tr>
<td>BH-13, BH-14, BH-19 and BH-25</td>
<td>Friction, between soil and concrete</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Friction, between soil and steel</td>
<td>0.25</td>
</tr>
</tbody>
</table>
construction using a surface settlement monument to ascertain that any vertical and horizontal movements are within allowable limits. Corrective action will be required by the contractor if deformations exceed the allowable limits.

10.6 Soil Corrosivity

The results of chemical testing of representative samples of alignment soils were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, Laboratory Testing Program in Table No. B-3, Summary of Corrosivity Test Results, and are discussed 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 vaults and equipment pads will be exposed to moisture from precipitation and irrigation. Based on the site locations and the results of chloride testing of the site soils, we do not anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

The measured value of the minimum electrical resistivity of the sample when saturated ranged from 1,200 to 17,000 Ohm-cm along the majority of the proposed alignment. This indicates that the soils tested of are corrosive to mildly corrosive to ferrous metals in contact with the soil (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures, for any ferrous metals in contact with the site and site soils.

11.0 TURNOUT FACILITY DESIGN RECOMMENDATIONS

Recommendations for the design and construction of the proposed turnout facilities are presented in the following sections. The recommendations provided are based on the assumption that, in preparing the site, the above earthwork recommendations will be implemented.
11.1 Shallow Foundation Design Parameters

The proposed at-grade structures at the turnout facilities may be supported by continuous/or isolated spread shallow footings supported by at least 24 inches of fill compacted to 90 percent of the laboratory maximum dry density as described in Section 9.4, Compacted Fill Placement. The footings should be at least 14 inches in width and embedded to at least 18 inches below the lowest adjacent grade. The footing dimensions and reinforcement should be based on structural design. Continuous and isolated footings can be designed based on the allowable net bearing capacity summarized in the following table.

<table>
<thead>
<tr>
<th>Facility</th>
<th>Allowable Net Bearing Capacity (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turnout Facility No. 1</td>
<td>2,500</td>
</tr>
<tr>
<td>Turnout Facility No. 2</td>
<td>2,000</td>
</tr>
<tr>
<td>Turnout Facility No. 3</td>
<td>2,000</td>
</tr>
<tr>
<td>Turnout Facility No. 4</td>
<td>2,000</td>
</tr>
</tbody>
</table>

The net bearing capacity is the ultimate bearing pressure minus the pressure from the weight of the soil between the contact surface and original ground (assuming the difference between unit weight of concrete and soil is negligible). 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.

11.2 Mat Foundation

Turnout facilities may be founded on mat foundations. The modulus of subgrade reaction (k) for design of flexible or rigid mat foundations was estimated from the available soil compressibility data and published charts. For design of flexible or rigid mat foundations, the following equation may be used.

\[ k = k_1 \left( \frac{B+1}{2B} \right)^2 \]

Where:
- \( k \) = vertical modulus of subgrade reaction for mat foundation, kips per cubic feet
- \( k_1 \) = 200 kcf, normalized modulus of subgrade reaction for 1 square foot footing
- \( B \) = foundation width, feet
E = 2.0 ksi, Young’s Modulus
ν = 0.30, Poisson’s Ratio

Rigid and flexible mat foundations may be designed based on an allowable net bearing capacity presented in Table No. 9, *Allowable Net Bearing Capacity*. The mat foundations should be embedded to at least 14 inches below the lowest adjacent grade. The mat foundations dimensions and reinforcement should be based on structural design.

### 11.3 Lateral Earth Pressures and Resistance to Lateral Loads

Lateral earth pressures and resistance to lateral loads were estimated by using on-site native soils strength parameters obtained from laboratory testing.

The active earth pressure behind any buried wall depends primarily on the allowable movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures. The field and laboratory data was evaluated to determine lateral earth pressures for each of the bore-and-jack pits. Due to the potential for variations in the subsurface conditions, we recommend that each bore-and-jack crossing be designed based on the most conservative lateral earth pressures from the borings associated with that crossing. The lateral earth pressures recommended for use in design of the bore-and-jack crossings are presented in the following table.

<table>
<thead>
<tr>
<th>Location / Station</th>
<th>Lateral Earth Pressure¹ (pcf)</th>
<th>Maximum Allowable Passive Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Active</td>
<td>At-Rest</td>
</tr>
<tr>
<td>Turnout Facility No. 1/0+00</td>
<td>40</td>
<td>61</td>
</tr>
<tr>
<td>Perris Boulevard Potential Bore and Jack Crossing/77+00</td>
<td>47</td>
<td>69</td>
</tr>
<tr>
<td>Kitching Channel Bore and Jack Crossing/104+10</td>
<td>40</td>
<td>60</td>
</tr>
<tr>
<td>Turnout Facility No. 2</td>
<td>43</td>
<td>65</td>
</tr>
<tr>
<td>Turnout Facility No. 3/183+00</td>
<td>45</td>
<td>68</td>
</tr>
<tr>
<td>Turnout Facility No. 4/233+00</td>
<td>45</td>
<td>68</td>
</tr>
</tbody>
</table>

¹ Assumes level ground conditions

Resistance to lateral loads can be assumed to be provided by friction acting at the base of foundations and by passive earth pressure. The passive earth pressures provided in the table above be used for resistance against recompacted native soils. A coefficient of friction of 0.30 between formed concrete and soil may be used with the dead load forces. A factor of safety of 1.5 was applied in calculating passive earth pressure.
Passive earth resistance values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above passive 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.

11.4 Slabs-on-Grade

Slabs-on-grade should be supported on properly compacted fill. Compacted fill used to support slabs-on-grade should be placed and compacted in accordance with Section 9.4 Compacted Fill Placement.

Structural design elements of slabs-on-grade, including but not limited to thickness, reinforcement, joint spacing, should be selected based on the analysis performed by the project structural engineer considering anticipated loading conditions and the modulus of subgrade reaction of the supporting materials.

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

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

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

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

11.5 Soil Expansion

Foundations, slabs, and other structures at the turnout facilities can be designed for very low expansive soil conditions (Expansion Index ≤ 20). We recommend that
additional expansion index tests be performed at the completion of sub-grade preparation, to verify the as-constructed expansion potential.

### 11.6 Settlement

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 40 feet.

Our analysis of the potential dynamic settlement is presented in Appendix D, *Settlement Analysis*. We estimate that the turnout facilities have the potential for up to 3.3 inches of dynamic settlement during a large earthquake. Differential settlement can be estimated as half of the total settlement over a distance of 40 horizontal feet.

The static and dynamic settlement estimates should not be combined for design purposes. The maximum combined static and dynamic settlement is not anticipated to exceed the maximum anticipated dynamic settlement.

### 11.7 Asphalt Concrete Pavement

Six representative soil samples were tested to determine the R-value of the subgrade soils. The tested R-values ranged from 26 to 58. For pavement design, we have utilized an R-value of 26 and design Traffic Indices (TIs) ranging from 5 to 10.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2017), 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.

<table>
<thead>
<tr>
<th>R-value</th>
<th>Traffic Index (TI)</th>
<th>Pavement Section</th>
<th>Pavement Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Option 1</td>
<td>Option 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Asphalt Concrete (inches)</td>
<td>Aggregate Base (inches)</td>
</tr>
<tr>
<td>26</td>
<td>5</td>
<td>4.0</td>
<td>6.0</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>4.0</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>4.5</td>
<td>10.0</td>
</tr>
</tbody>
</table>
R-value | Traffic Index (TI) | Pavement Section | Option 1 | Option 2 |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Asphalt Concrete (Inches)</td>
<td>Aggregate Base (inches)</td>
<td>Full AC Section (Inches)</td>
</tr>
<tr>
<td>26</td>
<td>8</td>
<td>5.0</td>
<td>12.0</td>
<td>11.0</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>6.0</td>
<td>14.0</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>6.5</td>
<td>12.0</td>
<td>14.5</td>
</tr>
</tbody>
</table>

Pavement sections should follow the Standard of the City of Moreno Valley or Table No. 11, *Recommended Preliminary Pavement Sections*. 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 Section 200-2.2, "Crushed Aggregate Base," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2015) or the Standard of the City of Moreno Valley and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC or the Standard of the City of Moreno Valley and should be placed in accordance with Section 302.5 of the SSPWC.

### 12.0 CONSTRUCTION RECOMMENDATIONS

Construction recommendations for the pipeline and turnout facilities are presented below.

#### 12.1 General

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

Vertical braced excavations are feasible along the pipeline alignment and bore-and-jack pits. Sloped excavations may not be feasible in locations adjacent to existing utilities or structures, including bore-and-jack pits adjacent to existing pavement, utilities,
channels, or other improvements. Recommendations pertaining to temporary excavations are presented in this section.

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, current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the owner's representative. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

### 12.2 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. The final determination of temporary slope gradients should be based on review of the encountered soils by a competent person employed by the contractor, in accordance with Section 1541 of the OSHA Construction Safety Orders.

#### Table No. 12, Slope Ratios for Temporary Excavations

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth of Cut (feet)</th>
<th>Recommended Maximum Slope (Horizontal:Vertical)¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy Silt, Clayey Sand</td>
<td>0-4</td>
<td>Vertical</td>
</tr>
<tr>
<td></td>
<td>4-20</td>
<td>1:1</td>
</tr>
<tr>
<td>Silty Sand, Sand</td>
<td>0-4</td>
<td>1:1</td>
</tr>
<tr>
<td></td>
<td>4-20</td>
<td>1.5:1</td>
</tr>
</tbody>
</table>

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

For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trench shields should be provided by the contractor as necessary to protect the workers in the excavation.

Surfaces exposed in sloped 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.
12.3 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. 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. 13, Lateral Earth Pressures for Temporary Shoring

<table>
<thead>
<tr>
<th>Lateral Resistance Soil Parameters*</th>
<th>Cactus Avenue, Indian Street</th>
<th>Alessandro Boulevard from Indian Street to Kitching Street</th>
<th>Perris Boulevard from Alessandro Blvd. to 350 ft North of Bay Ave.</th>
<th>Alessandro Boulevard from Kitching Street to Darwin Drive</th>
<th>Nason St., Moreno Beach Dr., Alessandro Blvd. from Darwin Dr. to Moreno Beach Dr.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Earth Pressure (Braced Shoring) (psf) (A)</td>
<td>28</td>
<td>30</td>
<td>24</td>
<td>28</td>
<td>31</td>
</tr>
<tr>
<td>Active Earth Pressure (Cantilever Shoring) (psf) (B)</td>
<td>44</td>
<td>46</td>
<td>40</td>
<td>43</td>
<td>47</td>
</tr>
<tr>
<td>At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)</td>
<td>67</td>
<td>68</td>
<td>60</td>
<td>64</td>
<td>70</td>
</tr>
<tr>
<td>Passive earth pressure (psf per foot of depth) (D)</td>
<td>260</td>
<td>250</td>
<td>240</td>
<td>250</td>
<td>250</td>
</tr>
<tr>
<td>Max. allowable bearing pressure against soil (psf)(E)</td>
<td>2,000</td>
<td>2,000</td>
<td>2,000</td>
<td>2,500</td>
<td>2,000</td>
</tr>
<tr>
<td>Coefficient of friction between sheet pile and native soils, fs (degree) (F)</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
</tbody>
</table>

* Parameters A through F are used in Figures No. 4 and 5.

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

Unrestrained (cantilever) design of cantilever shoring consisting of soldier piles spaced at least two diameters on-center or sheet piles, can be based on Figure No. 5, Lateral Earth Pressures on Temporary Cantilever Wall.

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

Earth 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 one 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 of time and in alternating sections.

12.4 Ground Classification for Trenchless Pipe Crossings

The Tunnelman’s Ground Classification (USDOT, 2009) categorizes predictive soil behaviors for saturated and unsaturated conditions as presented in the following table:

<table>
<thead>
<tr>
<th>Ground Classification</th>
<th>Ground Behavior</th>
<th>Typical Soil Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard</td>
<td>Tunnel heading may be advanced without roof support.</td>
<td>Cemented sand and gravel and over-consolidated clay above the ground water table.</td>
</tr>
<tr>
<td>Firm</td>
<td>Heading can be advanced without initial support, and final lining can be</td>
<td>L cess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.</td>
</tr>
<tr>
<td></td>
<td>constructed before ground starts to move.</td>
<td></td>
</tr>
<tr>
<td>Ground Classification</td>
<td>Ground Behavior</td>
<td>Typical Soil Types</td>
</tr>
<tr>
<td>-----------------------</td>
<td>------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Raveling</td>
<td>Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and &quot;brittle&quot; fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.</td>
<td>Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.</td>
</tr>
<tr>
<td>Squeezing</td>
<td>Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.</td>
<td>Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.</td>
</tr>
<tr>
<td>Swelling</td>
<td>Ground absorbs water, increases in volume, and expands slowly into the tunnel.</td>
<td>Highly preconsolidated clay with plastic index in excess of about 30, generally containing significant percentages of montmorillonite.</td>
</tr>
<tr>
<td>Running</td>
<td>Granular materials without cohesion are unstable at a slope greater than their angle of repose (approx 30° -35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.</td>
<td>Clean, dry angular materials.</td>
</tr>
<tr>
<td>Cohesive Running</td>
<td>Granular materials without cohesion are unstable at a slope greater than their angle of repose (approx 30° -35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.</td>
<td>Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs.</td>
</tr>
<tr>
<td>Flowing</td>
<td>A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.</td>
<td>Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.</td>
</tr>
</tbody>
</table>
It is our opinion that trenchless construction at the project site can be accomplished by an experienced contractor using jacking/micro-tunneling equipment. Provisions for controlling raveling, running, or flowing soils should be provided during the trenchless operation to minimize ground loss and ground subsidence.

Site-specific ground conditions and soil classifications pertaining to this project are presented in Table No. 15, *Site Specific Ground Classifications*.

Based on five months of monitoring of groundwater wells, groundwater may be encountered in the jacking and receiving pits. It is the responsibility of the contractor to design a dewatering system capable of lowering the groundwater to two feet below the bottom of the pits.

It is the contractor’s responsibility to design and select the appropriate tunnel construction method, support system and to follow the requirements of the health and safety rules of the State of California pertaining to tunnel construction and permit requirements of the City of Moreno Valley, Riverside County, and other local agencies, if applicable.

### 12.5 Trenchless Pipe Crossing Construction

Bore-and-jack is a trenchless construction method for pipe where open-cut technique is not feasible. This is a multi-stage process of construction which includes a temporary horizontal jacking platform and a starting alignment track in an entrance pit at a desired elevation. Manual control is used to jack the pipe at the starting point of the alignment with simultaneous excavation of the soil being accomplished by a rotating cutting head in the leading edge of the pipe’s annular space.

The selection of trenchless pipe crossing methods and equipment depends on pipe material, length of crossing, and anticipated ground conditions, and should be made by the contractor. Bore-and-jack pipe construction operations involve the initial construction of a jacking/tunneling pit and a receiving pit at each end of the pipe segment to be jacked. Site-specific ground conditions and soil classifications pertaining to this project are presented in the following table.
Table No. 15, Site Specific Ground Classifications

<table>
<thead>
<tr>
<th>Crossing Location</th>
<th>Boring No.</th>
<th>Approximate Pit Depth (Feet)</th>
<th>Soil Types</th>
<th>Hard</th>
<th>Firm</th>
<th>Raveling/Running</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perris Boulevard¹</td>
<td>BH-10</td>
<td>20</td>
<td>SM</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Perris Boulevard</td>
<td>BH-11</td>
<td>20</td>
<td>SM, SC</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kitching Channel¹³</td>
<td>BH-13</td>
<td>23.5</td>
<td>SM, ML</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kitching Channel¹²³</td>
<td>BH-14</td>
<td>23.5</td>
<td>SM, SC, Bedrock</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Utility crossings in Morrison Street</td>
<td>BH-19</td>
<td>15</td>
<td>SM, ML</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-inch crossing Alessandro Boulevard Between Pearl Lane and Oliver Street</td>
<td>BH-25⁴</td>
<td>20</td>
<td>SM, ML, SP-SM</td>
<td>X</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹ Bore and jack construction methods may be utilized at the Perris Boulevard crossing along Alessandro Boulevard
² Weathered bedrock encountered at 25 feet bgs in boring BH-14.
³ Pit depths extend below the observed groundwater level.
⁴ Boring located approximately 120 feet west of the crossings.

The working/access shafts are utilized to remove the spoil and to transport the construction materials and personnel for a tunnel project. The vertical face of the working shaft may be shored with sheet piles and/or soldier piles and lagging. The face of the shaft also can be supported by ribs and laggings. The design of sheet piling, soldier beam and lagging system may be designed according to the recommendations provided in Section 12.3, Shoring Design. Frequent contact grouting may be necessary to reinforce the support during construction.

The total load that can be developed in the jacking plate would depend on the depth and area of the plate. The jacking equipment should not impose a reaction of more than the allowable net bearing pressure summarized in Table No. 6, Resistance to Lateral Loads on the stabilized soils within the jacking pit. Pipes for use with the micro-tunneling systems must be designed to withstand the high axial jacking forces, and this is likely to be a far more significant design parameter than any post installation loading.

Grouting through the pipe casing after jacking is recommended to fill any possible voids created by the jacking operation. Jacking operations should be performed in accordance with the Standard Specifications for Public Works Construction, Sections 306-2 and 306-3 (Public Works Standards, 2015). Contractor should maintain standard grouting method so that no heave occurs.

Excavation procedures and shoring systems should be properly designed and implemented/installed to minimize the effect of settlement during construction. The contractor is responsible for minimizing impacts of crossing operations. Ground distress
potential along a crossing alignment depends on a number of factors, including type of soils, type of face support, internal pressure maintained to support the face, length of unlined zone, if any, and the amount of gap between the shield and the surrounding soils. The potential of any significant ground distress at the surface can be minimized by selecting the proper equipment and construction method. The zone of influence of properly performed pipe crossing should be limited to a distance of about 2D above the crown of the shield, where D is the diameter of the shield. When the depth of crown cover is about 2D or more, maximum ground surface settlement, if any, can be expected to be less than the thickness of the gap around the pipe. Higher ground settlement may occur for less depth of cover and inadequately supported pits can induce significant ground movement or even collapse.

It is the contractor's responsibility to document the existing pre-construction conditions of streets and any facilities and monitor deformations during construction. We recommend that the ground surface above crossing operations be continuously monitored during construction using a surface settlement monument to make sure any vertical and horizontal movements are within allowable limits. Corrective action will be required by the contractor if deformations exceed the allowable limits.

13.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Converse 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.

Converse should be present to observe conditions during construction. Testing should be performed to determine density and moisture of the during pipeline and turnout facility installation. 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.

14.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Black & Veatch, EMWD, and its authorized agents, to assist in the design and construction of the proposed pipeline project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

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

AMERICAN CONCRETE INSTITUTE (ACI), 2011, Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary, first printing August 2011.


BLAKE, T. F., 2000, EQFAULT, Computer Programs for Performing Probabilistic, and Seismic Coefficient Analysis and Historical Earthquake Search.

CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2013, California Building Code (CBC).

CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2016, California Building Code (CBC).


U.S. BUREAU OF RECLAMATION (USBR) MODULUS OF SOIL REACTION (E’), January 1977.


Appendix A

Field Exploration
APPENDIX A

FIELD EXPLORATION

Our field investigation included site reconnaissance and a subsurface exploration program consisting of drilling soil borings. Proposed boring locations were reviewed and approved by Black & Veatch. During the site reconnaissance, the surface conditions were noted and the boring locations were marked along the street using nearby landmarks as a guide. The boring locations should be considered accurate only to the degree implied by the method used to identify them in the field.

Thirty-three exploratory borings (BH-1 through BH-33) were drilled between February 15 and February 22, 2017 to investigate subsurface conditions. The borings were drilled to depths ranging from approximately 15.5 to 51.5 feet below the existing ground surface (bgs). Two additional exploratory borings (BH-34 and BH-35) were drilled on June 21, 2018 to their maximum planned depths of 16.5 and 51.5 feet bgs for the alignment change along Perris Boulevard and new location of turnout facility number 2.

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soil sampling. Encountered earth materials were continuously logged by a Converse geologist and visually classified in the field in accordance with the Unified Soil Classification System. Where appropriate, 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 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 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory.

Standard Penetration Tests (SPTs) were performed at selected depths 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 each 6 inches of sampler penetration are shown on the boring logs. The standard penetration tests were performed in accordance with the ASTM Standard D1586-84 test method. Bulk samples retrieved inside the SPT sampler were collected in zip-lock plastic bags for delivery to our laboratory.

In addition to drive samples, representative bulk samples were collected from selected depths within the borings. Bulk samples were obtained from drill cuttings and placed in large plastic bags for delivery to our laboratory.
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 driven samples are indicated in the log at the top of the next drive sample.

Following the completion of logging and sampling, borings were backfilled following the procedure summarized in the following table. If construction is delayed, the surface may settle over time. Therefore, we recommend the owner monitor the boring locations and backfill any depressions that might occur, or if possible, provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

Table No. A-1, Borehole Backfill Conditions

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Boring Backfill Conditions</th>
<th>Surface Patched</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-02</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-03</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-04</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-05</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-06</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-07</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-08</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-09</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-10</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-11</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-12</td>
<td>Per City of Moreno Valley Standard MVSI-132D-0</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-13</td>
<td>Per City of Moreno Valley Standard MVSI-132D-0</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-14</td>
<td>Per City of Moreno Valley Standard MVSI-132D-0</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-15</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-16</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-17</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-18</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-19</td>
<td>Per City of Moreno Valley Standard MVSI-132D-0</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-20</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-21</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-22</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-23</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>Boring No.</td>
<td>Boring Backfill Conditions</td>
<td>Surface Patched</td>
</tr>
<tr>
<td>-----------</td>
<td>--------------------------------------------</td>
<td>---------------------------</td>
</tr>
<tr>
<td>BH-24</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-25</td>
<td>Per City of Moreno Valley Standard MVSI-132D-0</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-26</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-27</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-28</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-29</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-30</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-31</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-32</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-33</td>
<td>Per City of Moreno Valley Standard MVSI-132D-0</td>
<td>N/A</td>
</tr>
<tr>
<td>BH-34</td>
<td>Soil cutting and compacted with auger</td>
<td>Cold Asphalt Concrete</td>
</tr>
<tr>
<td>BH-35</td>
<td>Soil cutting and compacted with auger</td>
<td>N/A</td>
</tr>
</tbody>
</table>

N/A – Boring was completed outside the existing pavement
City of Moreno Valley Standard MVSI-132D-0 consists of a slurry backfill and a grind and pave of the pavement surface.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. Logs of the exploratory borings are presented in Drawings No. A-2 through A-36, *Logs of Borings*. 
NOTES:
1. FOR TURNOUT FACILITY ENLARGED PLAN, SEE FIGURE NO. 5 IN APPENDIX D.
EASTERN MUNICIPAL WATER DISTRICT

CACTUS II FEEDER

PIPELINE ALIGNMENT

FIGURE 1"=40'

MATCHLINE SEE FIGURE 3

MATCHLINE SEE FIGURE 5

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- R/W RIGHT-OF-WAY
- EXIST TELEPHONE
- EXIST ELECTRIC

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.

CACTUS AVE

BH-4

15'

PROJECT
190120

EMWD
EASTERN MUNICIPAL WATER DISTRICT

PIPELINE ALIGNMENT
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST TELEPHONE
- EXIST GAS
- EXIST ELECTRIC

MATCHLINE SEE FIGURE 6
MATCHLINE SEE FIGURE 7

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.
LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST TELEPHONE
- EXIST GAS
- EXIST ELECTRIC

MATCHLINE SEE FIGURE 7
MATCHLINE SEE FIGURE 9

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.
MATCHLINE SEE FIGURE 8

MATCHLINE SEE FIGURE 10

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- R/W - RIGHT-OF-WAY
- EXIST SEWER
- EXIST TELEPHONE
- EXIST GAS
- EXIST ELECTRIC

Soil cuttings and cold patch. Borehole must be located within pipeline trench width.
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

FIGURE

MATCHLINE SEE FIGURE 11

LEGEND:

PROPOSED CML&C STEEL WATER PIPE
EXIST WATER
RIGHT-OF-WAY
EXIST SEWER
EXIST GAS
EXIST ELECTRIC

1"=40'

EASTERN MUNICIPAL WATER DISTRICT
CACTUS II FEEDER
PIPELINE ALIGNMENT

190120

EMWD

BLACK & BLATCH
BLACK & BLATCH

15°
Soil cuttings and cold patch. Borehole must be located within pipeline trench width.
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

FIGURE

1"=40'

MATCHLINE SEE FIGURE 15

MATCHLINE SEE FIGURE 17

LEgend:

- PROPOSED CM&G STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST GAS
- EXIST ELECTRIC
- EXIST STORM DRAIN

City restoration approach per City detail.
MATCHLINE SEE FIGURE 18

MATCHLINE SEE FIGURE 20

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST GAS
- EXIST ELECTRIC
- JACKING PIT

City restoration approach per City detail.
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST GAS
- EXIST ELECTRIC

MATCHLINE SEE FIGURE 19
MATCHLINE SEE FIGURE 21

FIGURE 1"=40'

ALESSANDRO BLVD

BH-15

2' GAS

12' SEWER

8' WATER

40' 20' 0 20' 40' 60' 80'

1"=40'

BLACK & VELVET
Geographical Surveying
PROJECT
190120

EASTERN MUNICIPAL WATER DISTRICT
CACTUS II FEEDER
PIPELINE ALIGNMENT

FIGURE
20
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST GAS
- EXIST ELECTRIC
- EXIST TELEPHONE

MILE MARKER 7.5

MATCHLINE SEE FIGURE 21

MATCHLINE SEE FIGURE 23

FIGURE 1"=40'

ALESSANDRO BLVD

BH-16

8" WATER

2" GAS

8" SEWER

PROJECT
190120

FIGURE 22

CACTUS II FEEDER
PIPELINE ALIGNMENT
LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST STORM DRAIN
- EXIST GAS
- EXIST TELEPHONE
- EXIST TIME WARNER

MATCHLINE SEE FIGURE 24
MATCHLINE SEE FIGURE 26

CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

FIGURE

PROJECT
190120

EMWD
CACTUS II FEEDER
PIPELINE ALIGNMENT

1"=40'
LEGEND:

- Proposed CML&C Steel Water Pipe
- Existing Water
- Right-of-Way
- Existing Electric
- Existing Telephone
- Existing Gas
- Existing Time Warner

MATCHLINE SEE FIGURE 28
MATCHLINE SEE FIGURE 30

EASTERN MUNICIPAL WATER DISTRICT

CACTUS II FEEDER
PIPELINE ALIGNMENT

FIGURE
Project

Blue Ribbon Ln

12" Water

16" Oil/Gas

BH-20

8" Water

ALLESSANDRO BLVD

1"=40'

0 20' 40' 60' 80'

40' 20' 0 40' 80'

1"=40'
LEGEND:

PROPOSED CML&C STEEL WATER PIPE
EXIST WATER
RIGHT-OF-WAY
EXIST SEWER
EXIST GAS
EXIST ELECTRIC
EXIST STORM DRAIN
EXIST TELEPHONE

NOTES:

1. FOR TURNOUT FACILITY ENLARGED PLAN, SEE FIGURE NO. 7 IN APPENDIX D.
FIGURE

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- R/W RIGHT-OF-WAY
- EXIST ELECTRIC
- EXIST TELEPHONE

MATCHLINE SEE FIGURE 32

MATCHLINE SEE FIGURE 34

EMWD
EASTERN MUNICIPAL WATER DISTRICT

CACTUS II FEEDER
PIPELINE ALIGNMENT

PROJECT
190120

FIGURE
33
CACTUS II FEEDER PIPELINE ALIGNMENT

EASTERN MUNICIPAL WATER DISTRICT

FIGURE

1"=40'

MATCHLINE SEE FIGURE 36

MATCHLINE SEE FIGURE 38

LEGEND:
- PROPOSED CM&L&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST ELECTRIC
- EXIST STORM DRAIN
- EXIST TELEPHONE

BH-25

ALESSANDRO BLVD

WATER METER ASSEMBLY

City restoration approach per City detail.

OLIVER ST

DRAINAGE CHANNEL

190120

CITY OF IMPERIAL VALLEY

BLACK & VeelOMIC

EMWD EASTERN MUNICIPAL WATER DISTRICT

PROJECT

37
LEGEND:

- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST ELECTRIC
- EXIST TELEPHONE
CACTUS II FEEDER
PIPELINE ALIGNMENT
EASTERN MUNICIPAL WATER DISTRICT

FIGURE

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST SEWER
- EXIST TELEPHONE
- EXIST GAS
- EXIST ELECTRIC

MORNO BEACH DR

BH-28

12" SEWER

12" WATER

16" OIL/GAS

MATCHLINE SEE FIGURE 41

MATCHLINE SEE FIGURE 43

1"=40'

BLACK & MCHATTAIN
PROJECT
190120

EASTERN MUNICIPAL WATER DISTRICT
CACTUS II FEEDER
PIPELINE ALIGNMENT
FIGURE 42
MATCHLINE SEE FIGURE 14

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST ELECTRIC
- EXIST SEWER
- EXIST GAS
- EXIST TELEPHONE

NOTES:
1. FOR TURNOUT FACILITY ENLARGED PLAN, SEE FIGURE NO. 6 IN APPENDIX D.
MATCHLINE SEE FIGURE 47

LEGEND:
- PROPOSED CML&C STEEL WATER PIPE
- EXIST WATER
- RIGHT-OF-WAY
- EXIST ELECTRIC
- EXIST SEWER
- EXIST GAS
- EXIST TELEPHONE

EASTERN MUNICIPAL WATER DISTRICT
CACTUS II FEEDER
PIPELINE ALIGNMENT

FIGURE

BH-31

NASON ST

12" WATER
4" GAS
18" SEWER

15'

1"=40'
Approximate Boring Locations Map
### SOIL CLASSIFICATION CHART

#### MAJOR_DIVISIONS

<table>
<thead>
<tr>
<th>SYMBOLS</th>
<th>TYPICAL DESCRIPTIONS</th>
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</thead>
<tbody>
<tr>
<td><strong>GW</strong></td>
<td>CLEAN GRAVELS (LITTLE OR NO FINES)</td>
</tr>
<tr>
<td><strong>GP</strong></td>
<td>POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>GM</strong></td>
<td>SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES</td>
</tr>
<tr>
<td><strong>GC</strong></td>
<td>CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES</td>
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<td><strong>SW</strong></td>
<td>WELL-GRADED SANDS, GRAVELY SANDS, LITTLE OR NO FINES</td>
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<td><strong>SP</strong></td>
<td>POORLY-GRADED SANDS, GRAVELY SAND, LITTLE OR NO FINES</td>
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<tr>
<td><strong>SM</strong></td>
<td>SILTY SANDS, SAND - SILT MIXTURES</td>
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<tr>
<td><strong>SC</strong></td>
<td>CLAYEY SANDS, SAND - CLAY MIXTURES</td>
</tr>
<tr>
<td><strong>ML</strong></td>
<td>INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOOR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY</td>
</tr>
<tr>
<td><strong>CL</strong></td>
<td>INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAY</td>
</tr>
<tr>
<td><strong>OL</strong></td>
<td>ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY</td>
</tr>
<tr>
<td><strong>MH</strong></td>
<td>INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS</td>
</tr>
<tr>
<td><strong>CH</strong></td>
<td>INORGANIC CLAYS OF HIGH PLASTICITY</td>
</tr>
<tr>
<td><strong>OH</strong></td>
<td>ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS</td>
</tr>
</tbody>
</table>

#### SAMPLE_TYPE

- STANDARD PENETRATION TEST
- DRIVE SAMPLE (2.42" I.D. sampler (CMS)).
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

### BORING LOG SYMBOLS

#### LABORATORY TESTING ABBREVIATIONS

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>STRENGTH</th>
<th>CLASSIFICATION</th>
<th>PLASTICITY</th>
<th>Grain Size Analysis</th>
<th>Failure No. 200 Sieve</th>
<th>Sand Equivalent</th>
<th>Expansion Index</th>
<th>Compaction Curve</th>
<th>Hydrometer</th>
<th>Disturb</th>
<th>Density (%)</th>
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<tr>
<td>Po</td>
<td>Pocket Penetrometer</td>
<td>p</td>
<td>Direct Shear</td>
<td>ds</td>
<td>Direct Shear (single point)</td>
<td>ds'</td>
<td>Unconfined Compression</td>
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<td>Triaxial Compression</td>
<td>tx</td>
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<td>Vane Shear</td>
<td>vs</td>
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<tr>
<td>Soil Cement</td>
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<td></td>
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</tr>
</tbody>
</table>

#### Notes:
- Dual symbols are used to indicate borderline soil classifications.

---

Converse Consultants
Cactus II Feeder
Approximately 29,700 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01
Drawing No. A-1

Project ID: 16-81-257-01.GPJ; Template: KEY
Summary of Subsurface Conditions

Alluvium:
Silty Sand (SM): fine to medium-grained, reddish-brown.

Sandy Silt (ML): fine to medium-grained sand, brown.
- reddish-brown

End of boring at 26.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/21/17.
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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td><strong>5&quot; ASPHALT CONCRETE / 4&quot; AGGREGATE BASE:</strong></td>
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<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to medium-grained, few gravel up to 1.5&quot; in largest dimension, brown.</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>5/17/50-5&quot; 13 125</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td><strong>Sandy Silt (ML):</strong> fine to coarse-grained sand, reddish-brown.</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>10/20/23 10 127</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>9/37/50-2&quot; 15 114</td>
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<td></td>
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<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to medium-grained, trace clay, brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6/5/4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Sandy Silt (ML):</strong> fine-grained sand, brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12/24/45 15 119</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5/9/14</td>
</tr>
</tbody>
</table>

Cactus II Feeder
Approximately 29,700 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01
Drawing No. A-3a
### Log of Boring No. BH-02

**Dates Drilled:** 2/15/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1548  
**Depth to Water (ft):** 23

#### 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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Sample Dates</th>
<th>Drive</th>
<th>Bulk</th>
<th>Moisture</th>
<th>Dry Unit Wt.</th>
<th>Other</th>
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<tr>
<td>40-45</td>
<td>Silty Sand (SM): fine to medium-grained, brown.</td>
<td>4/12/37</td>
<td>11</td>
<td>127</td>
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</tr>
<tr>
<td>45-50</td>
<td>- fine to coarse-grained</td>
<td>11/17/23</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 50-55      | End of boring at 51.5 feet bgs.  
Groundwater was encountered at 23 feet bgs.  
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/15/17. | 3/20/30 | | | | |

---

Cactus II Feeder  
Approximately 29,700 Linear Feet of Pipeline  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch
**Log of Boring No. BH-03**

Dates Drilled: 2/15/2017  
Logged by: William Buckley  
Checked By: Scot Mathis  
Equipment: 8" HOLLOW STEM AUGER  
Driving Weight and Drop: 140 lbs / 30 in  
Ground Surface Elevation (ft): 1551  
Depth to Water (ft): NOT ENCOUNTERED

**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.

### 5" ASPHALT CONCRETE / 5" AGGREGATE BASE:

- **ALLUVIUM:**
  - SILTY SAND (SM): fine to medium-grained, few gravel up to 1" in largest dimension, brown.
  - SAMPLES: 5/5/3, 6/12/16
  - MOISTURE: 10
  - DRY UNIT WT.: 116

- **SANDY SILT (ML):** fine to medium-grained sand, reddish-brown.
  - SAMPLES: 3/3/4, 4/8/11, 6/11/42
  - MOISTURE: 6, 12
  - DRY UNIT WT.: 120, 110, 119

End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/15/17.
### 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.

#### 5" Asphalt Concrete / 5" Aggregate Base:

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Alluvium: Silty Sand (SM): fine to medium-grained, few gravel up to 1&quot; in largest dimension, brown.</th>
<th>Samples</th>
<th>Drive</th>
<th>Blows</th>
<th>Moisture</th>
<th>Dry Unit Wt.</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>3/4/9</td>
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<td></td>
<td>8/13/20</td>
<td>2</td>
<td>dist.</td>
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<td>14/17/25</td>
<td>13</td>
<td>118</td>
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End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/15/17.
**Log of Boring No. BH-05**

Dates Drilled: 2/15/2017

Logged by: William Buckley

Checked By: Scot Mathis

Equipment: 8" HOLLOW STEM AUGER

Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 1564

Depth to Water (ft): NOT ENCOUNTERED

---

**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.

---

**8" ASPHALT CONCRETE / 4" AGGREGATE BASE:**

- **ALLUVIUM:**
  - SILTY SAND (SM): fine to medium-grained, brown.

- **SANDY SILT (ML):** fine-grained sand, brown.

- **SAND with SILT (SP-SM):** fine to coarse-grained, light brown.

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/15/17.
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.

8" ASPHALT CONCRETE / 4" AGGREGATE BASE:

ALLUVIUM:
SILTY SAND (SM): fine to medium-grained, trace gravel up to 1.5" in largest dimension, trace clay, brown.
- fine to coarse-grained, light brown to brown

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/16/17.

Graphic Log

Depth (ft) | SAMPLES
---|---
0 | DRILL
5 | 8/5/4
10 | 3/6/6
15 | 3/5/7

Interstitial Moisture:

| Depth (ft) | MOISTURE |
---|---|
7 | 123 |
7 | 109 |
3 | 108 |
8 | 115 |

BULK

Interstitial Moisture:

| Depth (ft) | MOISTURE |
---|---|
7 | ma,max |
3 | se,ca,er |
3 | 109 |
8 | 108 |
8 | 115 |

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/16/17.
Log of Boring No. BH-07

Dates Drilled: 2/16/2017  Logged by: William Buckley  Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1577  Depth to Water (ft): NOT ENCOUNTERED

---

### 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.

#### 5" ASPHALT CONCRETE / 4" AGGREGATE BASE:

**ALLUVIUM:**

**Silty Sand (SM):** fine-grained, trace gravel up to 1.5" in largest dimension, brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
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<tbody>
<tr>
<td>3/8/8</td>
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<td>3/2/4</td>
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<td>5</td>
<td></td>
<td></td>
<td>107</td>
</tr>
</tbody>
</table>

**Sandy Silt (ML):** fine-grained sand, brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
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<tbody>
<tr>
<td>3/5/12</td>
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<td></td>
<td></td>
<td></td>
<td>109</td>
</tr>
</tbody>
</table>

**Silty Sand (SM):** fine to medium-grained, gravel up to 1" in largest dimension, brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
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<td></td>
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</table>

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/16/17.
End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/16/17.

### Summary of Subsurface Conditions

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BOWLS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT</th>
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<td>3/6/6</td>
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<td>3/4/10</td>
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<td>111</td>
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<td>15</td>
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<td></td>
<td>5/9/14</td>
<td>11</td>
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</tr>
<tr>
<td>15-81-158-01 A-9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6/14/16</td>
<td>10</td>
<td>111</td>
<td></td>
</tr>
</tbody>
</table>

**ALLUVIUM:**

**Silty Sand (SM):** fine to medium-grained, brown.

- Fine-grained sand
### Log of Boring No. BH-09

**Dates Drilled:** 2/20/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 1576  
**Depth to Water (ft):** NOT ENCOUNTERED

### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th><strong>ALLUVIUM:</strong></th>
<th><strong>SILTY SAND (SM):</strong> fine to medium-grained, brown.</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>9/14/16 9 132</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td><strong>SANDY SILT (ML):</strong> fine-grained sand, brown.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>7/10/11 20 107</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- fine to medium-grained sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>10/19/25 8 120</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of boring at 16.5 feet bgs.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>No groundwater encountered.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled with soil cuttings on 2/20/17.</td>
<td></td>
</tr>
</tbody>
</table>

---

**Cactus II Feeder**  
Approximately 29,700 Linear Feet of Pipeline  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch

**Project No.** 15-81-158-01  
**Drawing No.** A-10
End of boring at 26.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with cold asphalt concrete on 2/16/17.
Log of Boring No. BH-11

Dates Drilled: 2/16/2017  Logged by: William Buckley  Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1568  Depth to Water (ft): NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Drive</td>
</tr>
<tr>
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<td>Bulk</td>
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<td>BLOWS</td>
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<td></td>
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<td></td>
<td>MOISTURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>DRY UNIT WT.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>OTHER</td>
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<td>0</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>8&quot; ASPHALT CONCRETE / 12&quot; AGGREGATE BASE:</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>ALLUVIUM:</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SILTY SAND (SM): fine to medium-grained, trace clay, brown.</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>CLAYEY SAND (SC): fine to medium-grained, brown.</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>SILTY SAND (SM): fine to coarse-grained, trace clay, reddish-brown.</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>SAND with SILT (SP-SM): fine to coarse-grained, light brown.</td>
<td></td>
</tr>
</tbody>
</table>

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 - 15</td>
<td></td>
<td><strong>5&quot; ASPHALT CONCRETE / 2&quot; AGGREGATE BASE:</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to medium-grained, brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Sandy Silt (ML):</strong> fine to medium-grained sand, brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- trace clay, reddish-brown</td>
</tr>
</tbody>
</table>

End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled and surface patched in accordance with City Standard MVSI-132-0 on 2/22/17.
### Summary of Subsurface Conditions

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#### 5" Asphalt Concrete / No Aggregate Base:

- **Alluvium:**
  - Silty Sand (SM): fine to medium-grained, gravel up to 1.5" in largest dimension, brown.
  - Sandy Silt (ML): fine to medium-grained sand, brown.
  - Silty Sand (SM): fine to medium-grained, reddish-brown.
  - Sandy Silt (ML): fine to medium-grained sand, reddish-brown.
  - Sand (SP): fine to coarse-grained, light brown.

#### Dates Drilled:

- 2/22/17

#### Driving Weight and Drop:

- 140 lbs / 30 in

#### Summary:

End of boring at 25.5 feet bgs. Perched groundwater was encountered at 25.5 feet bgs. Borehole backfilled and surface patched in accordance with City Standard MVSI-132-0 on 2/22/17.
**Log of Boring No. BH-14 (Bore & Jack)**

**Dates Drilled:** 2/22/2017

**Logged by:** William Buckley

**Checked By:** Scot Mathis

**Equipment:** 8" HOLLOW STEM AUGER

**Driving Weight and Drop:** 140 lbs / 30 in

**Ground Surface Elevation (ft):** 1564

**Depth to Water (ft):** NOT ENCOUNTERED

---

**SUMMARY OF SUBSURFACE CONDITIONS**

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**6" ASPHALT CONCRETE / 3" AGGREGATE BASE:**

**ALLUVIUM:**

**SILTY SAND (SM):** fine to coarse-grained, brown.

- light brown

**CLAYEY SAND (SC):** fine to coarse-grained, reddish-brown.

**GRANITIC BEDROCK:**

**Excavates as SAND (SP):** fine to coarse-grained, light brown.

End of boring at 26.0 feet bgs.
No groundwater encountered.
Borehole backfilled and surface patched in accordance with City Standard MVSI-132-0 on 2/22/17.

---

**Converse Consultants**

Approximately 29,700 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Black & Veatch

**Project No.** 15-81-158-01  
**Drawing No.** A-15
**Log of Boring No. BH-15**

**Dates Drilled:** 2/20/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1573  
**Depth to Water (ft):** NOT ENCOUNTERED

**SUMMARY OF SUBSURFACE CONDITIONS**

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Summary</th>
</tr>
</thead>
</table>
| 0-5       |             | **ALLUVIUM:**
|           |             | **SILTY SAND (SM):** fine to medium-grained, brown. |
|           |             | - light brown |
| 5-10      |             | - fine to coarse-grained sand, reddish-brown |
| 10-15     |             | **GRANITIC BEDROCK:**
|           |             | **Excavates as SAND (SP):** fine to coarse-grained, orangish-brown. |
| 15-15.5   |             | End of boring at 15.5 feet bgs.  
|           |             | No groundwater encountered.  
|           |             | Borehole backfilled with soil cuttings on 2/20/17. |

**SAMPLES**

<table>
<thead>
<tr>
<th>Depth</th>
<th>Blow Count</th>
<th>Moisture</th>
<th>Bulk Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/4/5</td>
<td>9</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>8/12/12</td>
<td>9</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>8/23/50-5&quot;</td>
<td>2</td>
<td>116</td>
<td></td>
</tr>
<tr>
<td>50-5&quot;</td>
<td>4 dist.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Additional Information**

- **Project No.:** 15-81-158-01  
- **Drawing No.:** A-16  
- **Cactus II Feeder:** Approximately 29,700 Linear Feet of Pipeline  
- **City of Moreno Valley, Riverside County, California:** For: Black & Veatch
Log of Boring No. BH-16

Dates Drilled: 2/20/2017  Logged by: William Buckley  Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1581  Depth to Water (ft): NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
</tr>
</tbody>
</table>

**SUMMARY OF SUBSURFACE CONDITIONS**

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**ALLUVIUM:**

**Silty Sand (SM):** fine to medium-grained, brown.

- fine to coarse-grained, light brown

**Granitic Bedrock:**

Excavates as Sand (SP): fine to coarse-grained, orangish-brown.

- olive-brown

<table>
<thead>
<tr>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRIVE</td>
</tr>
<tr>
<td>BULK</td>
</tr>
<tr>
<td>BLOWS</td>
</tr>
<tr>
<td>MOISTURE</td>
</tr>
<tr>
<td>DRY UNIT WT (pcf)</td>
</tr>
<tr>
<td>OTHER</td>
</tr>
</tbody>
</table>

- 50-4"  5  dist.
- 20/50-4"  8  120
- 50-4.5"  5  dist.

End of boring at 15.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/20/17.
### Log of Boring No. BH-17

**Dates Drilled:** 2/20/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 1595  
**Depth to Water (ft):** NOT ENCOUNTERED

---

#### SUMMARY OF SUBSURFACE CONDITIONS

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### Depth (ft) | Graphic Log | Summary | SAMPLES
---|---|---|---
5 | ALLUVIUM: SILTY SAND (SM): fine to coarse-grained, light brown to brown.
10 | GRANITIC BEDROCK: Excavates as SAND (SP): fine to coarse-grained, orangish-brown.
15 | - light brown to brown
- no recovery

End of boring at 15.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings on 2/20/17.

---

**SAMPLES**

<table>
<thead>
<tr>
<th>DRIVE</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/36/50-3&quot;</td>
<td>3</td>
<td>127</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50-4&quot;</td>
<td>9</td>
<td>dist.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50/50-2&quot;</td>
<td>4</td>
<td>116</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50-3&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
SUMMARY OF SUBSURFACE CONDITIONS

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**ALLUVIUM:**
Silty Sand (SM): fine to coarse-grained, brown.

**GRANITIC BEDROCK:**
Excavates as Silty Sand (SM): fine to coarse-grained, orangish-brown.
- light brown
- white

End of boring at 15.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/20/17.
Log of Boring No. BH-19

Dates Drilled: 2/22/2017
Logged by: William Buckley
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1582
Depth to Water (ft): NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>6&quot; ASPHALT CONCRETE / 2&quot; AGGREGATE BASE:</td>
</tr>
<tr>
<td>5</td>
<td>ALLUVIUM:</td>
</tr>
<tr>
<td></td>
<td>SILTY SAND (SM): fine to coarse-grained, few gravel up to 1.5&quot; in largest dimension, grayish-brown.</td>
</tr>
<tr>
<td>10</td>
<td>- reddish-brown</td>
</tr>
<tr>
<td>15</td>
<td>SANDY SILT (ML): fine to medium-grained, brown.</td>
</tr>
<tr>
<td></td>
<td>End of boring at 16.5 feet bgs.</td>
</tr>
<tr>
<td></td>
<td>No groundwater encountered.</td>
</tr>
<tr>
<td></td>
<td>Borehole backfilled and surface patched in accordance with City Standard MVSI-132-0 on 2/22/17.</td>
</tr>
</tbody>
</table>

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**Log of Boring No. BH-20**

Dates Drilled: 2/20/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1586  
Depth to Water (ft): NOT ENCONTRATED

---

### SUMMARY OF SUBSURFACE CONDITIONS

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#### ALLUVIUM:

- **SILTY SAND (SM):** fine to medium-grained, gravel up to 1" in largest dimension, brown.
  - light brown

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>
| 15        | End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings on 2/20/17. |

---

**SAMPLES**

<table>
<thead>
<tr>
<th></th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT</th>
<th>OTHER</th>
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</thead>
<tbody>
<tr>
<td>5</td>
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<td>10</td>
<td>5/8/8</td>
<td>15</td>
<td>114</td>
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<td>1</td>
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<tr>
<td>15</td>
<td>2/3/6</td>
<td>8</td>
<td>113</td>
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<tr>
<td>15</td>
<td>5/8/12</td>
<td>7</td>
<td>102</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/20/17.
**Log of Boring No. BH-22 (Turnout Facility No. 3)**

**Dates Drilled:** 2/21/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 1588  
**Depth to Water (ft):** NOT ENCOUNTERED

---

**SUMMARY OF SUBSURFACE CONDITIONS**

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
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<tbody>
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<td></td>
<td></td>
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<td></td>
<td></td>
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<td>8</td>
<td>113</td>
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<td></td>
<td>se, ca, er, col, ds</td>
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<td></td>
<td>3/12/23</td>
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<td>119</td>
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<tr>
<td>25</td>
<td></td>
<td></td>
<td>9/11/14</td>
<td>3</td>
<td>120</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**ALLUVIUM:**

**SILTY SAND (SM):** fine to medium-grained, few gravel up to 1.5" in largest dimension, dark brown.

**SANDY SILT (ML):** fine to medium-grained sand, brown.

**- fine to coarse-grained sand**

**SILTY SAND (SM):** fine to medium-grained, light brown.

---

**Log of Boring No. BH-23**

Dates Drilled: 2/21/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1582  
Depth to Water (ft): NOT ENCOUNTERED

---

### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Alluvium: Sandy Silt (ML): Fine to medium-grained sand, brown.</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>- Light brown</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>Alluvium: Sandy Silt (ML): Fine to medium-grained sand, brown.</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>- Light brown</td>
</tr>
<tr>
<td>15-61-71-01 A-24</td>
<td></td>
<td>-</td>
</tr>
</tbody>
</table>

End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings on 2/21/17.
### SUMMARY OF SUBSURFACE CONDITIONS

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#### ALLUVIUM:

- **Silty Sand (SM):** fine to medium-grained, brown.

- **Sandy Silt (ML):** fine to medium-grained sand, reddish-brown.

- **Silty Sand (SM):** fine to coarse-grained, reddish-brown.

- **-** fine to coarse-grained

- **-** brown

#### SAMPLES

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Drive</th>
<th>Blows</th>
<th>Moisture</th>
<th>Dry Unit WT.</th>
<th>Other</th>
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<td>3/9/18</td>
<td>7</td>
<td>123</td>
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**Log of Boring No. BH-24**

**Dates Drilled:** 2/21/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1582  
**Depth to Water (ft):** NOT ENCLOSED

---

**SUMMARY OF SUBSURFACE CONDITIONS**

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SANDY SILT (ML): fine to medium-grained sand, reddish-brown.</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
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<td></td>
</tr>
<tr>
<td>40</td>
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</tr>
<tr>
<td>55</td>
<td></td>
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</tr>
<tr>
<td>60</td>
<td></td>
<td>- light brown</td>
</tr>
</tbody>
</table>

End of boring at 51.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings on 2/21/17.
**Log of Boring No. BH-25**

**Dates Drilled:** 2/22/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 1573  
**Depth to Water (ft):** NOT ENCOUNTERED

---

### SUMMARY OF SUBSURFACE CONDITIONS

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**6" ASPHALT CONCRETE / 4" AGGREGATE BASE:**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td></td>
<td></td>
<td>26/19/18</td>
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<td>122</td>
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</tr>
<tr>
<td>10-15</td>
<td></td>
<td></td>
<td>3/6/7</td>
<td>13</td>
<td>113</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15-20</td>
<td></td>
<td></td>
<td>2/6/7</td>
<td>9</td>
<td>123</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20-25</td>
<td></td>
<td></td>
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<td>25-26.5</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5/13/17</td>
<td>8</td>
<td>123</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 26.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled and surface patched in accordance with City Standard MVSI-132-0 on 2/22/17.
### Log of Boring No. BH-26

**Dates Drilled:** 2/20/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1575  
**Depth to Water (ft):** NOT ENCOUNTERED

---

### 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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Alluvium: Silty Sand (SM)</th>
<th>Samples</th>
<th>Dry Unit Wt. (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td></td>
<td></td>
<td>3/3/5</td>
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<td></td>
<td></td>
<td>5/5/9</td>
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<td></td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td>106</td>
</tr>
</tbody>
</table>

**ALLUVIUM:**  
**Silty Sand (SM):** fine to medium-grained, olive brown.

**SAND (SP):** fine to medium-grained, gravel up to 1.5" in largest dimension, grayish-brown.

**Sandy Silt (ML):** fine to medium-grained sand, brown.

End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings on 2/20/17.
Log of Boring No. BH-27 (Turnout Facility No. 4)

Dates Drilled: 2/20/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1588  
Depth to Water (ft): NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>SILTY SAND (SM):</strong> fine to medium-grained, olive brown to brown.</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>- reddish-brown</strong></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>- trace gravel up to 1&quot; in largest dimension</strong></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
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<td><strong>- light brown</strong></td>
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<tr>
<td>20</td>
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<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>End of boring at 26.0 feet bgs.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No groundwater encountered.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled with soil cuttings on 2/20/17.</td>
</tr>
</tbody>
</table>

Sample Dates:
- 2/4/5 ei, col, se, ca, er
- 2/4/6 11 103
- 10/18/35 7 120
- 2/26/45 3 117
- 21/50-5" 5 107

Converse Consultants
Approximately 29,700 Linear Feet of Pipeline
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01  
Drawing No. A-28
## Summary of Subsurface Conditions

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### Alluvium:
- **Silty Sand (SM):** fine to medium-grained, olive brown to brown.

### Sandy Silt (ML):
- fine-grained sand, reddish-brown.
- Few gravel up to 1" in largest dimension

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/20/17.

---

**Table: Summary of Subsurface Conditions**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Summary</th>
<th>Drive</th>
<th>Blows</th>
<th>Moisture</th>
<th>Dry Unit Wt.</th>
<th>Other</th>
</tr>
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<tbody>
<tr>
<td>5</td>
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<td></td>
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<td>5/5/10</td>
<td>10</td>
<td>113</td>
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<td>15-81-158-01 A-29</td>
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<td>3/6/7</td>
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</tr>
</tbody>
</table>

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*Dates Drilled: 2/20/2017  Logged by: William Buckley  Checked By: Scot Mathis*

*Equipment: 8" Hollow Stem Auger  Driving Weight and Drop: 140 lbs / 30 in*

*Ground Surface Elevation (ft): 1583  Depth to Water (ft): NOT ENCOUNTERED*
Log of Boring No. BH-29

Dates Drilled: 2/21/2017
Logged by: William Buckley
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1564
Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS
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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
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<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

- ALLUVIUM:
  - SANDY SILT (ML): fine to medium-grained sand, brown.
  - SILTY SAND (SM): fine to coarse-grained, brown.
  - SANDY SILT (ML): fine to medium-grained sand, reddish-brown.
  - fine to coarse-grained sand

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/21/17.
## Log of Boring No. BH-30 (Turnout Facility No. 2)

Dates Drilled: 2/22/2017  
Logged by: William Buckley  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1608  
Depth to Water (ft): NOT ENCLOSED

### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td>DRIVE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BULK</td>
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<tr>
<td></td>
<td></td>
<td>BLOWS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MOISTURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DRY UNIT WT: (pcf)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OTHER</td>
</tr>
</tbody>
</table>

**ALLUVIUM:**  
**SILTY SAND (SM):** fine to medium-grained, brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
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</tr>
</thead>
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<td></td>
<td>4</td>
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<tr>
<td></td>
<td></td>
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</tbody>
</table>

**CLAYEY SAND (SC):** fine to medium-grained, brown.

- reddish-brown

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
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<table>
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<th>Graphic Log</th>
<th>SAMPLES</th>
</tr>
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<tr>
<td></td>
<td></td>
<td>109</td>
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</tbody>
</table>

**SILTY SAND (SM):** fine to coarse-grained, trace clay, reddish-brown.

End of boring at 26.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled with soil cuttings on 2/22/17.
### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
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</thead>
<tbody>
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<td>ALLUVIUM:</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SILTY SAND with GRAVEL (SM): fine to medium-grained, gravel up to 2&quot; in largest dimension, dark brown.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>8/14/14</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>7/12/22</td>
<td>6</td>
<td>123</td>
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<td></td>
</tr>
<tr>
<td>7/12/16</td>
<td>3</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>SILTY SAND (SM): fine to medium-grained, brown.</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings on 2/22/17.
### Log of Boring No. BH-32

**Dates Drilled:** 2/21/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1643  
**Depth to Water (ft):** NOT ENCOUNTERED

---

**SUMMARY OF SUBSURFACE CONDITIONS**

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
</tr>
<tr>
<td>5-10</td>
<td></td>
<td><strong>SILTY SAND (SM):</strong> fine to medium-grained, brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- light brown</td>
</tr>
<tr>
<td>10-15</td>
<td></td>
<td><strong>SAND (SP):</strong> fine to coarse-grained, light brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of boring at 16.5 feet bgs.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No groundwater encountered.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled with soil cuttings on 2/21/17.</td>
</tr>
</tbody>
</table>

---

**SAMPLES**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Date</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/6/11</td>
<td>6</td>
<td>118</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/6/8</td>
<td>2</td>
<td>112</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/13/15</td>
<td>2</td>
<td>115</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Cactus II Feeder**  
Approximately 29,700 Linear Feet of Pipeline  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch

---

**Converse Consultants**

Project ID: 15-81-158-01.GPJ; Template: LOG  
Project No. 15-81-158-01  
Drawing No. A-33
**Log of Boring No. BH-33**

**Dates Drilled:** 2/22/2017  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 1566  
**Depth to Water (ft):** NOT ENCOUNTERED

### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>8&quot; ASPHALT CONCRETE / 4&quot; AGGREGATE BASE:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to medium-grained, dark brown.</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td><strong>Sandy Silt (ML):</strong> fine to medium-grained sand, brown.</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to coarse-grained, few gravel up to 1&quot; in largest dimension, light brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 16.5 feet bgs.  
No groundwater encountered.  
Borehole backfilled and surface patched in accordance with City Standard MVSI-132-0 on 2/22/17.
### 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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**6" ASPHALT CONCRETE / 12" AGGREGATE BASE**

**ALLUVIUM**

**Silty Sand (SM):** fine to medium-grained, dark brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/12/7</td>
<td></td>
<td></td>
<td>9</td>
<td>120</td>
<td></td>
<td>col</td>
</tr>
<tr>
<td>4/6/12</td>
<td></td>
<td></td>
<td>6</td>
<td>108</td>
<td></td>
<td>de se r ca, er, ma, max</td>
</tr>
<tr>
<td>11/18/22</td>
<td></td>
<td></td>
<td>8</td>
<td>121</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8/14/16</td>
<td></td>
<td></td>
<td>5</td>
<td>115</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17/39/50-5&quot;</td>
<td></td>
<td></td>
<td>10</td>
<td>122</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**End of boring at 16.5 feet bgs.**

**No groundwater encountered.**

**Borehole backfilled with soil cuttings, tamped and surface patched with asphalt concrete on 6/21/18.**
### Log of Boring No. BH-35 (Turnout Facility No. 2)

**Dates Drilled:** 6/21/2018  
**Logged by:** William Buckley  
**Checked By:** Scot Mathis

**Equipment:** 6" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in

**Ground Surface Elevation (ft):** 1583  
**Depth to Water (ft):** 37

#### 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.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS</th>
<th>MOISTURE</th>
<th>DRY UNIT WT</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td></td>
<td>4/9/15</td>
<td>4</td>
<td>102</td>
<td></td>
<td></td>
<td>col</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>19/20/18</td>
<td>6</td>
<td>108</td>
<td></td>
<td></td>
<td>ds</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>8/13/20</td>
<td>5</td>
<td>115</td>
<td></td>
<td></td>
<td>c</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>20/50-4&quot;</td>
<td>10</td>
<td>123</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>13/18/25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>wa</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>26/30/50</td>
<td>3</td>
<td>122</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>6/10/11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>wa</td>
<td></td>
</tr>
</tbody>
</table>

**ALLUVIUM:**

**SILTY SAND (SM):** fine to medium-grained, brown.

- fine to coarse-grained, - light brown

---

**Converse Consultants**

Eastern Municipal Water District

Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities

City of Moreno Valley, Riverside County, California

For: Black & Veatch

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**Project No.:** 15-81-158-01  
**Drawing No.:** A-36a
**Log of Boring No. BH-35**

Dates Drilled: 6/21/2018  
Logged by: William Buckley  
Checked By: Scot Mathis

Equipment: 6" HOLLOW STEM AUGER  
Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 1583  
Depth to Water (ft): 37

---

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td></td>
</tr>
<tr>
<td>15-25</td>
<td></td>
</tr>
<tr>
<td>25-40</td>
<td></td>
</tr>
<tr>
<td>40-45</td>
<td></td>
</tr>
<tr>
<td>45-50</td>
<td></td>
</tr>
<tr>
<td>50-55</td>
<td></td>
</tr>
</tbody>
</table>

**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.

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/16/25</td>
<td>18</td>
<td>115</td>
<td>wa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9/20/36</td>
<td>wa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45/50-4&quot;</td>
<td>10</td>
<td>130</td>
<td>wa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16/16/40</td>
<td>wa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**ALLUVIUM**

SILTY SAND (SM): fine to medium-grained, brown.

End of boring at 51.5 feet bgs.  
Groundwater encountered at 37 feet bgs.  
Borehole backfilled with soil cuttings and tamped on 6/21/18.

**SAND WITH SILT (SP-SM):** fine to medium-grained, brown.
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

Results of these tests, which were performed on relatively undisturbed ring samples, in accordance to ASTM Standard D2216 and ASTM Standard D7263, were used to aid in soils classification and to provide quantitative measure of the in-situ dry density and moisture content. Data obtained from these tests provides qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Logs of Borings in Appendix A, Field Exploration.

Expansion Index

Seven representative bulk samples were tested to evaluate the expansion potential of the materials encountered. The tests were conducted in accordance with ASTM Standard D4829. Test results are presented in the following table.

Table No. B-1, Results of Expansion Index Tests

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Expansion Index</th>
<th>Expansion Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-11</td>
<td>15-20</td>
<td>Clayey Sand (SC)</td>
<td>51</td>
<td>Medium</td>
</tr>
<tr>
<td>BH-13</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-22</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-27</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-30</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-35</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>7</td>
<td>Very Low</td>
</tr>
</tbody>
</table>

Sand Equivalent

Eight 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-2, Sand Equivalent Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Sand Equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>15</td>
</tr>
<tr>
<td>BH-06</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>23</td>
</tr>
<tr>
<td>BH-10</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>13</td>
</tr>
<tr>
<td>BH-16</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>15</td>
</tr>
<tr>
<td>BH-22</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>12</td>
</tr>
<tr>
<td>BH-27</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>14</td>
</tr>
<tr>
<td>BH-30</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>13</td>
</tr>
<tr>
<td>BH-34</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>21</td>
</tr>
</tbody>
</table>

Soil Corrosivity Tests

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

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

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>pH</th>
<th>Soluble Sulfates (CA 417) (% by weight)</th>
<th>Soluble Chlorides (CA 422) (ppm)</th>
<th>Min. Resistivity (CA 643) (Ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>5-10</td>
<td>8.39</td>
<td>0.001</td>
<td>235</td>
<td>1,800</td>
</tr>
<tr>
<td>BH-06</td>
<td>5-10</td>
<td>8.62</td>
<td>0.006</td>
<td>160</td>
<td>17,000</td>
</tr>
<tr>
<td>BH-11</td>
<td>15-20</td>
<td>8.32</td>
<td>0.004</td>
<td>205</td>
<td>3,500</td>
</tr>
<tr>
<td>BH-13</td>
<td>15-20</td>
<td>8.32</td>
<td>0.007</td>
<td>245</td>
<td>1,200</td>
</tr>
<tr>
<td>BH-16</td>
<td>5-10</td>
<td>7.89</td>
<td>0.001</td>
<td>175</td>
<td>5,300</td>
</tr>
<tr>
<td>BH-22</td>
<td>5-10</td>
<td>7.95</td>
<td>0.003</td>
<td>155</td>
<td>5,300</td>
</tr>
<tr>
<td>BH-27</td>
<td>5-10</td>
<td>8.50</td>
<td>0.008</td>
<td>125</td>
<td>1,600</td>
</tr>
<tr>
<td>BH-30</td>
<td>5-10</td>
<td>8.48</td>
<td>0.001</td>
<td>125</td>
<td>5,200</td>
</tr>
<tr>
<td>BH-34</td>
<td>5-10</td>
<td>8.60</td>
<td>0.004</td>
<td>32</td>
<td>6,615</td>
</tr>
<tr>
<td>BH-35</td>
<td>0-5</td>
<td>8.20</td>
<td>0.005</td>
<td>35</td>
<td>4,043</td>
</tr>
</tbody>
</table>
Collapse Tests

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

Table No. B-4, Collapse Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Classification</th>
<th>Percent Swell + Percent Collapse -</th>
<th>Collapse Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>7.5</td>
<td>Silty Sand (SM)</td>
<td>-2.9</td>
<td>Moderate</td>
</tr>
<tr>
<td>BH-22</td>
<td>7.5</td>
<td>Silty Sand (SM)</td>
<td>-0.7</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-27</td>
<td>5.0</td>
<td>Silty Sand (SM)</td>
<td>-0.7</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-30</td>
<td>7.5</td>
<td>Silty Sand (SM)</td>
<td>-2.1</td>
<td>Moderate</td>
</tr>
<tr>
<td>BH-34</td>
<td>2.5</td>
<td>Silty Sand (SM)</td>
<td>-1.5</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-35</td>
<td>2.5</td>
<td>Silty Sand (SM)</td>
<td>-1.8</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-35*</td>
<td>10.0</td>
<td>Silty Sand (SM)</td>
<td>-1.5</td>
<td>Slight</td>
</tr>
</tbody>
</table>

(* consolidation test)

R-value

Six representative bulk soil samples were tested for resistance value (R-value) in accordance with the California Test Method 301. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table.

Table No. B-5, R-Value Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Classification</th>
<th>Measured R-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-02</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>35</td>
</tr>
<tr>
<td>BH-08</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>55</td>
</tr>
<tr>
<td>BH-14</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>32</td>
</tr>
<tr>
<td>BH-20</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>58</td>
</tr>
<tr>
<td>BH-27</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>41</td>
</tr>
<tr>
<td>BH-34</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>26</td>
</tr>
</tbody>
</table>
Percent Passing #200 Sieve

Tests of the percent passing the #200 sieve were performed on selected samples in accordance with ASTM Standard D2844. The results, including percent passing #200 sieve results from the grain size analyses, are presented in the following table.

Table No. B-6, Percentage Passing #200 Sieve Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Description</th>
<th>Passing #200 Sieve (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-01</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>34.01</td>
</tr>
<tr>
<td>BH-04</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>33.01</td>
</tr>
<tr>
<td>BH-06</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>29.01</td>
</tr>
<tr>
<td>BH-10</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>34.51</td>
</tr>
<tr>
<td>BH-11</td>
<td>15-20</td>
<td>Clayey Sand (SC)</td>
<td>48.01</td>
</tr>
<tr>
<td>BH-13</td>
<td>1-5</td>
<td>Clayey Sand (SC)</td>
<td>49.81</td>
</tr>
<tr>
<td>BH-14</td>
<td>15-20</td>
<td>Clayey Sand (SC)</td>
<td>39.01</td>
</tr>
<tr>
<td>BH-16</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>31.51</td>
</tr>
<tr>
<td>BH-18</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>25.71</td>
</tr>
<tr>
<td>BH-22</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>36.41</td>
</tr>
<tr>
<td>BH-25</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>44.51</td>
</tr>
<tr>
<td>BH-27</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>47.21</td>
</tr>
<tr>
<td>BH-30</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>36.61</td>
</tr>
<tr>
<td>BH-34</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>32.01</td>
</tr>
<tr>
<td>BH-35</td>
<td>0.5</td>
<td>Silty Sand (SM)</td>
<td>37.01</td>
</tr>
<tr>
<td>BH-35</td>
<td>20-21.5</td>
<td>Silty Sand (SM)</td>
<td>42.9</td>
</tr>
<tr>
<td>BH-35</td>
<td>30-31.5</td>
<td>Silty Sand (SM)</td>
<td>40.7</td>
</tr>
<tr>
<td>BH-35</td>
<td>40-41.5</td>
<td>Silty Sand (SM)</td>
<td>22.2</td>
</tr>
<tr>
<td>BH-35</td>
<td>50-51.5</td>
<td>Silty Sand (SM)</td>
<td>37.0</td>
</tr>
</tbody>
</table>

1 Result from grain size analysis.

Grain-Size Analyses

Representative soil samples at various depths were tested to determine the relative grain size distribution in accordance with the ASTM Standard C136. Test results are presented in the following table and graphically presented in Drawings No. B-1a through B-1c, Grain Size Distribution Test Results.
Table No. B-7, Grain Size Distribution Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Size in mm of percent passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>D_{60}</td>
</tr>
<tr>
<td>BH-01</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0.449</td>
</tr>
<tr>
<td>BH-04</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0.434</td>
</tr>
<tr>
<td>BH-06</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0.511</td>
</tr>
<tr>
<td>BH-10</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0.497</td>
</tr>
<tr>
<td>BH-11</td>
<td>15-20</td>
<td>Clayey Sand (SC)</td>
<td>0.231</td>
</tr>
<tr>
<td>BH-13</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0.149</td>
</tr>
<tr>
<td>BH-14</td>
<td>15-20</td>
<td>Clayey Sand (SM)</td>
<td>0.424</td>
</tr>
<tr>
<td>BH-16</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0.358</td>
</tr>
<tr>
<td>BH-18</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>0.711</td>
</tr>
<tr>
<td>BH-22</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0.357</td>
</tr>
<tr>
<td>BH-25</td>
<td>1-5</td>
<td>Silty Sand (SM)</td>
<td>0.145</td>
</tr>
<tr>
<td>BH-27</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0.180</td>
</tr>
<tr>
<td>BH-30</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0.334</td>
</tr>
<tr>
<td>BH-34</td>
<td>5-10</td>
<td>Silty Sand (SM)</td>
<td>0.411</td>
</tr>
<tr>
<td>BH-35</td>
<td>0-5</td>
<td>Silty Sand (SM)</td>
<td>0.272</td>
</tr>
</tbody>
</table>

Maximum Density and Optimum Moisture Content Tests

Seven laboratory maximum dry density and optimum moisture content relationship tests were performed on representative bulk samples. The tests were conducted in accordance with the ASTM Standard D1557 test method. Test results are presented on Drawings No. B-2a and 2b, Moisture-Density Relationship Results, and in the following table.

Table No B-8, Summary of Moisture-Density Relationship Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Optimum Moisture (%)</th>
<th>Maximum Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-06</td>
<td>1-5</td>
<td>Silty Sand (SM), brown</td>
<td>6.7</td>
<td>134.8</td>
</tr>
<tr>
<td>BH-13</td>
<td>1-5</td>
<td>Silty Sand (SM), brown</td>
<td>9.5</td>
<td>130.5</td>
</tr>
<tr>
<td>BH-14</td>
<td>15-20</td>
<td>Clayey Sand (SC), reddish-brown</td>
<td>9.8</td>
<td>130.0</td>
</tr>
<tr>
<td>BH-22</td>
<td>1-5</td>
<td>Silty Sand (SM), dark brown</td>
<td>7.5</td>
<td>133.5</td>
</tr>
<tr>
<td>BH-30</td>
<td>0-5</td>
<td>Silty Sand (SM), brown</td>
<td>8.2</td>
<td>134.5</td>
</tr>
<tr>
<td>BH-34</td>
<td>5-10</td>
<td>Silty Sand (SM), Brown</td>
<td>8.2</td>
<td>135.4</td>
</tr>
<tr>
<td>BH-35</td>
<td>0-5</td>
<td>Silty Sand (SM), Brown</td>
<td>9.0</td>
<td>130.0</td>
</tr>
</tbody>
</table>
Direct Shear Tests

Ten direct shear tests were performed on relatively undisturbed samples in soaked moisture condition in accordance with ASTM D3080. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawings No. B-3 through B-12, *Direct Shear Test Results*, and the following table.

### Table No. B-9, Summary of Direct Shear Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Ultimate Strength Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Friction Angle (degrees)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cohesion (psf)</td>
</tr>
<tr>
<td>BH-01</td>
<td>2.5-4.0</td>
<td>Silty Sand (SM)</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>260</td>
</tr>
<tr>
<td>BH-05</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
</tr>
<tr>
<td>BH-10</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90</td>
</tr>
<tr>
<td>BH-11</td>
<td>20.0-21.5</td>
<td>Silty Sand (SM)</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>130</td>
</tr>
<tr>
<td>BH-13</td>
<td>5.0-6.5</td>
<td>Sandy Silt (ML)</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
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<tr>
<td>BH-14</td>
<td>15.0-16.5</td>
<td>Silty Sand (SM)</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>340</td>
</tr>
<tr>
<td>BH-22</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50</td>
</tr>
<tr>
<td>BH-30</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>110</td>
</tr>
<tr>
<td>BH-34</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>BH-35</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80</td>
</tr>
</tbody>
</table>

Consolidation Tests

This 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-13, *Consolidation Test Result*.

**Sample Storage**

Soil samples currently stored in our laboratory will be discarded thirty days after the date of the final report, unless this office receives a specific request to retain the samples for a longer period.
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants
Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01
Drawing No. B-1a

Project ID: 15-81-158-01.GPJ; Template: GRAIN SIZE
GRAIN SIZE DISTRIBUTION RESULTS

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>Description</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Cc</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-13</td>
<td>1-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-14</td>
<td>15-20</td>
<td>CLAYEY SAND (SC)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-16</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-18</td>
<td>5-10</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-22</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-13</td>
<td>1-5</td>
<td>9.5</td>
<td>0.149</td>
<td></td>
<td></td>
<td>3.0</td>
<td>47.2</td>
<td>49.8</td>
<td></td>
</tr>
<tr>
<td>BH-14</td>
<td>15-20</td>
<td>9.5</td>
<td>0.424</td>
<td></td>
<td></td>
<td>3.0</td>
<td>58.0</td>
<td>39.0</td>
<td></td>
</tr>
<tr>
<td>BH-16</td>
<td>0-5</td>
<td>9.5</td>
<td>0.358</td>
<td></td>
<td></td>
<td>2.3</td>
<td>65.6</td>
<td>31.5</td>
<td></td>
</tr>
<tr>
<td>BH-18</td>
<td>5-10</td>
<td>9.5</td>
<td>0.711</td>
<td>0.112</td>
<td></td>
<td>5.0</td>
<td>69.3</td>
<td>25.7</td>
<td></td>
</tr>
<tr>
<td>BH-22</td>
<td>0-5</td>
<td>9.5</td>
<td>0.357</td>
<td></td>
<td></td>
<td>3.0</td>
<td>60.6</td>
<td>36.4</td>
<td></td>
</tr>
</tbody>
</table>

Converse Consultants

Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project ID: 15-81-158-01.GPJ; Template: GRAIN SIZE
GRAIN SIZE DISTRIBUTION RESULTS

Boring No. Depth (ft) Description LL PL PI Cc Cu

COBBLES

GRAVEL

SAND

SILT OR CLAY

BH-25 1-5 SILTY SAND (SM)
BH-27 0-5 SILTY SAND (SM)
BH-30 0-5 SILTY SAND (SM)
BH-34 5-10 SILTY SAND (SM)
BH-35 0-5 SILTY SAND (SM)

Boring No. Depth (ft) D100 D60 D30 D10 %Gravel %Sand %Silt %Clay

BH-25 1-5 12.5 0.146 5.4 47.1 44.5
BH-27 0-5 9.5 0.18 2.0 50.8 47.2
BH-30 0-5 9.5 0.334 1.0 62.4 36.6
BH-34 5-10 9.5 0.411 2.0 66.0 32.0
BH-35 0-5 9.5 0.272 1.0 62.0 37.0

Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Converse Consultants

Project No. 15-81-158-01
Drawing No. B-1c
### Curves of 100% Saturation for Specific Gravity Equal to:
- 2.80
- 2.70
- 2.60

### MOISTURE-DENSITY RELATIONSHIP RESULTS

**Astm Test Method**

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>BORING NO.</th>
<th>DEPTH (ft)</th>
<th>DESCRIPTION</th>
<th>ASTM TEST METHOD</th>
<th>OPTIMUM WATER, %</th>
<th>MAXIMUM DRY DENSITY, pcf</th>
</tr>
</thead>
<tbody>
<tr>
<td>●</td>
<td>BH-06</td>
<td>1-5</td>
<td>SILTY SAND (SM), Brown</td>
<td>D1557 - B</td>
<td>6.7</td>
<td>134.8</td>
</tr>
<tr>
<td>□</td>
<td>BH-13</td>
<td>1-5</td>
<td>SILTY SAND (SM), Brown</td>
<td>D1557 - B</td>
<td>9.5</td>
<td>130.5</td>
</tr>
<tr>
<td>▲</td>
<td>BH-14</td>
<td>15-20</td>
<td>CLAYEY SAND (SC), Reddish Brown</td>
<td>D1557 - B</td>
<td>9.8</td>
<td>130.0</td>
</tr>
<tr>
<td>☆</td>
<td>BH-22</td>
<td>0-5</td>
<td>SILTY SAND (SM), Dark Brown</td>
<td>D1557 - B</td>
<td>7.5</td>
<td>133.5</td>
</tr>
<tr>
<td>◊</td>
<td>BH-30</td>
<td>0-5</td>
<td>SILTY SAND (SM), Brown</td>
<td>D1557 - B</td>
<td>8.2</td>
<td>134.5</td>
</tr>
</tbody>
</table>

**Project No.**: 15-81-158-01

**Drawing No.**: B-2a

**Converse Consultants**

Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

**Project ID**: 15-81-158-01.GPJ; **Template**: COMPACTION
### DIRECT SHEAR TEST RESULTS

**Cactus II Feeder**  
**Eastern Municipal Water District**  
**Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities**  
**City of Moreno Valley, Riverside County, California**  
**For: Black & Veatch**  

**Project ID:** 15-81-158-01  
**Drawing No.:** B-3

#### BORING NO.: BH-01  
**DEPTH (ft):** 2.5-4.0

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>SIETY SAND (SM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COHESION (psf)</td>
<td>260</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>5.3</td>
</tr>
<tr>
<td>FRICTION ANGLE (degrees)</td>
<td>37</td>
</tr>
<tr>
<td>DRY DENSITY (pcf)</td>
<td>107.3</td>
</tr>
</tbody>
</table>

**NOTE:** Ultimate Strength.
DIRECT SHEAR TEST RESULTS

Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01
Drawing No. B-4

BORING NO. : BH-05
DESCRIPTION : SILTY SAND (SM)
COHESION (psf) : 60
MOISTURE CONTENT (%) : 8.6
DEPTH (ft) : 5.0-6.5
FRICTION ANGLE (degrees) : 30
DRY DENSITY (pcf) : 107.4

NOTE: Ultimate Strength.
DIRECT SHEAR TEST RESULTS

Cactus II Feeder
East Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DESCRIPTION</th>
<th>DEPTH (ft)</th>
<th>COHESION (psf)</th>
<th>FRICTION ANGLE (degrees)</th>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-10</td>
<td>SILTY SAND (SM)</td>
<td>5.0-6.5</td>
<td>90</td>
<td>29</td>
<td>7.5</td>
<td>106.3</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
DIRECT SHEAR TEST RESULTS

Boring No.: BH-11  
Depth (ft): 20.0-21.5

Description: Silty Sand (SM)

Cohesion (psf): 130  
Friction Angle (degrees): 33

Moisture Content (%): 7.0  
Dry Density (pcf): 111.0

NOTE: Ultimate Strength.
**DIRECT SHEAR TEST RESULTS**

**Cactus II Feeder**
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

**Project No.:** 15-81-158-01  **Drawing No.:** B-7

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DESCRIPTION</th>
<th>DEPTH (ft)</th>
<th>COHESION (psf)</th>
<th>FRICTION ANGLE (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-13</td>
<td>SANDY SILT (ML)</td>
<td>5.0-6.5</td>
<td>100</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td>MOISTURE CONTENT (%)</td>
<td>12.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DRY DENSITY (pcf)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:** Ultimate Strength.
Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01
Drawing No. B-8

Converse Consultants

**DIRECT SHEAR TEST RESULTS**

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-14</th>
<th>DESCRIPTION</th>
<th>CLAYEY SAND (SC)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COHESION (psf)</td>
<td>340</td>
<td>FRICTION ANGLE (degrees):</td>
<td>30</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>11.9</td>
<td>DRY DENSITY (pcf) :</td>
<td>119.6</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

**DIRECT SHEAR TEST RESULTS**

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DESCRIPTION</th>
<th>DEPTH (ft)</th>
<th>COHESION (psf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>FRICTION ANGLE (degrees):</th>
<th>DRY DENSITY (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-22</td>
<td>SILTY SAND (SM)</td>
<td>5.0-6.5</td>
<td>50</td>
<td>7.6</td>
<td>29</td>
<td>113.0</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
**DIRECT SHEAR TEST RESULTS**

**Project ID:** 15-81-158-01  
**Drawing No.:** B-10

Cactus II Feeder  
Eastern Municipal Water District  
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch

**BORING NO.:** BH-30  
**DEPTH (ft):** 7.5-9.0

**DESCRIPTION:** SILTY SAND (SM)

**COHESION (psf):** 110  
**FRICTION ANGLE (degrees):** 31

**MOISTURE CONTENT (%):** 8.0  
**DRY DENSITY (pcf):** 111.0

NOTE: Ultimate Strength.
DIRECT SHEAR TEST RESULTS

Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01
Drawing No. B-11

Project ID: 15-81-158-01.GPJ; Template: DIRECT SHEAR

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-34</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESCRIPTION</td>
<td>SILTY SAND (SM)</td>
</tr>
<tr>
<td>DEPTH (ft)</td>
<td>5.0-6.5</td>
</tr>
<tr>
<td>COHESION (psf)</td>
<td>10</td>
</tr>
<tr>
<td>FRICTION ANGLE (degrees)</td>
<td>32</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>6.0</td>
</tr>
<tr>
<td>DRY DENSITY (pcf)</td>
<td>109.0</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
### DIRECT SHEAR TEST RESULTS

**Cactus II Feeder**  
Eastern Municipal Water District  
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DESCRIPTION</th>
<th>DEPTH (ft)</th>
<th>COHESION (psf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SHEAR STRENGTH, psf</th>
<th>FRICTION ANGLE (degrees)</th>
<th>DRY DENSITY (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-35</td>
<td>SILTY SAND (SM)</td>
<td>5.0-6.5</td>
<td>80</td>
<td>5.6</td>
<td>108.3</td>
<td>28</td>
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</tr>
</tbody>
</table>

**NOTE:** Ultimate Strength.
NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

BORING NO. : BH-35 DEPTH (ft) : 10.0-11.5
DESCRIPTION : SILTY SAND (SM)

<table>
<thead>
<tr>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>PERCENT SATURATION</th>
<th>VOID RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL 7</td>
<td>118</td>
<td>46</td>
<td>0.401</td>
</tr>
<tr>
<td>FINAL 13</td>
<td>121</td>
<td>100</td>
<td>0.363</td>
</tr>
</tbody>
</table>

Converse Consultants
Cactus II Feeder
Eastern Municipal Water District
Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project No. 15-81-158-01 Drawing No. B-13
Appendix C

Seismic Refraction Survey
APPENDIX C

SEISMIC REFRACTION SURVEY

Southwest Geophysics, Inc. was retained to perform a seismic refraction survey of selected portions of the alignment. Seven seismic refraction traverses were conducted at locations selected to supplement information obtained from the soil borings. The complete Southwest Geophysics report is included in this appendix. Additional interpretation and application of the seismic refraction data is presented in Section 5.4, *Excavatability*.

The results of the seismic refraction traverses are summarized in the following table. This summary is provided for convenience only. When interpreting the results, it is necessary to review the seismic profiles contained in the Southwest Geophysics report, as well as the discussion provided in that report and in Section 5.4, *Excavatability*. It should be anticipated that rock conditions will vary locally from the values indicated below, and that excavation conditions outside the seismic traverse areas may be more difficult than indicated.

**Table No. C-1, Summary of Seismic Refraction Traverses**

<table>
<thead>
<tr>
<th>Traverse</th>
<th>Location</th>
<th>Approximate Depth to Difficult Excavation or Blasting</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL-1</td>
<td>Alessandro Boulevard between Kitching Street and Chara Street</td>
<td>9 to 15 feet</td>
</tr>
<tr>
<td>SL-2</td>
<td>Alessandro Boulevard between Chervil Court and Lasselle Street</td>
<td>10 to 25 feet</td>
</tr>
<tr>
<td>SL-3</td>
<td>Alessandro Boulevard between Lasselle Street and Darwin Drive</td>
<td>0 to 5 feet</td>
</tr>
<tr>
<td>SL-4</td>
<td>Alessandro Boulevard between Lasselle Street and Darwin Drive</td>
<td>0 to 5 feet</td>
</tr>
<tr>
<td>SL-5</td>
<td>Alessandro Boulevard between Lasselle Street and Darwin Drive</td>
<td>0 to 10 feet</td>
</tr>
<tr>
<td>SL-6</td>
<td>Alessandro Boulevard east of Darwin Drive</td>
<td>10 to 20 feet</td>
</tr>
<tr>
<td>SL-7</td>
<td>Alessandro Boulevard between Chara Street and Chervil Court</td>
<td>15 to 25 feet</td>
</tr>
</tbody>
</table>
SEISMIC REFRACTION SURVEY
CACTUS II FEEDER
MORENO VALLEY, CALIFORNIA

PREPARED FOR:
Converse Consultants
10391 Corporate Drive
Redlands, CA 92373

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

March 17, 2017
Project No. 117078
March 17, 2017  
Project No. 117078

Mr. James Burnham  
Converse Consultants  
10391 Corporate Drive  
Redlands, CA  92373

Subject: Seismic Refraction Survey  
Cactus II Feeder  
Moreno Valley, California

Dear Mr. Burnham:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Cactus II Feeder project located in Moreno Valley, California. Specifically, our survey consisted of performing seven seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions please contact the undersigned at your convenience.

Sincerely,

SOUTHWEST GEOPHYSICS, INC.

Afrildo Iko Syahrial  
Project Geologist/Geophysicist

Principal Geologist/Geophysicist

AIS/HV/hv

Distribution: Addressee (electronic)
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2. SCOPE OF SERVICES ............................................................................................................1
3. SITE DESCRIPTION ...............................................................................................................1
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Figure 3b – Site Photographs (SL-5 to SL-7)
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Figure 4d – Seismic Profile, SL-4
Figure 4e – Seismic Profile, SL-5
Figure 4f – Seismic Profile, SL-6
Figure 4g – Seismic Profile, SL-7
1. INTRODUCTION
In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Cactus II Feeder project located in Moreno Valley, California (Figure 1). Specifically, our survey consisted of performing seven seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES
Our scope of services included:
• Performance of seven seismic P-wave refraction lines at the project site.
• Compilation and analysis of the data collected.
• Preparation of this data report presenting our results, conclusions and recommendations.

3. SITE DESCRIPTION
The project site is located along Alessandro Boulevard roughly between Kitching Street and Darwin Drive in Moreno Valley, California (Figure 1). The seismic lines were located along the dirt shoulder adjacent to Alessandro Boulevard. Topography immediately adjacent to the roadway is generally flat; however, hills with exposures of granitic rock are nearby particularly in the area of lines SL-3 through SL-6. Figures 2, 3a and 3b depict the site conditions in the area of the seismic traverses.

Based on our discussions with you it is our understanding that the project involves the construction of a new water pipeline. Cuts up to 15 feet deep may be performed.

4. SURVEY METHODOLOGY
A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the rippability characteristics of the subsurface materials and to develop subsurface velocity profiles of the areas surveyed. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves
generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Seven seismic lines (SL-1 through SL-7) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogeneous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assume that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock rippability. These characteristics may also vary with location and depth.
For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

<table>
<thead>
<tr>
<th>Table 1 – Rippability Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seismic P-wave Velocity</strong></td>
</tr>
<tr>
<td>0 to 2,000 feet/second</td>
</tr>
<tr>
<td>2,000 to 4,000 feet/second</td>
</tr>
<tr>
<td>4,000 to 5,500 feet/second</td>
</tr>
<tr>
<td>5,500 to 7,000 feet/second</td>
</tr>
<tr>
<td>Greater than 7,000 feet/second</td>
</tr>
</tbody>
</table>

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2011). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. ANALYSIS AND RESULTS

As previously indicated, seven seismic traverses were conducted as part of our study. The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Figures 4a through 4g present the velocity models generated from our study. The approximate locations of the seismic refraction traverses are shown on the Line Location Map (Figure 2). In
general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil overlying granitic bedrock with varying degrees of weathering. Distinct vertical and lateral velocity variations are evident in the models. These inhomogeneities are likely related to the presence of remnant boulders, intrusions and differential weathering of the bedrock materials. It is also evident in the tomography models that the depth to bedrock is highly variable across the site.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. Furthermore, blasting may be required depending on the excavation depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophys-
ics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties’ sole risk.
8. SELECTED REFERENCES


Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

TOMOGRAPHY MODEL

Relative Elevation (ft)

Distance (ft)

Velocity (ft/s)

SEISMIC PROFILE
SL-1
Cactus II Feeder
Moreno Valley, California
Note: Contour Interval = 1,000 feet per second

Project No.: 117078
Date: 03/17

Figure 4a
TOMOGRAPHY MODEL

SEISMIC PROFILE
SL-4
Cactus II Feeder
Moreno Valley, California

Note: Contour Interval = 1,000 feet per second
Appendix D

Settlement Analysis
APPENDIX D

SETTLEMENT ANALYSIS

The subsurface data obtained from the three 51.5-foot borings (Boring BH-02, BH-24 and BH-35) drilled during the field investigation were used to evaluate the dynamic settlement due to potential liquefaction and densification of relatively loose sediments subjected to ground shaking during earthquakes.

The dynamic analysis was performed using Liquefy Pro (Civiltech, 2012). An earthquake magnitude of M7.7 and a peak ground acceleration (PGA) of 0.568g for boring BH-02 and BH-35 and 0.756g for BH-24, where g is the acceleration due to gravity, were selected for this analysis. The PGA was based on the CBC seismic design parameters presented in Section 8.2, CBC Seismic Design Parameters. An analysis considering both historical and current groundwater conditions were performed for each boring.

The results of our analyses are presented on Plates D-1 through D-6 and summarized in the following table.

Table D-1, Estimated Dynamic Settlement

<table>
<thead>
<tr>
<th>Location</th>
<th>Groundwater Conditions (feet bgs)</th>
<th>Dynamic Settlement (inches)</th>
<th>Differential Dynamic Settlement (inch/40 linear feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-02</td>
<td>14 (HHGL*)</td>
<td>0.52</td>
<td>0.26</td>
</tr>
<tr>
<td>BH-02</td>
<td>23 (Current)</td>
<td>0.14</td>
<td>0.07</td>
</tr>
<tr>
<td>BH-24</td>
<td>25(HHGL*)</td>
<td>3.25</td>
<td>1.63</td>
</tr>
<tr>
<td>BH-24</td>
<td>&gt;50 (Current)</td>
<td>2.84</td>
<td>1.42</td>
</tr>
<tr>
<td>BH-35</td>
<td>25(HHGL*)</td>
<td>0.91</td>
<td>0.46</td>
</tr>
<tr>
<td>BH-35</td>
<td>37 (Current)</td>
<td>0.48</td>
<td>0.24</td>
</tr>
</tbody>
</table>

* Historic High Groundwater Level (HHGL)

Based on our analysis, the turnout facilities have the potential for up to 3.3 inches of dynamic settlement. Differential settlement is estimated as half of the total settlement over a horizontal distance of 40 feet.
LIQUEFACTION ANALYSIS
Cactus II Feeder

Hole No. = BH-24  Water Depth = 50 ft  Surface Elev. = 1582
Magnitude = 7.7  Acceleration = 0.756g

Soil Description:
- Silty Sand
- Sandy Silt
- Silty Sand
- Sandy Silt

Shear Stress Ratio:

Factor of Safety:

Settlement:
- S = 2.84 in.

Shaded Zone has Liquefaction Potential

CivilTech Corporation
Plate D-4
Appendix E

Trench Backfill and Roadway Repair
BACKFILL MATERIAL OR CONTROL DENSITY FILL PER EMWD SPECIFICATIONS AND BACKFILL RESOLUTION NO. 3224. PLACE PRIOR TO THE END OF EACH WORK DAY UNLESS OTHERWISE APPROVED BY THE ENGINEER. COMPACTION AND PLACEMENT METHODS PER THE SPECIFICATIONS.

NOTES:

1.) SEE ADDITIONAL REQUIREMENTS, STD No MVSI-132C-MOD, EMWD STD. DWG B-286B, EMWD SPECS AND DRAWINGS.

2.) ALL TRENCHING AND BACKFILL SHALL BE DONE IN ACCORDANCE WITH SECTION 306, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, LATEST EDITION, AND EMWD SPECIFICATIONS.

3.) PER EMWD SPECS. PROTRUDING PARTICLES LARGER THAN 2" SHALL BE REMOVED FROM THE TRENCH BOTTOM.

4.) INCREASE BEDDING UNDER PIPE FROM 4" TO 6" FOR ROCK SUBGRADES.

5.) USE THIS STANDARD PLAN FOR UP TO AND INCLUDING 12" DIAMETER WATER LINE WITH 36" COVER OVER PIPE.

6.) LANE WIDTH REQUIREMENT MAY BE REDUCED AT DISCRETION OF CITY ENGINEER.

7.) 1-1/2 SACK CEMENT SLURRY MAY BE USED IF USED FOR TRENCH BACKFILL.

6.) ITEMS IN BOLD REFLECT MODIFICATIONS FROM THE CITY STANDARD.
FINISH OVERLAY TO BE PLACED NO LATER THAN 15 DAYS AFTER BASE PLACING, APPLY TACK AND LEVELING COURSE TO BRING WITHIN 0.10' OF EXISTING GRADE. SURFACE COURSE SHALL BE TYPE III C3-P664-10 AC.

BASE COURSE SHALL BE TYPE III B2-PG64-10 AC

EXISTING PAVEMENT MATCH EXIST AGGREGATE BASE - 4" MIN IF EXIST IS 0" TO 4" (SEE NOTE 7)

MATCH EXIST PAVEMENT + 1" 4" MINIMUM 1.5" -

COLD MILL EXISTING PAVEMENT 1.5" DEEP

FOR PERPENDICULAR TRENCH, MIN WIDTH = 6 FEET

FOR PARALLEL TRENCH FULL LANE WIDTH (TO NEAREST STRIPING) (SEE NOTE 6)

BACKFILL MATERIAL OR CONTROL DENSITY FILL PER EMWD SPECIFICATIONS AND BACKFILL RESOLUTION NO.3224. PLACE PRIOR TO THE END OF EACH WORK DAY UNLESS OTHERWISE APPROVED BY THE ENGINEER. COMPACTION AND PLACEMENT METHODS PER THE SPECIFICATIONS.

STRUCTURAL ZONE

INTERMEDIATE ZONE

PIPE ZONE

BEDDING:
BEDDING MATERIAL PER EMWD SPECIFICATIONS

COMPACITION AND PLACEMENT METHODS PER THE SPECIFICATIONS.

NOTES:

1.) SEE ADDITIONAL REQUIREMENTS, STD No MVSI-132C-1-MOD, EMWD STD. DWG B-286B, EMWD SPECS AND DRAWINGS.

2.) ALL TRENCHING AND BACKFILL SHALL BE DONE IN ACCORDANCE WITH SECTION 306, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, LATEST EDITION, AND EMWD SPECIFICATIONS.

3.) ALL TEMPORARY PAVING SHALL HAVE A MINIMUM 2" OF AC ON LOCAL STREETS AND 3" ON ALL OTHERS.

4.) PER EMWD SPECS. PROTRUDING PARTICLES LARGER THAN 2" SHALL BE REMOVED FROM THE TRENCH BOTTOM.

5.) USE THIS STANDARD PLAN FOR WATER LINE OF GREATER THAN 12" DIAMETER. MINIMUM COVER OVER PIPE IS 48''.

6.) LANE WIDTH REQUIREMENT MAY BE REDUCED AT DISCRETION OF CITY ENGINEER.

7.) 1/2 SACK CEMENT SLURRY MAY BE USED IF USED FOR TRENCH BACKFILL.

8.) ITEMS IN BOLD REFLECT MODIFICATIONS FROM THE CITY STANDARD.
Appendix F

Monitoring Well Installation Report
MONITORING WELL INSTALLATION REPORT

CACTUS II FEEDER PROJECT
CITY OF MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 15-81-158-03

Prepared For:
BLACK & VEATCH
Mr. Jeremy Clemmons, PE
300 Ranchero Drive, Suite 250
San Marcos, CA 92069

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

January 10, 2019
January 10, 2019

Mr. Jeremy Clemmons, PE
Engineering Manager
Black & Veatch
300 Ranchero Drive, Suite 250
San Marcos, CA 92069

Subject: **MONITORING WELL INSTALLATION REPORT**

**Cactus II Feeder Project**
City of Moreno Valley, Riverside County, California
Converse Project No. 15-81-158-03

Dear Mr. Clemmons:

Converse Consultants (Converse) has prepared this letter to provide the results of the installation of Monitoring Wells MW-1 and MW-2 within the City of Moreno Valley, Riverside County, California. This task was completed in accordance with our revised proposal dated April 23, 2018 and your Amendment No. 3 dated June 5, 2018.

**BACKGROUND**

The purpose of the wells is to monitor the groundwater levels for the Cactus II Feeder project at the west end of the pipeline alignment located within an Eastern Municipal Water District (EMWD) facility and near the Kitching Channel crossing at Alessandro Boulevard. The Cactus II Feeder project will consist of approximately 31,000 linear feet of 30-inch to 42-inch diameter water pipeline. The pipeline construction will also include 4 turnout facilities.

A monitoring well plan created by Converse and approved by Black & Veatch described the installation of 2 piezometer monitoring wells at different locations along the Cactus II Feeder Project within the City of Moreno Valley, Riverside County, California. Monitoring Well 1 (MW-1) is located at the Kitching Channel crossing in a landscape median on Alessandro Boulevard, directly west of Kitching Street. Monitoring Well 2 (MW-2) is located within an EMWD facility located southeast of the intersection of Cactus Avenue and Heacock Street. The depths of the wells are 30 and 25 feet bgs, respectively.

**SOIL BORINGS**

Two soil borings (MW-1 and MW-2) were drilled on September 7 and 10, 2018 to their planned depths of 30 and 25 feet below existing ground surface (bgs), respectively. The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow stem auger. Soils were continuously logged and classified in the field by a
Converse geologist based on visual/manual examination in accordance with the Unified Soil Classification System.

Groundwater was encountered during drilling at depths of approximately 27 and 23 feet bgs in MW-1 and MW-2, respectively.

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

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

**MONITORING WELL CONSTRUCTION**

The monitoring wells were constructed in general accordance with the monitoring well plan created by Converse that was approved by Black & Veatch.

The well casings consisted of 2-inch-diameter, Schedule 80 PVC with flush-threaded ends. Slotted casing with 0.01-inch slots was installed from 1 foot above the bottom of each borehole up to the existing ground surface. A perforated cap was placed on the bottom of each casing. Solid casing was installed from the top of the slotted casing to above the ground surface. Vented slip caps were placed on the top of each casing. A filter pack consisting of #2/12 sand was placed outside of the casing to approximately 1 foot above the top of the slotted casing. Bentonite chips were placed above the filter sand up to approximately 1.5 feet bgs and hydrated to form a seal. The borehole annulus above the seal to the ground surface was filled with cement-bentonite grout with a cement:water:bentonite ratio of 1:6.6:0.4 by weight. Surface completions consisting of bolted steel covers set in concrete pads were installed on September 7 and 10, 2018 for MW-1 and MW-2, respectively, as shown in the pictures below. The initial depths to water were measured after well installation at 21.5 feet bgs in MW-1 and 20.5 feet bgs in MW-2.
Photograph No. 1: MW-1 located in a raised median west of the intersection of Kitching Street and Alessandro Boulevard.

Photograph No. 2: MW-2 located within an EMWD facility southeast of the intersection of Heacock Street and Cactus Avenue.
MONITORING WELL MEASUREMENTS

The depths of water for both wells monitored and recorded between September 10, 2018 and December 12, 2018 are shown in the table below.

Table No. 1, Summary of Groundwater Depth Measurements

<table>
<thead>
<tr>
<th>Date</th>
<th>Monitoring Well</th>
<th>Location</th>
<th>Depth of Well (ft)</th>
<th>Groundwater Depth (ft. bgs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/10/18</td>
<td>MW-1</td>
<td>West of Alessandro Blvd. and Kitching St.</td>
<td>30</td>
<td>21.5</td>
</tr>
<tr>
<td></td>
<td>MW-2</td>
<td>EMWD Facility</td>
<td>25</td>
<td>20.5</td>
</tr>
<tr>
<td>10/10/18</td>
<td>MW-1</td>
<td>West of Alessandro Blvd. and Kitching St.</td>
<td>30</td>
<td>20.5</td>
</tr>
<tr>
<td></td>
<td>MW-2</td>
<td>EMWD Facility</td>
<td>25</td>
<td>20.2</td>
</tr>
<tr>
<td>11/16/18</td>
<td>MW-1</td>
<td>West of Alessandro Blvd. and Kitching St.</td>
<td>30</td>
<td>20.4</td>
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<tr>
<td></td>
<td>MW-2</td>
<td>EMWD Facility</td>
<td>25</td>
<td>20.2</td>
</tr>
<tr>
<td>12/12/18</td>
<td>MW-1</td>
<td>West of Alessandro Blvd. and Kitching St.</td>
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<tr>
<td></td>
<td>MW-2</td>
<td>EMWD Facility</td>
<td>25</td>
<td>20.2</td>
</tr>
</tbody>
</table>

The water levels in each monitoring well were measured monthly for a period of 3 months after the installation.

CLOSURE

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

Converse Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others. Site exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others.
We appreciate the opportunity to be of continuing service to Black & Veatch. If you should have any questions, or if we can provide any additional assistance, please call the undersigned at 909-796-0544.

CONVERSE CONSULTANTS

James Burnham, PG
Project Geologist

Distr.: Address (e-mail)

Attachments: Figure No. 1, Approximate Site Locations Map
Figures No. 2a & 2b, Approximate Well Location Map
Drawing No. 1, Unified Soil Classification and Key to Boring Log Symbols
Drawings No. 2 and 3, Logs of Borings
Appendix

Figures 1, 2a, 2b and Logs of Borings
Approximate Site Locations

Project: Cactus II Feeder Project
Location: Intersections of Alessandro & Kitching and Cactus & Heacock
City of Moreno Valley, Riverside County, California
For: Black and Veatch

Converse Consultants

FIGURE NO. 1
Approximate Well Location Map

Project: Cactus II Feeder Project
Location: West of Intersection of Alessandro Blvd and Kitching Street
City of Moreno Valley, Riverside County, California
For: Black and Veatch

MW-1
Number and Approximate Location of Monitoring Well

© 2018 Google
Approximate Well Location Map

Project: Cactus II Feeder Project
Location: Southeast of Intersection of Cactus Ave and Heacock Street
City of Moreno Valley, Riverside County, California
For: Black and Veatch

Project No. 15-81-158-03

Figure No. 2b
### SOIL CLASSIFICATION CHART

<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>SYMBOLS</th>
<th>TYPICAL DESCRIPTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GRAVEL AND GRAVELLY SOILS</strong></td>
<td>GW</td>
<td>WELL-GRADED GRAVELS, GRAVELY - SAND MIXTURES, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>SAND AND SANDY SOILS</strong></td>
<td>SP</td>
<td>POORLY-GRADED SANDS, GRAVELY SAND, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>SANDS WITH FINES</strong></td>
<td>SC</td>
<td>CLAYEY SANDS, SAND - CLAY MIXTURES</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>ML</td>
<td>INORGANIC SOTS AND VERY FINE SANDS, ROCK FLOOR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SOTS WITH SLIGHT PLASTICITY</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>OL</td>
<td>ORGANIC SLOTS AND ORGANIC CLAYEY SLOTS OF LOW PLASTICITY</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>MH</td>
<td>INORGANIC CLAYS OF HIGH PLASTICITY</td>
</tr>
<tr>
<td><strong>FINE GRAINED SOILS</strong></td>
<td>CH</td>
<td>ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY</td>
</tr>
<tr>
<td><strong>HIGHLY ORGANIC SOILS</strong></td>
<td>PT</td>
<td>PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS</td>
</tr>
</tbody>
</table>

### SAMPLE TYPE

- **STANDARD PENETRATION TEST**
  - Drive Sample: 2.42" I.D. sampler (CMS).
  - Bulk Sample: No recovery

### BORING LOG SYMBOLS

- **GROUNDWATER WHILE DRILLING**
- **GROUNDWATER AFTER DRILLING**

### LABORATORY TESTING ABBREVIATIONS

- **STRENGTH**
  - Pocket Penetrometer: p
  - Vane Shear: vs
  - Consolidation: c
  - Collapse Test: col
  - Triaxial Compression: tc
  - Unconfined Compression: uc
  - Direct Shear (single point): ds
  - Resistance (R) Value: r
  - Electrical Resistivity: ce
  - Permeability: perm

- **CONSISTENCY**
  - Very Soft: < 2
  - Soft: 2.4 - 5.6
  - Medium: 5.7 - 9.5
  - Stiff: 9.6 - 16.3
  - Very Stiff: > 16.4

- **APPEARANCE**
  - Very Loose: < 4
  - Loose: 4 - 11
  - Medium: 11 - 30
  - Dense: 31 - 50
  - Very Dense: > 50

### UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

- **SPT (N)**
  - CA Sampler: < 5
  - CA Sampler: < 20

### CONVERSE CONSULTANTS

- Cactus II Feeder
- Eastern Municipal Water District
- Approximately 31,000 Linear Feet of Pipeline and Turnout Facilities
- City of Moreno Valley, Riverside County, California
- For: Black & Veatch

**Project No.: 15-81-158-03**
**Drawing No.: 1**

**Project ID:** 15-81-158-01.GPJ; Template: KEY
### Log of Boring No. MW-1

**Dates Drilled:**  
9/7/2018

**Logged by:**  
William Buckley

**Checked By:**  
Jay Burnham

**Equipment:**  
8" HOLLOW STEM AUGER

**Ground Surface Elevation (ft):**  
1566

**Depth to Water (ft):**  
27

---

**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**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>ALLUVIUM: SILTY SAND (SM): fine to medium-grained, dark brown.</td>
</tr>
<tr>
<td>5-15</td>
<td>SANDY SILT (ML): fine to medium-grained sand, reddish-brown.</td>
</tr>
</tbody>
</table>

**SUMMARY OF WELL INSTALLATIONS**

- Threaded well cap and bolted traffic box
- Bentonite/Cement Grout Seal
- Solid casing within bentonite annular space
- Slotted casing with filter pack
- Cap

---

**Converse Consultants**

Cactus II Feeder  
City of Moreno Valley, Riverside County, California  
For: Black & Veatch

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Project Name: Cactus II Feeder  
Project No.: 15-81-158-03  
Drawing No.: 2
Log of Boring No. MW-2

Dates Drilled: 9/10/2018
Logged by: William Buckley
Checked By: Jay Burnham
Equipment: 8" HOLLOW STEM AUGER
Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 1541
Depth to Water (ft): 23

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.

ALLUVIUM: SILTY SAND (SM): fine to medium-grained, reddish-brown.

SANDY SILT (ML): fine to medium-grained sand, brown.


SUMMARY OF WELL INSTALLATIONS
Threaded well cap and bolted traffic box
Bentonite/Cement Grout Seal
Solid casing within bentonite annular space
Slotted casing with filter pack
Cap

Equipment: 8" HOLLOW STEM AUGER

Converse Consultants
Cactus II Feeder
City of Moreno Valley, Riverside County, California
For: Black & Veatch

Project Name
Project No. 15-81-158-03
Drawing No. 3