APPENDIX 6. - GEOTECHNICAL INVESTIGATION
GEOTECHNICAL INVESTIGATION REPORT

La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
Converse Project No. 14-81-137-01

February 26, 2016

Prepared For:

Albert A. Webb Associates
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Prepared By:

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February 26, 2016

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Subject: GEOTECHNICAL INVESTIGATION REPORT
La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
Converse Project No. 14-81-137-01

Dear Mr. Zhang:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of the Western Municipal Water District (WMWD) La Sierra Pipeline located within the City of Riverside and an unincorporated portion of Riverside County, California. This report was prepared in accordance with our proposal dated April 7, 2014, and your Task Order Agreement # 2014-0216 dated July 7, 2015.

Based upon our field investigation, laboratory data, and analyses, the proposed pipeline 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 Albert A. Webb Associates and WMWD. Should you have any questions, please do not hesitate to contact us at (909) 796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, Ph.D., G. E., P.E.
Principal Engineer

Dist: 4/Addressee
JB/SM/HSQ/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.
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 pipeline for the La Sierra Pipeline project consists of approximately 21,000 linear feet of pipeline, to be installed along the Arlington Channel, Arizona Channel, Line C-1 Channels, and La Sierra Avenue in the City of Riverside and unincorporated Riverside County, California.

- The pipes will be installed using the open cut-and-cover technique with a typical pipe cover of 5 feet below ground surface. The pipe will consist of 24-inch diameter cement mortar lined and coated welded steel pipe (CML/CMC). A total of approximately 350 linear feet of 24-inch pipeline will be installed in 36-inch diameter casing utilizing bore-and-jack trenchless construction methods to cross the Arlington Channel/BNSF Railway, Line C-1 Channel, Riverside Canal, and the Gage Canal. The jacking and receiving pits will be approximately 15 to 20 feet deep.

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

- Twenty-seven exploratory borings (BH-1 through BH-27) were planned to investigate the subsurface conditions along the pipeline alignment. BH-23 could not be drilled due to the proximity to existing underground utilities. The borings were drilled between November 16 and November 18, 2015 at locations approved by Albert A. Webb Associates. The borings were drilled to depths ranging from approximately 6.0 to 21.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations.

- A seismic refraction survey consisting of six seismic 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 within the upper 10 feet along the northern portion of the proposed pipeline alignment predominantly consist of silty sand and sandy silt with localized layers of clayey sand and sandy clay. Scattered gravel was also encountered. Approximately 10 feet of dense fill was encountered along the Arlington Channel.

- Weathered bedrock was encountered at depths of 1 to 7 feet bgs in the portion of the alignment south of McAllister Parkway. Rock hardness increases with depth.
• Difficult excavation and possible blasting is anticipated in a portion of the alignment between Lake Knoll Parkway and the La Sierra Tank, and locally in other areas. The depth to the hard bedrock varies significantly. Large corestones will be encountered.

• Groundwater was not encountered in any of the boring locations, to the maximum explored depth of 21.5 feet bgs. The historic high groundwater level in the vicinity of the alignment is estimated to be approximately 15 feet bgs. Groundwater is not expected to be encountered during the construction of this project. Perched groundwater may be present locally.

• The alignment is not located within a currently designated State of California or Riverside County Earthquake Fault Zone. The alignment is, however, 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 2013 California Building Code are presented in the text of this report.

• The potential impact to the project alignment from surface fault rupture, lateral spreading, and tsunamis is considered to be low.

• The portion of the alignment north of Cleveland Avenue is zoned as susceptible to liquefaction.

• Portions of the alignment south of Orange Lane are adjacent to large slopes, which may be susceptible to landsliding.

• The alignment may be at risk for flooding due to seiching or dam failure at Lake Mathews during a large earthquake.

• The sulfate and chloride contents of the majority of the soils along the proposed alignment correspond to American Concrete Institute (ACI) exposure category S0 and C1, respectively.

• The measured values of the minimum electrical resistivity when saturated ranged from 2,300 to 4,200 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. Such utilities should either be protected in-place, or removed and replaced during construction as required by the project specifications.
• Earthwork for the project 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 1 foot of backfill beneath the pavement sections should be compacted to at least 95 percent of the laboratory maximum dry density.

• 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, and pipeline design parameters 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.
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1.0 INTRODUCTION

This report contains the findings of the geotechnical investigation performed by Converse for approximately 21,000 linear feet of 24-inch diameter water pipeline, to be installed along La Sierra Avenue and the Riverside County Flood Control Arlington Channel, Arizona Channel, and Line C-1 Channel. The alignment is located in the City of Riverside and the adjacent unincorporated portion of Riverside County, California. The approximate location of the proposed pipeline alignment is shown in Figure No. 1, Approximate Alignment 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.

This report was prepared for the project described herein and is intended for use solely by Albert A. Webb Associates 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 SITE DESCRIPTION

The alignment originates at the planned Arlington Desalter Booster Pump Station (BPS) and continues adjacent to the Arlington Channel, Arizona Channel, Line C-1 Channel, along La Sierra Avenue, and terminates at the existing La Sierra Tank tie-in near the intersection of La Sierra Avenue and El Sobrante Road. The portion of the alignment adjacent to the channels is located on unpaved access roads adjacent to the concrete-lined channels. La Sierra Avenue varies from one to three lanes in each direction, and experiences high traffic throughout the day. The proposed alignment has not been finalized, but will is expected to be along the southbound lanes and shoulders.

The portion of the alignment north of Cleveland Avenue is within the City of Riverside. The remainder of the alignment south of Cleveland Avenue is in unincorporated Riverside County.

La Sierra Avenue south of Cleveland Avenue has road cuts into steep rocky hillsides. The road cuts vary in height and are mostly located on the east side of the road. There is one large road cut on the west side of La Sierra Avenue just to the north of the La Sierra Tank access road.
Approximate Alignment Location Map

Project: La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
Location: City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

Project No
14-81-137-01
3.0 PROJECT DESCRIPTION

The La Sierra Pipeline project will consist of the following.

- Approximately 350 linear feet of pipeline from the planned Arlington Desalter BPS to Arlington Channel.
- Approximately 3,000 linear feet of pipeline along Arlington Channel to Arizona Channel.
- Bore-and-jack crossing under Arlington Channel and BNSF Railway to Arizona Channel.
- Approximately 1,800 linear feet of pipeline along the Arizona Channel to the Line C-1 Channel junction.
- Approximately 900 linear feet of pipeline along Line C-1 Channel to La Sierra Avenue.
- Bore-and-jack crossing under Line C-1 Channel.
- Approximately 14,500 linear feet of pipeline along La Sierra Avenue from Line C-1 Channel to the existing La Sierra Tank tie-in.
- Bore-and-jack crossings under Riverside Canal
- Bore-and-jack crossing under Gage Canal.

We understand that the pipeline will be constructed of 24-inch diameter cement mortar lined and coated welded steel pipe (CML/CMC) and that the typical depth to the top of the pipe will be approximately 5 feet below ground surface. The majority of the pipeline will be installed using the open cut-and-cover technique.

We understand that a total of approximately 350 linear feet of 24-inch pipeline will be installed in 36-inch diameter casings utilizing bore-and-jack trenchless construction methods to cross the Arlington Channel/BNSF Railway, Line C-1 Channel, Riverside Canal, and Gage Canal. The jacking and receiving pits will be approximately 15 to 20 feet deep.

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 Albert A. Webb Associates along the pipe alignment.
- Obtained encroachment permits from the City of Riverside, Riverside County, and Riverside County Flood Control and Water Conservation District.
• 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 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

Twenty-seven (27) exploratory borings (BH-1 through BH-25) were planned to investigate the subsurface conditions along the pipeline alignment. BH-23 could not be drilled due to the proximity of existing underground utilities. The borings were drilled between November 16 and November 18, 2015, at locations selected approved by Albert A. Webb Associates. The borings were drilled to depths ranging from approximately 6.0 to 21.5 feet below the existing ground surface (bgs). Where encountered, existing pavement thicknesses were measured at the boring locations.

The approximate locations of the borings are also shown on Figure No. 2a through 2e, 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 and Details

<table>
<thead>
<tr>
<th>Boring Number</th>
<th>Date Drilled</th>
<th>Location</th>
<th>Depth (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>11/17/15</td>
<td>Northwest side of Arlington Channel</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-2</td>
<td>11/17/15</td>
<td>Northwest side of Arlington Channel</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-3</td>
<td>11/17/15</td>
<td>Northwest side of Arlington Channel</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-4</td>
<td>11/17/15</td>
<td>Northwest side of Arizona Channel</td>
<td>21.5</td>
</tr>
<tr>
<td>BH-5</td>
<td>11/17/15</td>
<td>Northeast side of Arizona Channel</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-6</td>
<td>11/18/15</td>
<td>Westbound outside lane of Indiana Avenue southwest of Arizona Channel crossing</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-7</td>
<td>11/17/15</td>
<td>Arizona Channel and Line C-1 Channel junction</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-8</td>
<td>11/16/15</td>
<td>North of Line C-1 Channel crossing</td>
<td>19.5</td>
</tr>
<tr>
<td>BH-9</td>
<td>11/17/15</td>
<td>South of Line C-1 Channel crossing</td>
<td>19.5</td>
</tr>
<tr>
<td>BH-10</td>
<td>11/16/15</td>
<td>Southbound right turn lane on La Sierra Avenue, northwest of Arizona Avenue crossing</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-11</td>
<td>11/16/15</td>
<td>Southbound left turn pocket of La Sierra Avenue, north of Riverside Canal</td>
<td>21.5</td>
</tr>
<tr>
<td>BH-12</td>
<td>11/16/15</td>
<td>Southbound left turn pocket of La Sierra Avenue, south of</td>
<td>21.5</td>
</tr>
</tbody>
</table>
Converse Consultants

Approximate Boring Locations Map

Project: La Sierra Pipeline Project
Location: City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

Project No: 14-81-137-01

Figure No. 2a

EXPLANATION

BH-6 Number and Approximate Location of Soil Boring

500’
Approximate Boring Locations Map

Project: La Sierra Pipeline Project
Location: City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

Project No: 14-81-137-01

Approximate Boring Locations Map

EXPLANATION

BH-14 Number and Approximate Location of Soil Boring

Scale: 500'
Approximate Boring Locations Map

Project: La Sierra Pipeline Project
Location: City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

EXPLANATION

BH-20  Number and Approximate Location of Soil Boring

500'

Converse Consultants
Approximate Boring Locations Map

Project: La Sierra Pipeline Project
Location: La Sierra Avenue Between Sterling Avenue and El Sobrante Road
For: City of Riverside and Unincorporated Riverside County, California

Explaination:
BH-25 Number and Approximate Location of Soil Boring

500'
Approximate Boring Locations Map

EXPLANATION

BH-27  Number and Approximate Location of Soil Boring

Project: La Sierra Pipeline Project
Location: La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

Project No: 14-81-137-01

Figure No. 2e
### 4.2.2 Seismic Refraction Survey

Southwest Geophysics, Inc. was retained to conduct a seismic refraction survey consisting of six seismic lines in areas of suspected hard bedrock. The purpose of the survey was to obtain a 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*.

<table>
<thead>
<tr>
<th>Boring Number</th>
<th>Date Drilled</th>
<th>Location</th>
<th>Depth (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-13</td>
<td>11/16/15</td>
<td>Southbound outside lane of La Sierra Avenue, southeast of Victoria Avenue</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-14</td>
<td>11/16/15</td>
<td>Southbound outside lane of La Sierra Avenue, southeast of Old Fashion Way</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-15</td>
<td>11/18/15</td>
<td>Median of La Sierra Avenue, northwest of McAllister Parkway</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-16</td>
<td>11/17/15</td>
<td>Dirt shoulder west of La Sierra Avenue and north of Gage Canal crossing</td>
<td>15.2</td>
</tr>
<tr>
<td>BH-17</td>
<td>11/17/15</td>
<td>Dirt shoulder west of La Sierra Avenue and south of Gage Canal crossing</td>
<td>15.2</td>
</tr>
<tr>
<td>BH-18</td>
<td>11/17/15</td>
<td>Dirt shoulder west of La Sierra Avenue and south of Orchard View Lane</td>
<td>10.25</td>
</tr>
<tr>
<td>BH-19</td>
<td>11/18/15</td>
<td>Dirt shoulder west of La Sierra Avenue and south of Orchard View Lane</td>
<td>10.1</td>
</tr>
<tr>
<td>BH-20</td>
<td>11/18/15</td>
<td>Southbound outside lane of La Sierra Avenue north of Lake Knoll Parkway</td>
<td>11.0</td>
</tr>
<tr>
<td>BH-21</td>
<td>11/18/15</td>
<td>Southbound outside lane of La Sierra Avenue south of Lake Knoll Parkway</td>
<td>10.25</td>
</tr>
<tr>
<td>BH-22</td>
<td>11/18/15</td>
<td>Southbound shoulder of La Sierra Avenue south of Lake Knoll Parkway</td>
<td>6.0</td>
</tr>
<tr>
<td>BH-23¹</td>
<td>-</td>
<td>Southbound outside lane of La Sierra Avenue south of Lake Crest Drive</td>
<td>-</td>
</tr>
<tr>
<td>BH-24</td>
<td>11/18/15</td>
<td>Southbound outside lane of La Sierra Avenue north of Blackburn Road</td>
<td>6.0</td>
</tr>
<tr>
<td>BH-25</td>
<td>11/18/15</td>
<td>Northbound dirt shoulder of La Sierra Avenue south of Blackburn Road</td>
<td>10.25</td>
</tr>
<tr>
<td>BH-26</td>
<td>11/18/15</td>
<td>Southbound outside lane of La Sierra Avenue north of Blackburn Road</td>
<td>11.5</td>
</tr>
<tr>
<td>BH-27</td>
<td>11/18/15</td>
<td>North side of La Sierra Tank driveway</td>
<td>11.5</td>
</tr>
</tbody>
</table>

¹ Planned boring BH-23 was not drilled due to conflicts with existing underground utility lines.
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:

- In situ moisture contents and dry densities (ASTM Standard D2216)
- Sand equivalent (ASTM D2419)
- Soil corrosivity tests (California Test Methods 643, 422, and 417)
- Swell/Collapse tests (ASTM D5333)
- Grain size analysis (ASTM Standard D422)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

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

4.4 Analysis and Report Preparation

Data obtained from the field exploration and laboratory testing program was 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 pipeline alignment.

5.0 ALIGNMENT CONDITIONS

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

5.1 Subsurface Profile

The thickness of the existing pavement section was measured in borings drilled in paved areas. The thickness of the existing asphalt concrete and aggregate base, where encountered, are summarized in the following table.
Table No. 2, Existing Pavement Thicknesses

<table>
<thead>
<tr>
<th>Boring Number</th>
<th>Asphalt Concrete Thickness (inches)</th>
<th>Aggregate Base Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-6</td>
<td>5.0</td>
<td>8.0</td>
</tr>
<tr>
<td>BH-10</td>
<td>4.0</td>
<td>7.0</td>
</tr>
<tr>
<td>BH-11</td>
<td>9.0</td>
<td>10.0</td>
</tr>
<tr>
<td>BH-12</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>BH-13</td>
<td>9.0</td>
<td>10.0</td>
</tr>
<tr>
<td>BH-14</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>BH-15</td>
<td>6.0</td>
<td>5.0</td>
</tr>
<tr>
<td>BH-19</td>
<td>5.0</td>
<td>7.0</td>
</tr>
<tr>
<td>BH-20</td>
<td>7.0</td>
<td>17.0</td>
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<tr>
<td>BH-21</td>
<td>7.0</td>
<td>16.0</td>
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<tr>
<td>BH-22</td>
<td>4.0</td>
<td>8.0</td>
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<td>BH-24</td>
<td>5.0</td>
<td>12.0</td>
</tr>
<tr>
<td>BH-26</td>
<td>5.0</td>
<td>6.0</td>
</tr>
<tr>
<td>BH-27</td>
<td>5.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

1 Borings not shown were drilled in unpaved locations

5.2 Subsurface Profile

Based on the exploratory borings, the subsurface materials within the upper 10 feet along the proposed pipeline alignment predominantly consist of silty sand and sandy silt with localized layers of clayey sand and sandy clay, and scattered gravel deposits. The thickness of the clayey layers increased at depths greater than 10 feet below ground surface (bgs). Approximately 10 feet of dense fill soils were encountered in boring BH-4, located along the Arlington Channel.

Weathered granitic bedrock was encountered in borings BH-16 through BH-25, located south of McAllister Parkway and north of the La Sierra Tank access road. The bedrock was encountered in the borings at depths ranging from 1 to 7 feet bgs. The degree of weathering decreased with depth, resulting in harder rock. Borings BH-22 and BH-24 encountered auger refusal at a depth of 6 feet bgs, while the other borings were able to penetrate the weathered bedrock to the planned depths of 10 feet or deeper.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-27, Logs of Borings, in Appendix A, Field Exploration.
5.3 **Groundwater**

Groundwater was not encountered during the investigation to the maximum explored depth of 21.5 feet bgs. Groundwater was not encountered to a maximum explored depth of 51.5 feet bgs during a recent investigation performed by Converse for the Arlington Desalter booster pump station and reservoir (Converse, 2015).

Regional groundwater data (SWRCB, 2015) from locations within close proximity to the alignment was reviewed to evaluate the current and historical groundwater levels.

- A site (ID No. T0606599150) located at the intersection of Magnolia Avenue and Pierce Street reported groundwater depths ranging from 29 feet to 55 feet bgs between 2001 and 2008.
- A site (ID No. T0606500171), also located at the intersection of Magnolia Avenue and Pierce Street reported groundwater at depths ranging from 16 feet to 19 feet bgs in 1990.
- A site (ID No. T0606599277) located approximately 600 feet west of the intersection of La Sierra Avenue and McAllister Parkway reported groundwater at a depth of approximately 33 feet bgs in 2012.
- A site (ID No. T0606500209) located approximately 1000 feet south of the intersection of La Sierra Avenue and El Sobrante Road reported groundwater at a depth of 28 feet bgs in 1990.

Based on the data reviewed, the historical high groundwater level in the vicinity of the alignment is estimated to be approximately 15 feet bgs and the current depth is greater than 50 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 up to 20 feet bgs and pipe trench to depths of up to approximately 10 feet bgs. Bedrock is not anticipated in the excavations north of McAllister Parkway. We anticipate that the sediments underlying this area will be readily excavatable with conventional trenching equipment, including excavators and trenching machines.

The pipeline excavations along La Sierra Avenue south of McAllister Parkway, which are anticipated to be up to 10 feet in depth, will encounter granitic bedrock of varying degrees of weathering at depths of 1 to 10 feet bgs.
The hollow-stem auger drill rig was able to penetrate the bedrock to 10 feet bgs or deeper in most areas. Refusal was encountered in borings BH-22 and BH-24 at approximately 6 feet bgs.

Southwest Geophysics, Inc. was retained to perform a seismic refraction survey for the purpose of evaluating bedrock rippability. Six seismic refraction traverses were conducted at selected locations between Lake Knoll Parkway and the La Sierra Tank access road. 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 the alignment, sometimes over short distances. In general, the bedrock is expected to be excavatable to depths of 10 feet or deeper with moderate difficulty. The difficulty of excavation will increase with depth as the degree of rock weathering decreases.

Areas of difficult excavation or blasting will be encountered within the segment of the alignment between borings BH-21 and BH-25, between Lake Knoll Parkway and Blackburn Road, and locally in other portions of the alignment south of McAllister Parkway. 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 LABORATORY TEST RESULTS

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the subsurface soils. Tests results are
Discussions of the various test results are presented below:

### 6.1 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 Standard D2216. Dry densities of soils along the proposed pipeline alignment ranged from 83 to 145 pcf with moisture contents of 2 to 28 percent. Results are presented in the log of borings in Appendix A, *Field Exploration*.

- 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 tested ranged from 13 to 51.

- Collapse Potential – The collapse potential of six 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 D5333 test method. The results showed a collapse of 0.2 to 2.9 percent, indicating a slight to moderate collapse potential.

- Grain Size Analysis – Six representative samples were tested to determine their relative grain size distributions in accordance with the ASTM Standard D422. Test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results*.

- Maximum Dry Density and Optimum Moisture Content – Results of four typical moisture-density relationships of representative soil samples tested, according to ASTM Standard D1557, are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program*. The laboratory maximum dry densities ranged from 127.5 to 135.0 pounds per cubic feet (pcf), with optimum moisture contents between 7.5 and 9.5 percent.

- Direct Shear – Six 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-8, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

### 6.2 Chemical Testing - Corrosivity Evaluation

Three 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 EG Labs 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 7.2 to 7.9.
• The sulfate contents of the samples tested ranged from 0.003 to 0.032 percent by weight.
• The chloride concentrations of the samples tested ranged from 180 to 280 ppm.
• The minimum electrical resistivities when saturated ranged from 2,300 to 4,200 Ohm-cm.

7.0 GEOLOGIC SETTING

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

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

7.2 Local Geology

Regional mapping (Morton and Miller, 2006) indicates that the subsurface along the alignment north of the vicinity of Cleveland Avenue primarily underlain by Pleistocene older alluvial fan deposits. The older alluvium generally consists of moderately consolidated silt and sand with local gravelly layers. The older alluvium is locally overlain by several feet of unconsolidated Holocene alluvium.
Granitic bedrock is exposed at the ground surface along the alignment to the southwest of Cleveland Avenue. The bedrock is Cretaceous in age and consists of granodiorite and gabbro. This portion of the alignment is characterized by hills and associated valleys and drainages.

8.0 FAULTING AND SEISMICITY

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

8.1 Faulting

The site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2015). The nearest active fault is the Chino-Central Avenue (Elsinore) fault, located approximately 8 miles from the project site.

8.2 CBC Seismic Design Parameters

Seismic parameters based on the California Building Code (CBSC, 2013) do not vary significantly between different portions of the alignment. The parameters provided in the following table were determined using the Seismic Design Maps application (USGS, 2015b) and are based on the approximate midpoint of the alignment.

<table>
<thead>
<tr>
<th>Table No. 3, CBC 2013 Seismic Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>Coordinates</td>
</tr>
<tr>
<td>Site Class</td>
</tr>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_s$</td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(1)), $F_a$</td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(2)), $F_v$</td>
</tr>
<tr>
<td>MCE 0.2-sec period Spectral Response Acceleration, $S_{Ms}$</td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{M1}$</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for short period $S_{gs}$</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for 1-second period, $S_{d1}$</td>
</tr>
<tr>
<td>Maximum Peak Ground Acceleration, $PGA_M$</td>
</tr>
</tbody>
</table>
8.3 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, 2007; Riverside County, 2015). 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 cohesion-less soil mass 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 generally occurs in submerged granular soils and non-plastic silts located within 50 feet of the ground surface during or after strong ground shaking. There are several general requirements for liquefaction to occur. They are as follows:

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

The portion of the alignment northwest of the intersection of La Sierra Avenue and Liverpool Lane are in a zone designated as being moderately susceptible to liquefaction by Riverside County. The alignment along La Sierra Avenue from approximately Liverpool Lane to Cleveland Avenue is in a zone designated as being highly susceptible to liquefaction by Riverside County.

Liquefaction may occur in the portion of the alignment north of Cleveland Avenue during a strong seismic event with high groundwater conditions. The portion of the alignment south of Cleveland Avenue is not considered to be susceptible to liquefaction due to the presence of shallow bedrock.

Seismic Settlement: Seismically-induced settlement occurs during ground shaking associated with earthquakes. A quantitative analysis of dry seismic settlement potential was not performed during this investigation. Based on the presence of relatively fine grained and medium to high density soils along the proposed alignment, and the
presence of shallow bedrock south of Cleveland Avenue, we anticipate that the potential for dry seismic settlement is relatively low along the alignment.

**Landslides:** Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The slopes along the channels are concrete-lined and are not considered at risk for landsliding. There are numerous ascending and descending slopes adjacent to the alignment south of Orange Lane. These slopes have not been individually evaluated for stability and may be susceptible to landsliding during a major seismic event.

**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 channels are concrete-lined and are not considered at risk for lateral spreading. The slopes adjacent to the alignment south of Orange Lane are in an area considered not at risk for lateral spreading due to shallow bedrock. 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. Portions of the alignment are adjacent to flood control channels. There is a potential for waves to overtop the sides of the channels during a seismic event. The southern end of the alignment is located approximately 1,700 feet from the base of the Lake Matthews dam. There is a potential for waves to overtop the Lake Mathews dam during a seismic event.

**Earthquake-Induced Flooding:** Dams or other water-retaining structures may fail as a result of large earthquakes, resulting in flooding. The south end of the alignment is located approximately 1,700 feet from the base of the Lake Mathews dam. In the event of a breach, there is a potential for water to drain along La Sierra Avenue resulting in flooding.

### 9.0 EARTHWORK RECOMMENDATIONS

Recommendations for earthwork associated with pipe trenching are presented in the following subsections.
9.1 **General**

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

Deleterious material, including organics, asphalt, and debris generated during excavation should not be placed as backfill.

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

9.2 **Pipeline Subgrade Preparation**

The final subgrade surface should be level, firm, uniform, 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.

9.3 **Pipe Bedding**

Bedding is defined as the material supporting and surrounding the pipe, to 12 inches above the pipe. 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. Care should be taken to densify the bedding material below the springline of the pipe.

Pipe design generally requires a granular material with a sand equivalent (SE) greater than 30. Since two of the four tested soil samples were below 30, some excavated soils may not be suitable for use as pipe bedding.

For a nominal pipe size of 24 inches, crushed rock used as bedding should have a maximum size of no larger than ¾ inches.
Migration of fines from the surrounding soils must be considered in selecting the gradation of any imported bedding material. To avoid migration of fines, commercially available geofabric used for filtration purposes (such as Mirafi 140N or equivalent) may be wrapped around the bedding material encasing the pipe to separate the bedding material from the surrounding native or fill soils.

9.4 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Detailed trench backfill recommendations are provided 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 1 foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.
- Particles larger than 1 inch should not be placed within 12 inches of the pavement subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than 3 inches in the largest dimension should not be placed as trench backfill.
- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse grained soil, and between optimum and 2 percent above optimum for fine grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- The field density of the compacted soil should be measured by the ASTM 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.
9.5 **Backfill of Jacking and Receiving Pits**

We anticipate that the depths of the jacking and receiving pits will be approximately 15 to 20 feet below the existing grade. The pits should be backfilled following construction of the pipe crossings.

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.

9.6 **Select Imported Fill Materials**

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria.

- Expansion Index less than 20
- Free of all deleterious materials
- Contain no particles larger than 3 inches in the largest dimension
- Contain less than 30 percent by weight retained on ¾-inch sieve
- Contain at least 15 percent fines (passing #200 sieve)
- Plasticity Index of 10 or less

Any import fill should be tested and approved by the owner’s representative prior to delivery to the site.
10.0 DESIGN RECOMMENDATIONS

General design recommendations, lateral and passive earth pressures, pipe design parameters, bearing pressures, and soil corrosivity is 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.

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

10.2 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>Table No. 4, Lateral Earth Pressures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crossing</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Arlington Channel / BNSF Railway</td>
</tr>
<tr>
<td>Line C-1 Channel</td>
</tr>
<tr>
<td>Riverside Canal</td>
</tr>
<tr>
<td>Gage Canal</td>
</tr>
</tbody>
</table>

¹ Assumes level ground conditions
² Per foot of depth.
Resistance to lateral loads can be assumed to be provided by friction acting at the base of thrust blocks and by passive earth pressure. The passive earth pressures provided in the table above be used for resistance against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 1,500 psf for native soils.

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.

10.3 Soil Parameters for Pipe and Structure 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 recommended values of the various soil parameters for design are provided in the following table.

<table>
<thead>
<tr>
<th>Table No. 5, Soil Parameters for Pipe and Structure Design</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Soil Parameters</strong></td>
</tr>
<tr>
<td>Average compacted fill total unit weight, $\gamma$</td>
</tr>
<tr>
<td>Angle of internal friction of soils, $\phi$</td>
</tr>
<tr>
<td>Soil cohesion, $c$</td>
</tr>
<tr>
<td>Coefficient of friction between formed concrete and native soils, $f_s$</td>
</tr>
<tr>
<td>Coefficient of friction between CML/CMC(^1) pipe and native soils, $f_s$</td>
</tr>
<tr>
<td>Bearing pressure against native soils</td>
</tr>
<tr>
<td>Coefficient of passive earth pressure, $K_p$</td>
</tr>
<tr>
<td>Coefficient of active earth pressure, $K_a$</td>
</tr>
<tr>
<td>Modulus of Soil Reaction $E'$ (psi)</td>
</tr>
</tbody>
</table>

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

10.4 Bearing Pressure for Anchor and Thrust Blocks

An allowable net bearing pressure of 2,500 psf may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 24 inches wide.
Resistance to lateral forces can be assumed to be provided by friction at the base of thrust blocks and by passive earth pressure. An ultimate value of coefficient of friction of 0.35 may be used between the thrust block and the supporting natural soil or compacted fill. A passive earth pressure of 250 psf per foot of depth may be used for the sides of thrust blocks or anchors poured against undisturbed or recompacted soils. The value of the passive lateral earth pressure should be limited to 1,500 psf. Frictional and passive resistance can be combined for the design of anchors and thrust blocks.

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 Soil Corrosivity

The results of chemical testing of representative samples of site 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-2, Summary of Corrosivity Test Results, and are discussed below.

The sulfate contents of the majority of the soils along the proposed alignment 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 will be exposed to moisture from precipitation and irrigation. Based on the alignment location 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 318-11, 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 2,300 to 4,200 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 (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures for ferrous metals in contact with the site soils.
11.0 CONSTRUCTION RECOMMENDATIONS

11.1 General

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

11.2 Temporary Sloped Excavations

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

<table>
<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>Clay, Silt, Clayey Sand, Silty</td>
<td>0-4</td>
<td>Vertical</td>
</tr>
<tr>
<td>Sand</td>
<td>4-10</td>
<td>1:1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>0-4</td>
<td>1.5:1</td>
</tr>
<tr>
<td></td>
<td>4-10</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.

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.

Restrained (braced) shoring systems should be designed to support a uniform rectangular lateral earth pressure of 30 psf, based on Figure No. 3, Recommended Lateral Earth Pressure for 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. 4, Recommended Lateral Earth Pressures on Cantilever Wall.

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).

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 lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier
TEMPORARY BRACED EXCAVATION
LATERAL EARTH PRESSURE

\[ P = P_q + P_a \]
\[ = 0.5q + 37H_1 \]  
- active earth pressure (Cantilever walls)

\[ P_p = 250H_2 \leq 1500 \text{ psf} \]  
- passive earth pressure (on native compacted soils)

\[ \mu = 0.35 \]  
- ultimate friction coefficient between pile and soil

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

2. \( P_p \) and \( P_a \) are the passive and active earth pressure respectively; \( P_q \) is the incremental surcharge earth pressure; and \( \mu \) is allowable friction coefficient, applied to dead normal loads acting on non-pile supported elements.

3. 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.

4. \( P_p \) includes a safety factor of 1.5.

5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement or slab.

6. For traffic surcharge, use a uniform pressure of 100 psf over the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURE FOR BRACED EXCAVATION
PERMANENT RETAINING WALLS

\[ P = P_q + P_a \]
\[ = 0.5q + 30H_1 \]  - active earth pressure (Cantilever walls)
\[ = 0.5q + 47H_1 \]  - at rest earth pressure (Restrained walls)
\[ P_p = 250 \, H_2 \leq 1500 \, \text{psf} \]  - passive earth pressure (on native compacted soils)

\[ \mu = 0.35 \]  - ultimate friction coefficient between backfill and native soils
\[ \mu = 0.25 \]  - ultimate friction coefficient between pipe and native soils

Notes:
1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp, Pa, and Po are the passive, active, and at-rest earth pressures, respectively; Pe is the incremental seismic earth pressure; Pq is the incremental surcharge earth pressure, and \( \mu \) is the allowable friction coefficient, applied to dead normal loads acting on non-pile supported elements.
3. For retained walls (not free to rotate), use at-rest (Po) earth pressure; increase Pe by 30 percent.
4. Base friction coefficient (\( \mu \)) and Pp include a safety factor of 1.5.
5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement or slab.
6. Surcharge load only applies to the upper 10 feet.
7. Drainage system should be provided for the retaining wall.
8. For traffic surcharge, assume a 100-psf uniform pressure along the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURES ON CANTILEVER WALL

Project: La Sierra Pipeline Project
Location: La Sierra Avenue Between Sterling Avenue and El Sobrante Road
For: City of Riverside and Unincorporated Riverside County, California

Albert A. Webb Associates

Converse Consultants
piles. Accordingly, the lagging between the piles may be designed based on the following guidelines:

- Lagging design load = 0.6 of shoring design load
- Maximum lagging load may be 250 psf without surcharges

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.

11.4 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>Loess 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>Raveling</td>
<td>Chunks or flakes of material begin to drop out of the arch or walls sometime</td>
<td>Residual soils or sand with small amounts of binder may be fast raveling below the water tale, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.</td>
</tr>
<tr>
<td></td>
<td>after the ground has been exposed, due to loosening or to over-stress and</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&quot;brittle&quot; fracture (ground separates or breaks along distinct surfaces, opposed</td>
<td></td>
</tr>
<tr>
<td></td>
<td>to squeezing ground). In fast raveling ground, the process starts within a few</td>
<td></td>
</tr>
<tr>
<td></td>
<td>minutes, otherwise the ground is slow raveling.</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>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 plasticity 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>

The results of our subsurface exploration indicate that medium dense to dense sandy to silty soil conditions will likely be encountered. 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 and running sand 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. 8, *Site Specific Ground Classifications* on the following page.
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 Riverside, Riverside County, and other local agencies, if applicable.

11.5 Trenchless Pipe Crossing Construction

Pipe jacking and micro-tunneling operations involve the initial construction of a jacking/tunneling pit and a receiving pit at each end of the pipe segment to be jacked. Micro-tunneling can be regarded as an extension of pipe jacking where a new pipe is pushed through a hole excavated ahead of the advancing pipe string. Whereas traditional pipe jacking requires a team of workers at the face, micro-tunneling replaces this manual work with a small tunnel boring machine (TBM).

Table No. 8, Site Specific Ground Classifications

<table>
<thead>
<tr>
<th>Crossing Location</th>
<th>Boring No.</th>
<th>Approximate Depth To Invert (Feet)</th>
<th>Soil Types</th>
<th>Hard</th>
<th>Firm</th>
<th>Raveling/ Running</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arlington Channel/BNSF Railway</td>
<td>BH-4</td>
<td>20</td>
<td>SM</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Arlington Channel/BNSF Railway</td>
<td>BH-5</td>
<td>20</td>
<td>SC</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Line C-1 Channel</td>
<td>BH-8</td>
<td>18</td>
<td>ML</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Line C-1 Channel</td>
<td>BH-9</td>
<td>18</td>
<td>SM</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Riverside Canal</td>
<td>BH-11</td>
<td>20</td>
<td>ML</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Riverside Canal</td>
<td>BH-12</td>
<td>20</td>
<td>SM</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Gage Canal</td>
<td>BH-16</td>
<td>15</td>
<td>Bedrock/SM</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Gage Canal</td>
<td>BH-17</td>
<td>15</td>
<td>Bedrock/SM</td>
<td></td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

1 Severely weathered bedrock below approximately 7 feet bgs excavates as silty sand.

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 11.3, Shoring Design. Frequent contact grouting may be necessary to backpack the support during construction to minimize settlement.

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 2,000 psf 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.

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. Grouting through the pipe casing after jacking is recommended to fill any possible voids created by the jacking operation. Jacking operations and tunneling operations should be performed in accordance with the Standard Specifications for Public Works Construction, Sections 306-2 and 306-3 (Public Works Standards, 2015).

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 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.

12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should be present to observe conditions and test the density and moisture of the backfill during pipeline installation. The excavations and backfill should be observed and tested for compliance with project specifications.

13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Albert A. Webb Associates 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.

Our conclusions and recommendations are based on the results of the field investigations and laboratory tests, combined with interpolation and extrapolation of soil conditions between and beyond the boring locations. 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 Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others.
14.0 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 STATE WATER RESOURCES CONTROL BOARD (SWRCB), 2015, GeoTracker database (http://geotracker.waterboards.ca.gov/), accessed on December 15, 2015.


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 Albert A. Webb Associates. 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.

Twenty-seven exploratory borings (BH-1 through BH-27) were drilled between November 16 and November 18, 2015. Boring BH-23 was not drilled due to presence of underground utilities in the vicinity of the planned excavation. The borings were drilled to depths ranging from 6.0 to 21.5 feet below existing ground surface (bgs).

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for 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. Bulk samples of typical soil types were also obtained.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between 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 loosely backfilled with soil cuttings and lightly tamped. Where the borings penetrated pavement, the surface was patched with asphalt concrete. As a result, the surface may settle over time. If construction is delayed, we recommend the owner monitor the boring locations and backfill any depressions that might occur, or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.
For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. Logs of the exploratory borings are presented in Drawings No. A-2 through A-27, *Logs of Borings*.


**Log of Boring No. BH-1**

**Dates Drilled:** 11/17/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 716  
**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 medium-grained, red brown.

**Sandy Silt (ML):** fine to medium-grained, trace clay, red brown.

Boring adjacent to Arlington Channel  
End of boring at 11.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.  
Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.

---

**SAMPLES**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Drive</th>
<th>Bulk</th>
<th>Blows</th>
<th>Moisture</th>
<th>Dry Unit Wt (pcf)</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>4/4/4</td>
<td>6</td>
<td>116</td>
<td></td>
<td></td>
<td>se, ca, er, ma, max</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>4/6/9</td>
<td>15</td>
<td>103</td>
<td></td>
<td></td>
<td>ds</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5/4/8</td>
<td>10</td>
<td>113</td>
<td></td>
<td></td>
<td>col</td>
</tr>
</tbody>
</table>

---

La Sierra Pipeline Project  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates
Log of Boring No. BH-2

Dates Drilled: 11/17/2015
Logged by: Jay Burnham
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 721
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, red brown.
- few gravel
- trace clay

#### CLAYEY SAND (SC):

fine to medium-grained, red brown.

Boring adjacent to Arlington Channel
End of boring at 11.5 feet bgs.
Groundwater not encountered.
Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.
Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>DAMO</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>DRY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>3/4/5</td>
<td>9</td>
<td>111</td>
<td>ma</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>5/11/14</td>
<td>9</td>
<td>125</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8/8/12</td>
<td></td>
<td>10</td>
<td>120</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

This log is part of the report prepared by Converse for this project and should be read together with the report. The data presented is a simplification of actual conditions encountered with the passage of time.
### Log of Boring No. BH-3

**Dates Drilled:** 11/17/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 732  
**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></td>
<td></td>
</tr>
</tbody>
</table>

#### ALLUVIUM:

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

- red brown

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>DRIVER</th>
<th>BULKS</th>
<th>MOISTURE</th>
<th>DRAY UNIT WT.</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/10/10</td>
<td>8</td>
<td>117</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/10/11</td>
<td>9</td>
<td>122</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5/8/13</td>
<td>9</td>
<td>135</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Boring adjacent to Arlington Channel  
End of boring at 11.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.  
Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.
# Log of Boring No. BH-4

**Dates Drilled:** 11/17/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in

**Ground Surface Elevation (ft):** 733  
**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>SUMMARY OF SUBSURFACE CONDITIONS</th>
<th>SAMPLES</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>FILL:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>SILTY SAND (SM):</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>fine to coarse-grained, trace clay, olive brown.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>- 3&quot; concrete fragment in shoe</td>
<td>17/19/22</td>
<td>5</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
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<td><strong>SILTY SAND (SM):</strong></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>fine to coarse-grained, trace clay, olive brown.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>15/22/27</td>
<td>9</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>Boring adjacent to Arlington Channel</td>
<td>7/12/10</td>
<td>6</td>
</tr>
<tr>
<td></td>
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<td>11/19/24</td>
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</tr>
<tr>
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<td></td>
<td>Groundwater not encountered.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Project No.:** 14-81-137-01  
**Drawing No.:** A-5

La Sierra Pipeline Project  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates
### Log of Boring No. BH-5

**Dates Drilled:** 11/17/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  

**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 741  
**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:</th>
<th>CLAYEY SAND (SC):</th>
<th>SAND (SP):</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
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<td>SILTY SAND (SM): fine to coarse-grained, olive brown.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>- trace clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>CLAYEY SAND (SC): fine to coarse-grained, olive brown.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>SAND (SP): fine to coarse-grained, gray brown.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Boring adjacent to Arizona Channel  
End of boring at 11.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.  
Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
</tr>
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<tbody>
<tr>
<td>5</td>
<td>5/4/6 7 114</td>
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<tr>
<td>15</td>
<td>8/11/15 14 121 ca, er, ma</td>
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<tr>
<td>20</td>
<td>8/8/12 1 disturbed</td>
</tr>
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</table>

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**Project ID:** 14-81-137-01.GPJ  
**Template:** LOG  
**La Sierra Pipeline Project**  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates
# 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.

## 5.0” Asphalt Concrete / 8.0” Aggregate Base

### Alluvium:

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

### Silty Sand with Clay (SM):

- Fine to medium-grained, olive brown.

Boring adjacent to Arizona and Line C-1 Channels
End of boring at 11.5 feet bgs.
Groundwater not encountered.
Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.
Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.

## Table of Data

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Summary of Subsurface Conditions</th>
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<td>Alluvium:</td>
</tr>
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<table>
<thead>
<tr>
<th>SAMPLES</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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<tr>
<td>5/6/8</td>
<td>12</td>
<td>111</td>
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<td>6/6/10</td>
<td>10</td>
<td>120</td>
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</tbody>
</table>

---

**Equipment:** 8” Hollow Stem Auger

**Dates Drilled:** 11/18/2015

**Logged by:** Jay Burnham

**Checked By:** Scot Mathis

---

**La Sierra Pipeline Project**

**La Sierra Avenue Between Sterling Avenue and El Sobrante Road**

**City of Riverside and Unincorporated Riverside County, California**

**For:** Albert A. Webb Associates

**Project No.:** 14-81-137-01

**Drawing No.:** A-7
### Log of Boring No. BH-7

**Dates Drilled:** 11/17/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis

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

**Ground Surface Elevation (ft):** 762  
**Depth to Water (ft):** NOT ENCOUNTERED

#### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
<thead>
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<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
<th>SAMPLES</th>
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<tbody>
<tr>
<td>0-5</td>
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<td>ALLUVIUM: SILTY SAND (SM): fine to medium-grained, red brown.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- fine to coarse-grained</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- trace clay</td>
<td></td>
</tr>
<tr>
<td>5-11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-16</td>
<td></td>
<td>Boring adjacent to Line C-1 Channel</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>End of boring at 11.5 feet bgs.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Groundwater not encountered.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/17/2015.</td>
<td></td>
</tr>
</tbody>
</table>

Converse Consultants

La Sierra Pipeline Project  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates

Project No.: 14-81-137-01  
Drawing No.: A-8
**Log of Boring No. BH-8**

**Dates Drilled:** 11/16/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in

**Ground Surface Elevation (ft):** 771  
**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: SILTY SAND (SM): fine to coarse-grained, olive brown.</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
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</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- trace clay

---

**Boring adjacent to Line C-1 Channel**

- End of boring at 11.5 feet bgs.
- Groundwater not encountered.
- Borehole backfilled from the bottom to 5 feet bgs with bentonite slurry seal.
- Borehole backfilled from 5 feet bgs to the ground surface with soil cuttings and tamped on 11/16/2015.

---

**Samples**

<table>
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<tr>
<th>Depth (ft)</th>
<th>Drill</th>
<th>Blows</th>
<th>Moisture</th>
<th>Unit Weight (pcf)</th>
<th>Other</th>
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<td>5/5/8</td>
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</tr>
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<td>10</td>
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<td>8/19/40</td>
<td></td>
<td>7</td>
<td>114</td>
</tr>
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<td>15</td>
<td></td>
<td>8/20/11</td>
<td></td>
<td>11</td>
<td>100</td>
</tr>
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<td>15.5</td>
<td></td>
<td>9/16/27</td>
<td></td>
<td>10</td>
<td>ds ma</td>
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**La Sierra Pipeline Project**

La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates

---

**Converse Consultants**

Project No. 14-81-137-01  
Drawing No. A-9
### 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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<tr>
<td>10</td>
<td></td>
</tr>
<tr>
<td>15</td>
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</tr>
</tbody>
</table>

**Alluvium:**

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

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

**Clayey Sand (SC):** fine to medium-grained, olive brown.

Boring adjacent to Line C-1 Channel.
End of boring at 19.5 feet bgs.
Groundwater not encountered.
Borehole backfilled loose with soil cuttings and lightly tamped on 11/17/2015.

### Samples

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Drive</th>
<th>Blows</th>
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<th>Unit Wt. (pcf)</th>
<th>Other</th>
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<tr>
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<td>11</td>
<td>109</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8/11/16</td>
<td>8/11/16</td>
<td>10</td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11/14/21</td>
<td>11/14/21</td>
<td>8</td>
<td>114</td>
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<td>16/26/40</td>
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</table>
**Summary of Subsurface Conditions**

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### 4.0" Asphalt Concrete / 7.0" Aggregate Base

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

---

**End of boring at 11.5 feet bgs.**
**Groundwater not encountered.**
**Borehole backfilled loose with soil cuttings and lightly tamped on 11/16/2015.**
Log of Boring No.  BH-11

Dates Drilled: 11/16/2015
Logged by: Jay Burnham
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 805
Depth to Water (ft): NOT ENCOUNTERED

9" ASPHALT CONCRETE / 10" AGGREGATE BASE

<table>
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<th>SUMMARY OF SUBSURFACE CONDITIONS</th>
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</thead>
<tbody>
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<td>15</td>
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<tr>
<td>20</td>
<td></td>
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<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ALLUVIUM:
SANDY SILT (ML): fine to medium-grained, trace clay, olive brown.
- fine to coarse-grained

End of boring at 21.5 feet bgs.
Groundwater not encountered.
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/16/2015.
La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and
El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

### Log of Boring No. BH-12

**Dates Drilled:** 11/16/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 806  
**Depth to Water (ft):** NOT ENCOUNTERED

#### SUMMARY OF SUBSURFACE CONDITIONS

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<table>
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<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>9&quot; ASPHALT CONCRETE / 9&quot; AGGREGATE BASE</th>
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</thead>
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<tr>
<td></td>
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<td><strong>ALLUVIUM:</strong></td>
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<tr>
<td></td>
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<td><strong>SILTY SAND (SM):</strong> fine to medium-grained, olive brown.</td>
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<tr>
<td></td>
<td></td>
<td>- trace clay, red brown</td>
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<tr>
<td>5</td>
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<td></td>
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<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
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<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>CLAYEY SAND (SC):</strong> fine-grained, red brown.</td>
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</tbody>
</table>

End of boring at 21.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/16/2015.
# Log of Boring No. BH-13

**Dates Drilled:** 11/16/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 825  
**Depth to Water (ft):** NOT ENCOUNTERED

---

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th><strong>SUMMARY OF SUBSURFACE CONDITIONS</strong></th>
<th>SAMPLES</th>
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<tr>
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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>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>9&quot; ASPHALT CONCRETE / 10&quot; AGGREGATE BASE</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>SILTY SAND (SM):</strong> fine to coarse-grained, dark brown.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>SAND (SP):</strong> fine to coarse-grained, dark brown.</td>
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</tr>
<tr>
<td></td>
<td></td>
<td><strong>SILTY SAND (SM):</strong> fine to coarse-grained, dark brown.</td>
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<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4/4/6</td>
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End of boring at 11.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/16/2015.
### Log of Boring No. BH-14

**Dates Drilled:** 11/16/2015

**Logged by:** Jay Burnham

**Checked By:** Scot Mathis

**Equipment:** 8" HOLLOW STEM AUGER

**Driving Weight and Drop:** 140 lbs / 30 in

**Ground Surface Elevation (ft):** 852

**Depth to Water (ft):** NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
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</tr>
</thead>
<tbody>
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</tr>
<tr>
<td></td>
<td></td>
<td><strong>SILTY SAND (SM):</strong> fine to medium-grained, olive brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>SANDY CLAY (CL):</strong> fine-grained sand, dark brown.</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>End of boring at 11.5 feet bgs. Groundwater not encountered. Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/16/2015.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRIVE</td>
</tr>
<tr>
<td>BULK</td>
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<tr>
<td>BLOWS</td>
</tr>
<tr>
<td>MOISTURE</td>
</tr>
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<td>DRY UNIT WT.</td>
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<td>OTHER</td>
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<td>4/4/7 4 113</td>
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<tr>
<td>5/5/6 5 107</td>
</tr>
<tr>
<td>12/16/26 28 90 col</td>
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</tbody>
</table>

**La Sierra Pipeline Project**
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

**Project No.:** 14-81-137-01
**Drawing No.:** A-15
**Log of Boring No. BH-15**

**Dates Drilled:** 11/18/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 900  
**Depth to Water (ft):** NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
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<tbody>
<tr>
<td>5</td>
<td></td>
</tr>
<tr>
<td>10</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>Depth (ft)</th>
<th>GRADES</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>8</td>
<td>8&quot; ASPHALT CONCRETE / 8&quot; AGGREGATE BASE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alluvium:</td>
<td>SILTY SAND (SM): fine to medium-grained, dark gray brown.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>- fine to coarse-grained</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>CLAYEY SAND (SC): fine to medium-grained, red brown.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 11.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/18/2015.
## Log of Boring No. BH-16

**Dates Drilled:** 11/17/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8” HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 982  
**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, red brown.</th>
<th>Bedrock: Mixed Granodiorite and Gabbro severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, yellow brown.</th>
<th>Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>- fine to coarse-grained</td>
<td></td>
<td>5/12/14</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td>50-3&quot;</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>End of boring at 15.2 feet bgs. Groundwater not encountered.</td>
<td>Borehole backfilled with soil cuttings and lightly tamped on 11/17/2015.</td>
<td>50-2&quot;</td>
</tr>
</tbody>
</table>

---

La Sierra Pipeline Project  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates

---

Converse Consultants  
Project No.: 14-81-137-01  
Drawing No.: A-17

---

Project ID: 14-81-137-01.GPJ; Template: LOG
## SUMMARY OF SUBSURFACE CONDITIONS

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### ALLUVIUM:

**SILTY SAND (SM):** fine to coarse-grained, red brown.

### GRAVELLY SAND with SILT (SP):

fine to coarse-grained, red brown.

### BEDROCK

**MIXED GRANODIORITE AND GABBRO** severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, yellow brown.


---

**La Sierra Pipeline Project**  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates
Log of Boring No. BH-18

Dates Drilled: 11/17/2015  Logged by: Jay Burnham  Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1027  Depth to Water (ft): NOT ENCLOSED

### SUMMARY OF SUBSURFACE CONDITIONS

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**ALLUVIUM:**

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

Bedrock:

Mixed Granodiorite and Gabbro severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, olive gray

End of boring at 10.25 feet bgs.

Groundwater not encountered.

Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/17/2015.

---

SAMPLES

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>GRAPHIC LOG</th>
<th>DRILL LOG</th>
<th>BULK</th>
<th>MOISTURE</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>50-3&quot;</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td>50-3&quot;</td>
<td>disturbed</td>
</tr>
<tr>
<td>10.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>no recovery</td>
</tr>
</tbody>
</table>

---

End of boring at 10.25 feet bgs.

Groundwater not encountered.

Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/17/2015.
### Log of Boring No. BH-19

**Dates Drilled:** 11/18/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  
**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1040  
**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>11</td>
<td></td>
<td><strong>5.0&quot; ASPHALT CONCRETE / 7.0&quot; AGGREGATE BASE</strong></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td><strong>BEDROCK</strong></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td><strong>MIXED GRANODIORITE AND GABBRO</strong></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>severely weathered. Excavates as Sand (SP): fine to coarse-grained, olive yellow.</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.
### Log of Boring No. BH-20

**Dates Drilled:** 11/18/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis  

**Equipment:** 8" HOLLOW STEM AUGER  
**Driving Weight and Drop:** 140 lbs / 30 in  
**Ground Surface Elevation (ft):** 1037  
**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>7.0&quot; ASPHALT CONCRETE / 17.0&quot; AGGREGATE BASE</th>
<th>SAMPLES</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
<td>DRIVE</td>
<td>BULK</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to coarse-grained, olive brown.</td>
<td>10/20/25</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- red brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>BEDROCK:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Mixed Granodiorite and Gabbro</strong> severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, gray brown.</td>
<td>40/50-3&quot;</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>44/50-5&quot;</td>
<td>4</td>
</tr>
</tbody>
</table>

---

End of boring at 11.0 feet bgs.  
Groundwater not encountered.  
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/18/2015.
### 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.

#### 7.0" ASPHALT CONCRETE / 16" AGGREGATE BASE

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Alluvium:</th>
<th>Bedrock:</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td></td>
<td>7/9/17</td>
<td>30/50-6(^*)</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>7</td>
<td>3</td>
</tr>
<tr>
<td>10.25</td>
<td></td>
<td>122</td>
<td>126</td>
</tr>
</tbody>
</table>

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

**BEDROCK:**
Mixed Granodiorite and Gabbro severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, olive gray.

End of boring at 10.25 feet bgs.
Groundwater not encountered.
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/18/2015.
Log of Boring No. BH-22

Dates Drilled: 11/18/2015  Logged by: Jay Burnham  Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1085  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>5</td>
<td></td>
<td><strong>4.0&quot; ASPHALT CONCRETE / 8.0&quot; AGGREGATE BASE</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>ALLUVIUM:</strong> Silty Sand (SM): fine to medium-grained, olive brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>BEDROCK:</strong> Mixed Granodiorite and Gabbro severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, olive gray.</td>
</tr>
</tbody>
</table>

La Sierra Pipeline Project  Project No. 14-81-137-01
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  Drawing No. A-23
City of Riverside and Unincorporated Riverside County, California  For: Albert A. Webb Associates
**Log of Boring No. BH-24**

Dates Drilled: 11/18/2015  
Logged by: Jay Burnham  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1171  
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>5.0&quot; ASPHALT CONCRETE / 12.0&quot; AGGREGATE BASE</th>
</tr>
</thead>
</table>
| 5          |             | ALLUVIUM:  
SILTY SAND (SM): fine to medium-grained, olive brown. |
|            |             | BEDROCK  
MIXED GRANODIORITE AND GABBRO severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, olive gray. |

Auger refusal at 6.0 feet bgs.  
Groundwater not encountered.  
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/18/2015.

La Sierra Pipeline Project  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates

**Converse Consultants**

Project No. 14-81-137-01  
Drawing No. A-24
**Log of Boring No. BH-25**

**Dates Drilled:** 11/18/2015  
**Logged by:** Jay Burnham  
**Checked By:** Scot Mathis

**Equipment:** 8" Hollow Stem Auger  
**Driving Weight and Drop:** 140 lbs / 30 in

**Ground Surface Elevation (ft):** 1208  
**Depth to Water (ft):** NOT ENCOUNTERED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th><strong>SUMMARY OF SUBSURFACE CONDITIONS</strong></th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<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>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td><strong>ALLUVIUM:</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Silty Sand (SM):</strong> fine to coarse-grained, scattered gravel, red brown.</td>
<td>50-5&quot; 3 disturbed</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td><strong>BEDROCK:</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Mixed Granodiorite and Gabbro</strong> severely weathered. Excavates as Silty Sand (SM): fine to coarse-grained, olive brown.</td>
<td>50-6&quot; 5 107 disturbed</td>
</tr>
<tr>
<td>10.25</td>
<td></td>
<td>End of boring at 10.25 feet bgs. Groundwater not encountered. Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/18/2015.</td>
<td>50-3&quot; no recovery</td>
</tr>
</tbody>
</table>

**Equipment:** 8" Hollow Stem Auger  
**Checked By:** Scot Mathis

---

**La Sierra Pipeline Project**  
La Sierra Avenue Between Sterling Avenue and El Sobrante Road  
City of Riverside and Unincorporated Riverside County, California  
For: Albert A. Webb Associates

**Project No.:** 14-81-137-01  
**Drawing No.:** A-25
Log of Boring No. BH-26

<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>
</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.

5.0" ASPHALT CONCRETE / 6.0" AGGREGATE BASE

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

<table>
<thead>
<tr>
<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>6/7/9</td>
<td>6</td>
<td>107</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4/8/8</td>
<td>7</td>
<td>111</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9/12/15</td>
<td>6</td>
<td>108</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 11.5 feet bgs.
Groundwater not encountered.
Borehole backfilled loose with soil cuttings and lightly tamped on 11/18/2015.
**Log of Boring No. BH-27**

Dates Drilled: 11/18/2015  
Logged by: Jay Burnham  
Checked By: Scot Mathis

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

Ground Surface Elevation (ft): 1260  
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.

#### 5.0” ASPHALT CONCRETE / NO AGGREGATE BASE

<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>

**ALLUVIUM:**

- **SILTY SAND (SM):** fine to coarse-grained, dark olive brown.
- trace clay, fine to medium-grained

<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>

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

End of boring at 11.5 feet bgs.  
Groundwater not encountered.  
Borehole backfilled loose with soil cuttings and surface patched with asphalt concrete on 11/18/2015.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

### SAMPLES

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date</th>
<th>Drive</th>
<th>Bulk</th>
<th>Moisture</th>
<th>Dry Unit Wt.</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/16/16</td>
<td></td>
<td></td>
<td>5</td>
<td>108</td>
<td>ma</td>
</tr>
<tr>
<td>4/4/3</td>
<td></td>
<td></td>
<td>11</td>
<td>111</td>
<td></td>
</tr>
<tr>
<td>5/6/9</td>
<td></td>
<td></td>
<td>11</td>
<td>118</td>
<td>col, ds</td>
</tr>
</tbody>
</table>

---
Appendix B

Laboratory Testing Program
APPENDIX B

LABORATORY TESTING PROGRAM

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

*In-Situ* Moisture Content and Dry Density

Results of these tests, which were performed on relatively undisturbed ring samples, 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*.

**Sand Equivalent**

Four 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>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Sand Equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM)</td>
<td>16</td>
</tr>
<tr>
<td>BH-7</td>
<td>5.0-10.0</td>
<td>Silty Sand (SM)</td>
<td>13</td>
</tr>
<tr>
<td>BH-19</td>
<td>0.0-5.0</td>
<td>Sand (SP)</td>
<td>51</td>
</tr>
<tr>
<td>BH-24</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM)</td>
<td>44</td>
</tr>
</tbody>
</table>

**Soil Corrosivity**

Three representative soil samples were tested in accordance with California Test Methods 643, 422, and 417, 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 EG Laboratory. Test results are presented on the following table.
Table No. B-2, Summary of Corrosivity Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>pH</th>
<th>Soluble Sulfates (CA 417) (percent by weight)</th>
<th>Soluble Chlorides (CA 422) (ppm)</th>
<th>Min. Resistivity (CA 643) (Ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>1.0-5.0</td>
<td>7.2</td>
<td>0.010</td>
<td>180</td>
<td>4,200</td>
</tr>
<tr>
<td>BH-11</td>
<td>5.0-10.0</td>
<td>7.9</td>
<td>0.003</td>
<td>190</td>
<td>2,300</td>
</tr>
<tr>
<td>BH-15</td>
<td>5.0-10.0</td>
<td>7.9</td>
<td>0.032</td>
<td>280</td>
<td>3,800</td>
</tr>
</tbody>
</table>

Collapse Tests

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, six representative ring samples were loaded up to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The tests were conducted in accordance with ASTM Standard D5333 laboratory procedure. The test results are presented in the following table.

Table No. B-3, Collapse Test Result

<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-1</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM)</td>
<td>-1.7</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-7</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM), trace Clay</td>
<td>-0.2</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-11</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM), trace Clay</td>
<td>-2.9</td>
<td>Moderate</td>
</tr>
<tr>
<td>BH-14</td>
<td>10.0-11.5</td>
<td>Sandy Clay (CL)</td>
<td>-0.2</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-26</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM)</td>
<td>-1.6</td>
<td>Slight</td>
</tr>
<tr>
<td>BH-27</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM)</td>
<td>-1.4</td>
<td>Slight</td>
</tr>
</tbody>
</table>

Grain-Size Analysis

To assist in classification of soils, six mechanical grain-size analyses were performed on selected samples in general accordance with the ASTM D422 method. Grain-size curves are shown in Drawing No. B-1, Grain Size Distribution Results.

Laboratory Maximum Dry Density

Laboratory maximum dry density and optimum moisture content relationship tests were performed on four representative bulk samples. The tests were conducted in accordance with ASTM Standard D1557 method. Test results are presented on Drawing No. B-2, Moisture-Density Relationship Results, and summarized in the following tables.
Table No. B-4, Laboratory Maximum Density Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Maximum Dry Density (pcf)</th>
<th>Optimum Moisture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM), Red Brown</td>
<td>132.0</td>
<td>8.3</td>
</tr>
<tr>
<td>BH-10</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM), Olive Yellow</td>
<td>126.0</td>
<td>8.5</td>
</tr>
<tr>
<td>BH-15</td>
<td>5.0-10.0</td>
<td>Silty Sand (SM), Brown</td>
<td>127.5</td>
<td>9.5</td>
</tr>
<tr>
<td>BH-19</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM), Olive Yellow</td>
<td>129.0</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Direct Shear

Six direct shear tests were performed on relatively undisturbed representative soil samples at soaked moisture conditions, in accordance with the ASTM D3080 method. For each 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-8, Direct Shear Test Results, and in the following table.

Table No. B-5, 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>BH-1</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM), trace Clay</td>
<td>27</td>
</tr>
<tr>
<td>BH-4</td>
<td>15.0-16.5</td>
<td>Silty Sand (SM), trace Clay</td>
<td>31</td>
</tr>
<tr>
<td>BH-8</td>
<td>15.0-16.5</td>
<td>Sandy Silt (ML)</td>
<td>30</td>
</tr>
<tr>
<td>BH-11</td>
<td>16.0-16.5</td>
<td>Silty Sand (SM), trace Clay</td>
<td>29</td>
</tr>
<tr>
<td>BH-15</td>
<td>5.0-6.5</td>
<td>Silty Sand (SM)</td>
<td>33</td>
</tr>
<tr>
<td>BH-27</td>
<td>10.0-11.5</td>
<td>Silty Sand (SM)</td>
<td>36</td>
</tr>
</tbody>
</table>

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

La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

Converse Consultants

Project ID: 14-81-137-01.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-19</td>
<td>1.0-5.0</td>
<td>Sand (SP)</td>
<td></td>
<td></td>
<td></td>
<td>1.22</td>
<td>10.48</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-19</td>
<td>1.0-5.0</td>
<td>12.5</td>
<td>1.11</td>
<td>0.378</td>
<td>0.106</td>
<td>1.0</td>
<td>98.9</td>
<td>0.1</td>
<td></td>
</tr>
</tbody>
</table>
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-1</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM) Red Brown</td>
<td>D1557 - B</td>
<td>8.3</td>
<td>132</td>
</tr>
<tr>
<td>□</td>
<td>BH-10</td>
<td>1.0-5.0</td>
<td>Silty Sand (SM), Olive Yellow</td>
<td>D1557 - B</td>
<td>9</td>
<td>126</td>
</tr>
<tr>
<td>▲</td>
<td>BH-15</td>
<td>5.0-10.0</td>
<td>Silty Sand (SM), Brown</td>
<td>D1557 - B</td>
<td>9.5</td>
<td>127.5</td>
</tr>
<tr>
<td>★</td>
<td>BH-19</td>
<td>1.0-5.0</td>
<td>Silty Sand (SP), Olive Yellow</td>
<td>D1557 - B</td>
<td>8.5</td>
<td>129</td>
</tr>
</tbody>
</table>

Converse Consultants
La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates
DIRECT SHEAR TEST RESULTS

La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and
El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

<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>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>Silty Sand (SM), trace Clay</td>
<td>5.0-6.5</td>
<td>130</td>
<td>27</td>
<td>15.2</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

DIRECT SHEAR TEST RESULTS

<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-4</td>
<td>Silty Sand (SM), trace Clay</td>
<td>15.0-16.5</td>
<td>160</td>
<td>31</td>
<td>6.4</td>
<td>109.8</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and
El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th></th>
<th>DEPTH (ft)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-8</td>
<td></td>
<td>15.0-16.5</td>
<td></td>
</tr>
</tbody>
</table>

DESCRIPTION : Sandy Silt (ML)

COHESION (psf) : 50
FRICITION ANGLE (degrees): 30

MOISTURE CONTENT (%) : 10.9
DRY DENSITY (pcf) : 99.8

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

**La Sierra Pipeline Project**
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-11</th>
<th>DEPTH (ft)</th>
<th>15.0-16.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESCRIPTION</td>
<td>Silty Sand (SM), trace Clay</td>
<td>MOISTURE CONTENT (%)</td>
<td>10.1</td>
</tr>
<tr>
<td>COHESION (psf)</td>
<td>100</td>
<td>FRICTION ANGLE (degrees):</td>
<td>29</td>
</tr>
<tr>
<td>DRY DENSITY (pcf)</td>
<td></td>
<td></td>
<td>113.2</td>
</tr>
</tbody>
</table>

**NOTE:** Ultimate Strength.

---

**Converse Consultants**

Project No. 14-81-137-01
Drawing No. B-6

Project ID: 14-81-137-01.GPJ; Template: DIRECT SHEAR
La Sierra Pipeline Project
La Sierra Avenue Between Sterling Avenue and El Sobrante Road
City of Riverside and Unincorporated Riverside County, California
For: Albert A. Webb Associates

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

<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-27</td>
<td>Sand with Silt (SP)</td>
<td>10.0-11.5</td>
<td>10</td>
<td>36</td>
<td>11.8</td>
<td>108.6</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
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. Six 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>
<thead>
<tr>
<th>Traverse</th>
<th>Location</th>
<th>Approximate Depth to Difficult Excavation or Blasting</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL-1</td>
<td>West of BH-25</td>
<td>10 to 12 feet</td>
<td></td>
</tr>
<tr>
<td>SL-2</td>
<td>BH-24</td>
<td>0 to 25 feet</td>
<td>Possible nested corestones</td>
</tr>
<tr>
<td>SL-3</td>
<td>Planned BH-23 location</td>
<td>12 to 22 feet</td>
<td>Possible corestones at depth</td>
</tr>
<tr>
<td>SL-4</td>
<td>BH-22</td>
<td>5 to 15 feet</td>
<td></td>
</tr>
<tr>
<td>SL-5</td>
<td>BH-21</td>
<td>10 to 30 feet</td>
<td>Possible corestones</td>
</tr>
<tr>
<td>SL-6</td>
<td>South of Lake Knoll Pkwy.</td>
<td>10 to 30 feet</td>
<td></td>
</tr>
</tbody>
</table>
December 11, 2015  
Project No. 115575

Mr. Scot Mathis  
Converse Consultants  
10391 Corporate Drive  
Redlands, CA 92373

Subject: Seismic Refraction Survey  
La Sierra Pipeline Project  
Riverside, California

Dear Mr. Mathis:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the La Sierra Pipeline Project located in Riverside, California. Specifically, our survey consisted of performing six 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/ev

Distribution: Addressee (electronic)
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Figure 4b – Seismic Profile, SL-2
Figure 4c – Seismic Profile, SL-3
Figure 4d – Seismic Profile, SL-4
Figure 4e – Seismic Profile, SL-5
Figure 4f – Seismic Profile, SL-6
1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the La Sierra Pipeline Project located in Riverside, California (Figure 1). Specifically, our survey consisted of performing six 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 six 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 consists of the shoulder areas along La Sierra Avenue roughly between its intersection with El Sobrante Road and Lake Knoll Parkway in Riverside, California (Figure 1). Topography along the roadway consists of hills and associated drainages. Granitic rock cut slopes are present along the project site. In addition, outcrops of granitic rock were observed in several areas along the project site. Figures 2a through 2c, and 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 line using conventional cut and cover methods. 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 sur-
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.

Six seismic lines (SL-1 through SL-6) 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 seis-
mic 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 homoge-
nous 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 assumes 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>Seismic P-wave Velocity</th>
<th>Rippability</th>
</tr>
</thead>
<tbody>
<tr>
<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 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, six 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 4f present the velocity models generated from our study. The approximate locations of the seismic refraction traverses are shown on the Line Location Maps (Figures 2a
through 2c). 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 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.

SITE PHOTOGRAPHS
(SL-4 through SL-6)
REVISED GEOTECHNICAL INVESTIGATION REPORT

La Sierra Pipeline Project
Arlington Desalter Booster Pump Station and Reservoir
East End of Sterling Avenue
City of Riverside, Riverside County, California
Converse Project No. 15-81-137-02

February 26, 2016

Prepared For:

Albert A. Webb Associates
3788 McCray Street
Riverside, CA 92506

Prepared By:

Converse Consultants
10391 Corporate Drive
Redlands, California 92374
October 19, 2015
Revised February 26, 2016

Mr. Siming Zhang, P.E.
Senior Engineer
Albert A. Webb Associates
3788 McCray Street
Riverside, CA 92506

Subject: REVISED GEOTECHNICAL INVESTIGATION REPORT
La Sierra Pipeline Project
Arlington Desalter Booster Pump Station and Reservoir
East End of Sterling Avenue
City of Riverside, Riverside County, California
Converse Project No. 15-81-137-02

Dear Mr. Zhang:

Converse Consultants (Converse) is pleased to submit this revised geotechnical investigation report to assist with the design and construction of the Western Municipal Water District (WMWD) Arlington Desalter booster pump station and reservoir located within the City of Riverside, Riverside County, California. This report has been modified to incorporate a revised site layout, including a 1.1 MG reservoir. This revised report was prepared in accordance with our proposal dated November 4, 2015 and your Task Order Agreement number 2014-0216 dated November 5, 2015.

Based on our investigation, testing, and analysis, the site is considered suitable from a geotechnical standpoint for the proposed booster station and reservoir provided the recommendations presented herein are incorporated during design and construction.

We appreciate the opportunity to be of service to Albert A. Webb Associates and the Western Municipal Water District. If you have any questions, please do not hesitate to contact us at (909) 796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, Ph.D., P.E., G.E.
Principal Engineer

Dist.: 4/Addressee
JB/SM/HSQ/kvg
PROFESSIONAL CERTIFICATION

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

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

Hashmi S.E. Quazi Ph.D., P. E., G.E.
Principal Engineer

Scot Mathis, C.E.G.
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 approximately 1.09-acre project site is located on the north side of the east end of Sterling Avenue within the City of Riverside, Riverside County, California. The currently undeveloped site is bounded by Sterling Avenue to the south, State Route 91 to the north, commercial/industrial developments to the west, and the Santa Ana Watershed Project and Arlington Desalter facilities to the east.

- The proposed Arlington Desalter booster pump station will include a pump building, surge tank, and associated piping. The building will house four active and one standby vertical pumps with a total initial capacity of 4,375 gpm, as well as electrical and generator rooms.

- The proposed 1.1 MG prestressed concrete water reservoir will be approximately 30 feet in height and 86 feet in diameter. The tank floor will be approximately 16 feet below existing grade.

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

- Two exploratory borings (BH-1 and BH-2) were drilled on September 17, 2015 to their maximum planned depths of 21.5 and 51.5 feet bgs, respectively.

- Two cone penetrometer test soundings (CPT-1 and CPT-2) were advanced at the proposed reservoir location on November 17, 2015 to depths of 50.5 and 46.1 feet bgs, respectively.

- The site is underlain to a depth of at least 51.5 feet by alluvial sediments. From the surface to approximately 20 feet bgs, the alluvium consists primarily of sandy silt with scattered thin clay layers. Below 20 feet bgs, the sediments encountered included silt and silty to clayey sand.

- Groundwater was not encountered in our exploratory borings to a maximum explored depth of 51.5 feet bgs. The historical high groundwater level is approximated to be 36 feet bgs. Groundwater is not expected to be encountered during the construction of this project.
Based on our subsurface exploration, we anticipate that the site soils will be excavatable with conventional heavy duty earthworking and trenching equipment.

The project site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone. There are no known active faults projecting toward or extending across the project site. Seismic design parameters for the site and proposed alignments are presented in the text of this report.

The site has the potential for liquefaction or dry settlement.

The potential for earthquake-induced lateral spreading, landsliding, or flooding at the site is considered low.

The site soils have very low expansion potential.

The site soils have a slight to moderate collapse potential.

The site soils do not contain elevated concentrations of soluble sulfates or chlorides, but are corrosive to ferrous metals. A corrosion engineer should be consulted for the corrosion mitigation measures for ferrous metals in contact with soil.

The reservoir footprint should be overexcavated to at least 3 feet below the bottom of the deepest footing. The depth of overexcavation should be uniform across the entire reservoir. The overexcavation should extend laterally at least 3 feet beyond the reservoir footprint. The overexcavation should be deepened as needed to remove any existing fill, and any very soft or saturated soil.

Prior to the start of any earthwork, the site should be cleared of all vegetation, existing fill, and debris. The materials resulting from the clearing and grubbing operations should be removed from the site.

The building footprint and any other areas to support structures except for the reservoir should be overexcavated to at least 12 inches below the bottom of the footings. The depth of overexcavation should be uniform across the entire structure. The overexcavation should extend at least 2 feet beyond the structure footprint. Pavement and flatwork areas should be overexcavated to a depth of at least 1 foot below subgrade. The overexcavations should extend at least 1 foot beyond the edge of pavement. The overexcavations should be deepened as needed to remove any existing fill, and any very soft or saturated soil.

Excavated onsite earth materials cleared of deleterious matter can be moisture conditioned and re-used as compacted fill.
Fill soils should be placed on properly prepared excavation bottoms, moisture conditioned, and compacted to at least 90 percent of the laboratory maximum dry density. At least the upper 12 inches of fill beneath pavement intended to support vehicle loads should be compacted to at least 95 percent of the laboratory maximum dry density.

The proposed building may be supported by continuous or isolated spread shallow footings. The footings should be at least 18 inches in width and embedded to at least 18 inches below the lowest adjacent grade. The footing reinforcement should be based on structural design. Footings can be designed based on an allowable net bearing capacity of 3,000 pounds per square foot (psf).

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 0.5 inch or less. The differential settlement resulting from static loads is anticipated to be 0.25 inches or less.

The site has the potential for up to 0.3 inches of dynamic settlement during a large earthquake under current groundwater conditions. Dynamic differential settlement under these conditions may be up to approximately 0.2 inches over 50 horizontal feet.

If the groundwater level returns to historical high levels from the site vicinity, the potential dynamic settlement may be up to 0.5 inches, and the dynamic differential settlement may be up to approximately 0.5 inches over 50 horizontal feet.

Lateral earth pressures, foundation design parameters, and pipeline design parameters are presented in the text of this report.

Recommendations for temporary sloped excavations and temporary shoring are provided in the text of this report.

Based on our investigation, we believe that the project site is suitable for construction of the proposed booster pump station and reservoir, 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 presents the results of our geotechnical investigation performed for the proposed Arlington Desalter booster pump station (BPS) and water reservoir, located on the north side of the east end of Sterling Avenue, within the City of Riverside, Riverside County, California, as shown in Figure No. 1, Site Location Map.

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

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

2.0 PROJECT DESCRIPTION

The project consists of the design and construction of a 1.1 million-gallon reservoir, pump building, surge tank, and associated piping. Detailed project plans were not available at the time of this report however, a preliminary site layout exhibit was provided (Michael Baker, 2015).

We understand that the reservoir will be constructed of pre-stressed concrete and will be approximately 86 feet in diameter, with a height of approximately 30 feet. The reservoir floor will be approximately 16 feet below the existing grade. We anticipate that the reservoir walls will be supported on a continuous ring footing and that the roof columns will be supported by isolated spread footings. We understand that the reservoir footings will be underlain by a minimum of 12 inches of Class 2 aggregate base, and the reservoir floor will be underlain by a minimum of 6 inches of compacted aggregate base.

The proposed pump building footprint is approximately 40 feet by 115 feet. We anticipate that the building will be a one-story masonry block wall structure founded on shallow footings with a slab-on-grade. Four active and one standby/rotational vertical pump cans are planned. Each pump is to have a capacity of approximately 1,094 gallons per minute (gpm), for a total initial BPS capacity of 4,375 gpm. The building constructed to house the pump station equipment will also include electrical and generator rooms. The surge tank will be located to the west of the pump building.
Site Location Map

Project: Arlington Desalter Booster Pump Station and Reservoir
Location: East End of Sterling Avenue, City of Riverside, California
Client: Albert A. Webb Associates

Converse Consultants

Project No.
14-81-137-02
FIGURE NO.
1
3.0 SITE DESCRIPTION

The approximately 1.09-acre proposed project site is located on the north side of the east end of Sterling Avenue within the City of Riverside, Riverside County, California. The currently undeveloped site is bounded by Sterling Avenue to the south, State Route 91 to the north, commercial/industrial developments to the west, and the Santa Ana Watershed Project and Arlington Desalter facilities to the east. The site has an approximate elevation of approximately 712 to 714 feet above mean sea level with a slight slope to the west (Michael Baker, 2015). It is covered with gravel and coarse-grained sand with a chain-link fence around the perimeter.

4.0 SCOPE OF WORK

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

4.1 Project Set-up

The project set-up consisted of the following:

- Preparing a investigation location map for review and approval by Albert A. Webb Associates.
- Marking the boring and CPT locations in the field such that drill rig access to all the locations was available.
- Notifying Underground Service Alert (USA) at least 48 hours prior to investigation to clear the locations of any conflict with existing underground utilities.
- Engaging drilling and CPT subcontractors.

4.2 Subsurface Exploration

Two exploratory borings (BH-1 and BH-2) were drilled on September 17, 2015 at the proposed BPS site. The borings were advanced to their maximum planned depths of 21.5 and 51.5 feet bgs respectively.

Two cone penetrometer test soundings (CPT-1 and CPT-2) were advanced at the proposed reservoir location on November 17, 2015 to depths of 50.5 and 46.1 feet bgs, respectively.

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

BH-2  Number and Approximate Location of Exploratory Boring

CPT-2  Number and Approximate Location of CPT Sounding

Approximate Boring and CPT Location Map

Project: Arlington Desalter Booster Pump Station and Reservoir
Location: East End of Sterling Avenue, City of Riverside, California
Client: Albert A. Webb Associates

Converse Consultants

Project No. 14-81-137-02
4.3 Laboratory Testing

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

- *In-situ* moisture contents and dry densities (ASTM Standard D2216).
- Expansion index (ASTM Standard D4829).
- Soil corrosivity tests (California Tests 643, 422, and 417).
- Swell/Collapse (ASTM Standard D5333).
- Atterberg Limits (ASTM Standard D4318).
- Grain size analysis (ASTM Standard D422).
- Maximum dry density and optimum-moisture content (ASTM Standard D1557).
- Direct shear (ASTM Standard D3080).

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

4.4 Analysis and Report Preparation

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

5.0 ENGINEERING GEOLOGY

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

5.1 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, San Andreas, and Cucamonga Fault Zones, 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 west-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.

5.2 Local Geology

The project site is situated in an alluvial valley at the base of the northern-most extent of the Temescal Mountains. Regional mapping (Morton et al., 2001; Morton and Miller, 2006) indicates that the site is generally underlain by Pleistocene-aged old alluvial fan deposits. This alluvium generally consists of moderately to well-consolidated silt, sand, and gravel.

The Elsinore Fault Zone is approximately 7 miles southwest of the site. Lake Mathews is approximately 3.6 miles southeast of the site. The Santa Ana River channel is approximately 5 miles northwest of the site.

6.0 SUBSURFACE CONDITIONS

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

6.1 Subsurface Profile

Based on the exploratory borings, CPT soundings, and laboratory test results, the site is underlain to a depth of at least 51.5 feet by alluvial sediments. From the surface to approximately 20 feet bgs, the alluvium consists primarily of sandy silt with scattered thin clay layers. Below 20 feet bgs, the sediments encountered included silt and silty to clayey sand.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawing Nos. A-2 and A-3, Logs of Borings, in Appendix A, Field Exploration.
6.2 Groundwater

Groundwater was not encountered in our exploratory borings to a maximum explored depth of 51.5 feet bgs. Groundwater data (SWRCB, 2015) from locations in close proximity to the site was reviewed to evaluate the historical groundwater levels.

The nearest reported groundwater measurements to the site were at two sites (T060653082 and T0606599150) located near the intersection of Magnolia Avenue and Pierce Street, approximately 1,800 feet southwest of the project site. The sites, which are at an elevation of approximately 698 feet above mean sea level (amsl), reported groundwater levels between 2001 and 2009. The shallowest groundwater depth reported was approximately 32 feet below ground surface (bgs), at an elevation of approximately 666 feet amsl, in 2001. Reported groundwater levels decreased steadily during the reporting period to approximately 645 feet amsl in the most recent measurements.

Three sites (T060653805, T0606500103, T0606540187) located near the intersection of Magnolia Avenue and La Sierra Avenue, approximately 3,700 feet northeast of the project site at an elevation of approximately 712 feet above mean sea level (amsl), reported groundwater levels between 1991 and 2003. The shallowest groundwater depth reported was approximately 13 feet below ground surface (bgs), at an elevation of approximately 699 feet amsl.

Based on interpolation between the historical high groundwater elevations at sites to the northeast and southwest, the at the historical high groundwater elevation at the project site is estimated to be 677 feet amsl or approximately 36 feet bgs.

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

6.3 Excavatability

Based on our field observations, subsurface materials at the site are anticipated to be readily excavatable by conventional heavy-duty earth moving equipment such as excavators, scrapers, and trenching machines.

6.4 Subsurface Variations

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

7.0 LABORATORY TEST RESULTS

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

7.1 Physical Testing

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

- **In-situ Moisture and Dry Density** – In-situ dry densities and moisture contents of the upper 10 feet of soil at the site ranged between 110 and 125 pounds per cubic feet (pcf) and between 6 and 14 percent respectively.

- **Expansion Index** – Two representative bulk samples from the site were tested to evaluate the expansion potential of the materials encountered at the site. The tests, conducted in accordance with ASTM Standard D4829, indicated that the expansion indices of the specimens were 2 and 8, corresponding to very low expansion potential.

- **Collapse Potential** – The collapse potential of two relatively undisturbed samples from the upper 5 feet were tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D5333 test method. The results showed a collapse of 1.0 to 3.9 percent, indicating a slight to moderate collapse potential.

- **Grain Size Analysis** – Two representative samples were tested to determine their relative grain size distributions in accordance with the ASTM Standard D422. Test results are graphically presented in Drawings No. B-1, Grain Size Distribution Results. The test results indicate that the site soils are primarily sandy silt (ML).

- **Maximum Dry Density and Optimum Moisture Content** – One typical moisture-density relationship of a representative soil sample was tested, according to ASTM Standard D1557-B, with the results presented in Drawing No. B-2, Moisture-Density Relationship Results, in Appendix B, Laboratory Testing Program. The laboratory maximum dry density was 134 pounds per cubic feet (pcf), with an optimum moisture content of 7 percent.
• Direct Shear – One direct shear test was performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The result of the direct shear test is presented in Drawing No. B-3, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

### 7.2 Chemical Testing - Corrosivity Evaluation

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The 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 EG Labs 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 are 7.66 and 8.15.
- The sulfate contents of the samples tested were 0.001 and 0.025 percent by weight.
- The chloride concentrations of the samples tested were 120 and 265 ppm.
- The minimum electrical resistivities when saturated were 1,900 and 2,600 ohm-cm.

### 8.0 FAULTING AND SEISMICITY

The approximate distance and seismic characteristics of nearby faults as well as seismic design coefficients are discussed in the following subsections.

#### 8.1 Faulting

The project site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2015). There are no known active faults projecting toward or extending across the project site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

The proposed site is situated in a seismically active region. As is the case for most areas of Southern California, ground shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site.

The following table contains a list of active and potentially active faults within one-hundred (100) kilometers of the subject site. The fault parameters and distances presented in the following table are based on the output from EQFAULT (Blake, 2000), revised in accordance with CGS fault parameters (Cao et. al., 2003).
Table No. 1, Seismic Characteristics of Nearby Active Faults

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Approximate Distance (km)</th>
<th>Moment Magnitude (Mw)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chino-Central Ave. (Elsinore)</td>
<td>7.8 (12.5)</td>
<td>6.7</td>
</tr>
<tr>
<td>Elsinore-Glen Ivy</td>
<td>8.2 (13.2)</td>
<td>6.8</td>
</tr>
<tr>
<td>Whittier</td>
<td>9.4 (15.1)</td>
<td>6.8</td>
</tr>
<tr>
<td>San Jacinto-San Bernardino</td>
<td>15.6 (25.1)</td>
<td>6.7</td>
</tr>
<tr>
<td>San Jacinto-San Jacinto Valley</td>
<td>16.6 (26.7)</td>
<td>6.9</td>
</tr>
<tr>
<td>Elsinore-Temecula</td>
<td>19.3 (31.0)</td>
<td>6.8</td>
</tr>
<tr>
<td>San Jose</td>
<td>19.7 (31.7)</td>
<td>6.4</td>
</tr>
<tr>
<td>Cucamonga</td>
<td>21.2 (34.1)</td>
<td>6.9</td>
</tr>
<tr>
<td>Elysian Park Thrust</td>
<td>22.4 (36.1)</td>
<td>6.7</td>
</tr>
<tr>
<td>Sierra Madre</td>
<td>22.4 (36.1)</td>
<td>7.2</td>
</tr>
<tr>
<td>San Andreas - San Bernardino</td>
<td>23.8 (38.3)</td>
<td>7.5</td>
</tr>
<tr>
<td>San Andreas - Southern</td>
<td>23.8 (38.3)</td>
<td>7.4</td>
</tr>
<tr>
<td>Cleghorn</td>
<td>27.5 (44.3)</td>
<td>6.5</td>
</tr>
<tr>
<td>San Andreas - 1857 Rupture</td>
<td>28.8 (46.4)</td>
<td>7.8</td>
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<tr>
<td>San Andreas - Mojave</td>
<td>28.8 (46.4)</td>
<td>7.4</td>
</tr>
<tr>
<td>Compton Thrust</td>
<td>29.6 (47.6)</td>
<td>6.8</td>
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<tr>
<td>North Frontal Fault Zone (West)</td>
<td>30.1 (48.5)</td>
<td>7.2</td>
</tr>
<tr>
<td>Newport-Inglewood (Offshore)</td>
<td>32.1 (51.7)</td>
<td>7.1</td>
</tr>
<tr>
<td>Newport-Inglewood (L.A.Basin)</td>
<td>32.3 (52.0)</td>
<td>7.1</td>
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<tr>
<td>Clamshell-Sawpit</td>
<td>33.4 (53.8)</td>
<td>6.5</td>
</tr>
<tr>
<td>San Jacinto-Anza</td>
<td>34.4 (55.3)</td>
<td>7.2</td>
</tr>
<tr>
<td>Raymond</td>
<td>35.7 (57.5)</td>
<td>6.5</td>
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<tr>
<td>Verdugo</td>
<td>41.3 (66.5)</td>
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<td>Palos Verdes</td>
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<td>Elsinore-Julian</td>
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<td>Pinto Mountain</td>
<td>45.1 (72.6)</td>
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<tr>
<td>North Frontal Fault Zone (East)</td>
<td>45.5 (73.2)</td>
<td>6.7</td>
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<tr>
<td>Hollywood</td>
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<td>Helendale - S. Lockhardt</td>
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<td>7.3</td>
</tr>
<tr>
<td>Coronado Bank</td>
<td>50.3 (80.9)</td>
<td>7.6</td>
</tr>
<tr>
<td>Rose Canyon</td>
<td>53.1 (85.4)</td>
<td>7.2</td>
</tr>
<tr>
<td>San Gabriel</td>
<td>54.2 (87.2)</td>
<td>7.2</td>
</tr>
<tr>
<td>Sierra Madre (San Fernando)</td>
<td>54.4 (87.5)</td>
<td>6.7</td>
</tr>
<tr>
<td>Santa Monica</td>
<td>54.9 (88.3)</td>
<td>6.6</td>
</tr>
<tr>
<td>Lenwood-Lockhart-Old Woman Springs</td>
<td>58.1 (93.5)</td>
<td>7.5</td>
</tr>
<tr>
<td>San Andreas - Coachella</td>
<td>58.3 (93.8)</td>
<td>7.2</td>
</tr>
<tr>
<td>Northridge (E. Oak Ridge)</td>
<td>59.2 (95.3)</td>
<td>7.0</td>
</tr>
<tr>
<td>Malibu Coast</td>
<td>60.9 (98.0)</td>
<td>6.7</td>
</tr>
</tbody>
</table>
8.2 CBC Seismic Design Parameters

Seismic parameters based on the California Building Code (CBSC, 2013) were determined using the Seismic Design Maps application (USGS, 2015b) and are provided in the following table.

Table No. 2, CBC Seismic Parameters

<table>
<thead>
<tr>
<th>Seismic Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Coordinates</td>
<td>33.896 N, 117.484 W</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>Risk Category</td>
<td>IV</td>
</tr>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_s$</td>
<td>1.500g</td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
<td>0.600g</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>1.500g</td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{M1}$</td>
<td>0.900g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for short period $S_{DS}$</td>
<td>1.000g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for 1-second period, $S_{D1}$</td>
<td>0.600g</td>
</tr>
<tr>
<td>Maximum Peak Ground Acceleration, $PGA_M$</td>
<td>0.527g</td>
</tr>
</tbody>
</table>

8.3 Secondary Effects of Seismic Activity

In general, secondary effects of seismic activity include surface fault rupture, soil liquefaction, landslides, lateral spreading, and settlement due to seismic shaking, tsunamis, seiches, and earthquake-induced flooding. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture: The site is not located within a currently designated State of California or Riverside County Earthquake Fault Zone (CGS, 2007; Riverside County, 2015). There are no known active faults projecting toward or extending across the project site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

Liquefaction: Liquefaction is defined as the phenomenon in which a cohesionless soil mass within the upper 50 feet of the ground surface, suffers a substantial reduction in its shear strength, due 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 generally occurs in submerged granular soils and non-plastic silts during or after strong ground shaking. There are several general requirements for liquefaction to occur. They are as follows:

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

Regional hazard maps (Riverside County, 2015) indicate that the site is within a zone designated as having moderate liquefaction susceptibility. The historical high groundwater level at the site is approximately 15 feet bgs. Based on the analysis presented in Appendix C, Liquefaction and Settlement Analysis, the site has a potential for up to approximately 0.6 inches of liquefaction-induced settlement.

**Seismic Settlement:** Seismically-induced settlement occurs in unsaturated, unconsolidated, granular sediments during ground shaking associated with earthquakes. The analysis presented in Appendix C, Liquefaction and Settlement Analysis indicates that the site has the potential for up to approximately 0.3 inches of dry seismic settlement.

**Landslides:** Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. Due to the relatively flat nature of the project site, the risk of landsliding is considered low.

**Lateral Spreading:** Seismically induced lateral spreading involves primarily lateral movement of earth materials over underlying materials which are liquefied due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Due to the relatively flat nature of the project site, and the low potential for liquefaction in the near-surface soils, the risk of lateral spreading is considered low.

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

**Seiches:** Seiches are large waves generated in enclosed bodies of water in response to ground shaking. The project site is located approximately 250 feet northwest of Arlington Channel. There is a low potential for seiching to affect the project site in the event of a major seismic event coinciding with high flow within the channel.
Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is located approximately 3.6 miles northwest of the Lake Mathews dam and approximately 700 feet lower in elevation. The project site is located within the Lake Mathews dam inundation zone (City of Riverside, 2012). The site is not located near any other major water-retaining structures.

9.0 EARTHWORK RECOMMENDATIONS

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

9.1 General

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

Prior to the start of construction, all 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.

9.2 Remedial Grading

Building footings, slabs-on-grade, and other shallow or at-grade structures 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.
Reservoir: The reservoir footprint should be overexcavated to at least 3 feet below the bottom of the deepest footing. The depth of overexcavation should be uniform across the entire reservoir. The overexcavation should extend laterally at least 3 feet beyond the reservoir footprint. The overexcavation should be deepened as needed to remove any existing fill, and any very soft or saturated soil.

Pump Building: The building footprint and any other areas to support structures should be overexcavated to a depth of 12 inches below the bottom of the footings. The depth of overexcavation should be uniform across the entire structure. The overexcavation should extend to at least 2 feet beyond the footprint of the structure.

Pavement: All areas to receive asphalt or concrete pavement should be overexcavated to a depth of 12 inches below subgrade. The overexcavation should extend at least 1 foot beyond the edge of pavement.

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 Fill Materials

Excavated on-site soils are generally considered suitable for re-use as compacted fill. Prior to re-use, excavated soils should be cleared of all debris, vegetation, rocks larger than 3 inches in maximum dimension, and other deleterious materials. Rocks larger than 1 inch in the largest dimension should not be placed within the upper 12 inches of fill beneath footings and slabs or within 3 feet of the reservoir wall.

Imported soils, if used as fill, should be predominantly granular and meet the following criteria:

- Expansion Index less than 20.
- Free of all deleterious materials.
- Contain no particles larger than 3 inches in the largest dimension.
- Contain less than 30 percent by weight retained on ¾-inch sieve.
- Have a Plasticity Index of 10 or less.
- Corrosivity not greater than the onsite soils.

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

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

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

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method, unless a higher compaction is specified herein. Fill placed in the reservoir overexcavation should be compacted to at least 95 percent of the laboratory maximum dry density. At least the upper 12 inches of subgrade soils underneath pavements intended to support vehicle loads should be scarified, moisture conditioned, and compacted to at least 95 percent of the laboratory maximum dry density.

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.

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

9.5 **Site Drainage**

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. The building pad should have a gradient of at least 2 percent towards drainage facilities. 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 Utility Trench Backfill

The following sections present earthwork recommendations for utility trench backfill, including subgrade preparation, pipe bedding, and trench zone backfill.

Open cuts adjacent to existing roadways and/or adjacent structures are not recommended within a 1:1 (horizontal:vertical) plane extending down and away from the roadway or structure perimeter.

Spoils from the trench excavation should not be stockpiled more than 6 feet in height or within a horizontal distance from the trench edge equal to the depth of the trench. Spoils should not be stockpiled behind the shoring, if any, within a horizontal distance equal to the depth of the trench, unless the shoring has been designed for such loads.

9.6.1 Pipeline Subgrade Preparation

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

Any loose, soft and/or unsuitable materials encountered at the pipe subgrade should be removed and replaced with 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 12 inches above the pipe. Specifications for bedding materials including required backfill requirements surrounding the pipe should be specified by the design engineer in accordance with the pipe manufacturer’s guideline.

To provide uniform and firm support for the pipeline, free-draining granular soil should be used as pipe bedding material. For flexible pipes, excavated sandy materials may be used as bedding material. Crushed rock or gravel may be used for rigid pipes. Bedding material for the pipes should be free from oversized particles greater than 1 inch in maximum dimension. Pipe design generally requires sand equivalent of 30 or greater for bedding materials. Based on the soil types encountered during our investigation, the onsite soils may not be suitable for use as pipe bedding.
Migration of fines from the surrounding soils must be considered in selecting the gradation of any imported bedding material. To avoid migration of fines if coarse bedding material is utilized, commercially available geofabric used for filtration purposes (such as Mirafi 140N, or equivalent) may be wrapped around the bedding material encasing the pipe to separate the bedding material from the surrounding native or fill soils.

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 particles larger than 6 inches in maximum dimension and deleterious matter may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site.

Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Trench backfill should be thoroughly mixed and evenly spread in maximum 8-inch, loose, horizontal lifts. Coarse-grained fill soils should be moisture conditioned to within 3 percent of the optimum moisture content and fine-grained fill soils should be moisture conditioned to 0 to 2 percent above optimum moisture content.

Rocks larger than 1 inch should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in largest dimension. Rocks shall be well mixed with finer soil.

Trench backfill should be compacted to a minimum of 90 percent of the laboratory maximum dry density by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers. Flooding and jetting are not anticipated to be effective compaction methods based on the observed soil types.

The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations.

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 be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.
10.0 DESIGN RECOMMENDATIONS

Design recommendations for the BPS and reservoir are presented in the following sections.

10.1 General Evaluation

Based on our field exploration, laboratory testing and analyses of subsurface conditions at the site, the proposed structures may be placed on compacted native materials as described in this report.

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

10.2 Footing Design Parameters

The proposed booster pump station building may be supported by continuous/or isolated spread shallow footings supported by at least 12 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 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. Footings can be designed based on an allowable net bearing capacity of 3,000 pounds per square foot (psf).

The reservoir may be supported by a continuous ring footing and interior isolated footings supported by at least 3 feet of fill compacted to 95 percent of the laboratory maximum dry density as described in Section 9.4, Compacted Fill Placement. We understand that the reservoir footings will be underlain by a minimum of 12 inches of Class 2 aggregate base, and the reservoir floor will be underlain by a minimum of 6 inches of compacted aggregate base. The reservoir footings should be at least 24 inches wide and embedded at least 24 inches below the lowest adjacent soil grade. Actual footing width and reinforcement should be based on structural design. Footings placed at a depth of 24 inches below lowest adjacent grade may be designed based on an allowable net bearing capacity of 4,500 pounds per square foot (psf).

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.
A backfill drag coefficient of 0.35 between formed concrete and fill soil placed and compacted as described in Section 9.4, *Compacted Fill Placement*.

### 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 site are presented in the following table.

**Table No. 3, Active and At-Rest Earth Pressures**

<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>

An equivalent fluid seismic pressure of 18H pcf may be assumed at the top of an inverted triangle pressure distribution where H is the height of the backfill behind the wall.

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 245 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.

10.4 Settlement

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 0.5 inch or less. The differential settlement resulting from static loads is anticipated to be 0.25 inches or less over 50 horizontal feet.

The site has the potential for up to 0.3 inches of dynamic settlement during a large earthquake under current groundwater conditions. Dynamic differential settlement under these conditions may be up to approximately 0.2 inches over 50 horizontal feet.

If the groundwater level returns to historical high levels from the site vicinity, the potential dynamic settlement may be up to 0.5 inches, and the dynamic differential settlement may be up to approximately 0.5 inches over 50 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.

10.5 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. 4, Soil Parameters for Pipe Design on the following page.

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

**Table No. 4, Soil Parameters for Pipe Design**

<table>
<thead>
<tr>
<th>Soil Parameters</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total unit weight of compacted backfill (assuming 92% average relative compaction), $\gamma$</td>
<td>132 pcf</td>
</tr>
<tr>
<td>Buoyant weight of backfill, $\gamma_b$</td>
<td>70 pcf</td>
</tr>
<tr>
<td>Angle of internal friction of soils, $\phi$</td>
<td>30°</td>
</tr>
<tr>
<td>Soil cohesion, $c$</td>
<td>100 pcf</td>
</tr>
<tr>
<td>Coefficient of friction between concrete and native soils, $f_s$</td>
<td>0.40</td>
</tr>
<tr>
<td>Coefficient of friction between pipe and native soils, $f_s$</td>
<td>0.25 for metal pipe, 0.30 for CML&amp;C steel pipe</td>
</tr>
<tr>
<td>Bearing pressure against Alluvial Soils</td>
<td>3,000 psf</td>
</tr>
<tr>
<td>Coefficient of passive earth pressure, $K_p$</td>
<td>3.00</td>
</tr>
<tr>
<td>Coefficient of active earth pressure, $K_a$</td>
<td>0.33</td>
</tr>
<tr>
<td>Modulus of Soil Reaction, $E'$</td>
<td>1000 psi</td>
</tr>
</tbody>
</table>

10.6 **Bearing Pressure for Anchor and Thrust Blocks**

An allowable net bearing pressure of 3,000 psf may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 24 inches wide.

Resistance to lateral forces can be assumed to be provided by friction at the base of thrust blocks and by passive earth pressure. An ultimate value of coefficient of friction of 0.40 may be used between the thrust block and the supporting natural soil or compacted fill. A passive earth pressure of 245 psf per foot of depth may be used for the sides of thrust blocks or anchors poured against undisturbed or recompacted soils. The value of the passive lateral earth pressure should be limited to 2,500 psf. Frictional and passive resistance can be combined for the design of anchors and thrust blocks.

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.

10.7 **Soil Corrosivity**

The results of chemical testing of two representative samples of soils from the site were evaluated for corrosivity with respect to common construction materials such as
concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program* and design recommendations pertaining to soil corrosivity are presented below.

The sulfate contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-11, Table 4.2.1). No concrete type restrictions are specified for exposure category S0 (ACI 318-11, Table 4.3.1). A minimum compressive strength of 2,500 psi is recommended.

We anticipate that concrete structures such as footings, slabs, and flatwork will be exposed to moisture from precipitation and irrigation. Based on the site location 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 318-11, Table 4.2.1). ACI provides concrete design recommendations in ACI 318-11, Table 4.3.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

The minimum electrical resistivities when saturated ranged from 1,900 to 2,600 ohm-cm. These values indicate that the site soils are moderately corrosive to 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 alignment soils.

11.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation, shoring design, and other general construction recommendations are presented in the following sections.

11.1 General

Both sloped and vertical braced excavations can be considered for the continuous foundations at the proposed structures and associate pipelines. Recommendations pertaining to temporary excavations are presented in this section.

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

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

### 11.2 Temporary Sloped Excavations

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

**Table No. 5, Slope Ratios for Temporary Excavations**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth of Excavation (ft)</th>
<th>Recommended Maximum Slope (Horizontal:Vertical)¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy Silt, Silty Sand,</td>
<td>0-4</td>
<td>Vertical</td>
</tr>
<tr>
<td>Silt, Clay</td>
<td>4-10</td>
<td>1:1</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 should be provided by the contractor as necessary, to protect the workers in the excavation. Design recommendations for temporary shoring can be provided if necessary.

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

### 11.3 Shoring Design

Temporary shoring will be required where open sloped excavations will not be feasible due to nearby existing structures or facilities. Temporary shoring may consist of the use of a trench box, conventional soldier piles, and lagging or sheet piles. The shoring for the pipe excavations may be laterally supported by walers and cross bracing or 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.
Braced shoring should be designed to support a uniform rectangular lateral earth pressure of 40 psf, as shown in Figure No. 3, *Recommended Lateral Earth Pressures for Braced Excavation*.

Design of cantilever shoring consisting of soldier piles spaced at least two diameters on-center or sheet piles, should be based on Figure No. 4, *Lateral Earth Pressures on Cantilever Wall*.

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).

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 lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier piles. Accordingly, the lagging between the piles may be designed based on the following guidelines:

- Lagging design load = 0.6 of shoring design load
- Maximum lagging load may be 300 psf without surcharges

Excavations for the proposed pipeline should not extend below a 1:1 (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 in length can be exposed at a time to prevent the instability. Backfill should be accomplished in the shortest period of time and in alternating sections.
TEMPORARY BRACED EXCAVATION
LATERAL EARTH PRESSURE

\[ P = P_q + P_a \]
\[ = 0.5q + 40H_1 \]
\[ P_p = 360 \, H_2 \leq 2000 \, \text{psf} \]
\[ \mu = 0.35 \]

- active earth pressure (Cantilever walls)
- passive earth pressure (on native compacted soils)
- ultimate friction coefficient between pile and soil

Notes:
1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp and Pa are the passive and active earth pressure respectively; Pq is the incremental surcharge earth pressure; and \( \mu \) is allowable friction coefficient, applied to dead normal loads acting on non-pile supported elements.
3. 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.
4. Pp includes a safety factor of 1.5.
5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement or slab.
6. For traffic surcharge, use a uniform pressure of 100 psf over the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURE FOR BRACED EXCAVATION
PERMANENT RETAINING WALLS

P = Pq + Pa
= 0.5q + 40H₁
= 0.5q + 60H₁

- active earth pressure (Cantilever walls)
- at rest earth pressure (Restrained walls)

Pp = 245  H₂ ≤ 2000 psf
- passive earth pressure (on native compacted soils)

μ = 0.35
μ = 0.25
- ultimate friction coefficient between backfill and native soils
- ultimate friction coefficient between pipe and native soils

Notes:
1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp, Pa, and Po are the passive, active, and at-rest earth pressures, respectively; Pe is the incremental seismic earth pressure; Pq is the incremental surcharge earth pressure; and μ is the allowable friction coefficient, applied to dead normal loads acting on non-pile supported elements
3. For retained walls (not free to rotate), use at-rest (Po) earth pressure; increase Pe by 30 percent.
4. Base friction coefficient (μ) and Pp include a safety factor of 1.5.
5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement or slab.
6. Surcharge load only applies to the upper 10 feet.
7. Drainage system should be provided for the retaining wall.
8. For traffic surcharge, assume a 100-psf uniform pressure along the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURES ON CANTILEVER WALL

Project: Arlington Desalter Booster Pump Station and Reservoir
Location: East End of Sterling Avenue, City of Riverside, California
Client: Albert A. Webb Associates
Project No: 14-81-137-02

Converse Consultants
12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should be present to observe conditions and test the density and moisture of the backfill during the earthwork for this project. The excavations and backfill should be observed and tested for compliance with project specifications.

13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Albert A. Webb Associates, Western Municipal Water District, and their authorized agents, to assist in the design and construction of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

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

As the project evolves, a 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), 2011, Building Code Requirements for Structural Concrete (ACI 318-11), dated August, 2011.


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

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


CALIFORNIA STATE WATER RESOURCES CONTROL BOARD (SWRCB), 2015, GeoTracker database (http://geotracker.waterboards.ca.gov/), accessed October, 2015.


MICHAEL BAKER INTERNATIONAL, 2015, Yard Piping Plan, La Sierra Pipeline and Pump Stations, undated.


WEBB, ALBERT A., AND ASSOCIATES (WEBB), 2015, La Sierra Pump Station-Suction Alignment Exhibit, 1 sheet, scale 1”=60’ received September 15, 2015.
Appendix A

Field Exploration
APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling soil borings and advancing cone penetrometer test (CPT) soundings. During the site reconnaissance, the surface conditions were noted and the investigation locations were marked. The borings and CPTs were located in the field using approximate distances from local street and buildings as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Soil Borings

Two borings (BH-1 and BH-2) were drilled on September 17, 2015 within the site. The borings were drilled to their respective maximum planned depths of 21.5 and 51.5 feet bgs.

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

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

Standard Penetration Testing (SPT) was also performed in accordance with the ASTM Standard D1586 test method in boring BH-2 at depths of 20.0, 30.0, 40.0 and 50.0 feet bgs using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

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

Converse retained Middle Earth GeoTesting, Inc. to perform two cone penetrometer tests (CPT-1 and CPT-2) at the project site on November 17, 2015 to a maximum depth of 50.5 feet below ground surface (bgs).

The CPT is an in-situ testing method used to determine the geotechnical properties of soils and delineating soil stratigraphy. The method consists of pushing an instrumented cone, 10 cm squared in size, with the tip facing down, into the ground at a controlled rate. For CPT test results, see the CPT Site Investigation prepared by Gregg Drilling and Testing Inc.

The CPT tests were backfilled loose with bentonite pellets. As a result, the surface may settle over time. If construction is delayed, we recommend the owner monitor the CPT locations and backfill any settlement or depressions that might occur, or provide protection around the CPT locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

Following the completion of logging and sampling, the borings were backfilled loosely with soil cuttings and lightly tamped. The CPTs were backfilled with hydrated bentonite pellets. The surface of the borings and CPTs may settle over time. If construction is delayed, we recommend the owner monitor the boring locations and backfill any depressions that might occur, or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings and CPTs, see Drawings No. A-2 and A-5, *Logs of Borings and CPTs*. 
**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, GRAVELS - SAND MIXTURES, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE</strong></td>
<td>GP</td>
<td>POORELY-GRADED GRAVELS, GRAVELS - SAND MIXTURES, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>SAND AND SANDY SOILS</strong></td>
<td>GM</td>
<td>SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES</td>
</tr>
<tr>
<td><strong>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE</strong></td>
<td>GC</td>
<td>CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES</td>
</tr>
<tr>
<td><strong>SANDS WITH FINES</strong></td>
<td>SW</td>
<td>WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>FINES</strong></td>
<td>SP</td>
<td>POORELY-GRADED SANDS, GRAVELY SAND, LITTLE OR NO FINES</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>SM</td>
<td>SILTY SANDS, SAND - SILT MIXTURES</td>
</tr>
<tr>
<td><strong>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE</strong></td>
<td>SC</td>
<td>CLAYEY SANDS, SAND - CLAY MIXURES</td>
</tr>
<tr>
<td><strong>FINE GRAINED SOILS</strong></td>
<td>ML</td>
<td>INORGANIC SILTS AND VERY FINE SANDS, RICH FLOOR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>CL</td>
<td>INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS</td>
</tr>
<tr>
<td><strong>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE</strong></td>
<td>OL</td>
<td>ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>MH</td>
<td>INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS</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>

**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**

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>STRENGTH</th>
<th>CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pocket Penetrometer</td>
<td>p</td>
<td>Plasticity</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>ds</td>
<td>Direct Shear (single point)</td>
</tr>
<tr>
<td>Unconfined Compression</td>
<td>uc</td>
<td>Sand Equivalent</td>
</tr>
<tr>
<td>Triaxial Compression</td>
<td>tx</td>
<td>Compaction Index</td>
</tr>
<tr>
<td>Consolation</td>
<td>c</td>
<td>Expansion Index</td>
</tr>
<tr>
<td>Collapse Test</td>
<td>col</td>
<td>Permeability</td>
</tr>
<tr>
<td>Chemical Analysis</td>
<td>ca</td>
<td>Disturb</td>
</tr>
<tr>
<td>Electrical Resistivity</td>
<td>er</td>
<td>SAND CEMENT</td>
</tr>
<tr>
<td>Permeability</td>
<td>perm</td>
<td>Dist.</td>
</tr>
</tbody>
</table>

**UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS**

Arlington Desalter Booster Pump Station and Reservoir
East End of Sterling Avenue, City of Riverside, California
For: Albert A. Webb Associates

Project No. 14-81-137-02
Drawing No. A-1

Converse Consultants
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:

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

SILT with CLAY (ML): dark gray.

End of boring at 21.5 feet bgs.
No groundwater encountered.
Borehole backfilled loose with soil cuttings and lightly tamped on 9/17/2015.
### 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:</th>
<th>SANDY SILT (ML): fine to medium-grained sand, trace clay, red brown.</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>- thin clay layers</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>- gray brown</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>SI 1T (ML): olive.</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>SI 1T with CLAY (ML): trace fine-grained sand, olive brown.</td>
<td></td>
</tr>
</tbody>
</table>

#### 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></td>
<td></td>
<td></td>
<td></td>
<td>ei</td>
</tr>
<tr>
<td>3/4/5</td>
<td>13</td>
<td>111</td>
<td></td>
<td>col, ds</td>
</tr>
<tr>
<td>15/27/28</td>
<td>7</td>
<td>125</td>
<td></td>
<td>ei, ma</td>
</tr>
<tr>
<td>14/16/18</td>
<td>11</td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13/14/24</td>
<td>13</td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8/12/14</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17/27/30</td>
<td>9</td>
<td>108</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/12/17</td>
<td></td>
<td></td>
<td></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>Depth (ft)</th>
<th>Graphic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

- **SILTY SAND (SM)**: fine-grained, yellow brown.

- fine to medium-grained

End of boring at 51.5 feet bgs.
No groundwater encountered.
Borehole backfilled loose with soil cuttings and lightly tamped on 9/17/2015.

---

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Drive Blows</th>
<th>Bulk Moisture</th>
<th>Dry Unit Wt.</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>BULK</th>
</tr>
</thead>
</table>

- Sample 1: 16/26/33
- Sample 2: 6/14/17
- Sample 3: 15/25/40
- Sample 4: 16/19/25
Converse Consultants

Project: WMWD La Sierra Pipeline
Operator: DG-RC
Cone Number: DDG1333
Filename: SDF(419).cpt

Job Number: 14-81-137-01
Hole Number: CPT-01
Date and Time: 11/17/2015 9:01:10 AM
Maximum Depth: 50.52 ft

Net Area Ratio: 0.8

CPT DATA

<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>TIP</th>
<th>FRICTION</th>
<th>Fs/Qt (%)</th>
<th>SPT N</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>600</td>
<td>14</td>
<td>9</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>600</td>
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<td>0</td>
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<td>15</td>
<td>600</td>
<td>14</td>
<td>9</td>
<td>0</td>
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<tr>
<td>20</td>
<td>600</td>
<td>14</td>
<td>9</td>
<td>0</td>
</tr>
<tr>
<td>25</td>
<td>600</td>
<td>14</td>
<td>9</td>
<td>0</td>
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<tr>
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<td>14</td>
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<td>14</td>
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</tr>
<tr>
<td>50</td>
<td>600</td>
<td>14</td>
<td>9</td>
<td>0</td>
</tr>
</tbody>
</table>

Soil Behavior Type:

1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size: 10cm squared

S* Soil behavior type and SPT based on data from UBC-1983
CPT DATA

- **Net Area Ratio**: 0.8
- **Maximum Depth**: 46.10 ft

**SOIL BEHAVIOR TYPE**
- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

**Cone Size**: 10cm squared

*(Soil behavior type and SPT based on data from UBC-1983)*
Appendix B

Laboratory Testing Program
APPENDIX B

LABORATORY TESTING PROGRAM

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

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

Results of these tests performed on relatively undisturbed ring samples were used to aid in the classification and to provide quantitative measure of the in situ dry density and moisture content. Data obtained from this test 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 Test**

Two representative bulk samples were tested to evaluate the expansion potential of the materials encountered at the site. The tests were conducted in accordance with ASTM Standard D4829. Test results are presented in the following table.

<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-1</td>
<td>5.0-10.0</td>
<td>Sandy Silt (ML)</td>
<td>8</td>
<td>Very Low</td>
</tr>
<tr>
<td>BH-2</td>
<td>0.0-5.0</td>
<td>Sandy Silt (ML)</td>
<td>2</td>
<td>Very Low</td>
</tr>
</tbody>
</table>

**Soil Corrosivity Tests**

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EG Labs in accordance to Caltrans Test Methods 643, 422 and 417. Test results are presented in the following table.
Table No. B-2, Summary of Soil Corrosivity Test Results

<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-1</td>
<td>0.0-5.0</td>
<td>7.66</td>
<td>0.001</td>
<td>120</td>
<td>2,600</td>
</tr>
<tr>
<td>BH-1</td>
<td>5.0-10.0</td>
<td>8.15</td>
<td>0.025</td>
<td>265</td>
<td>1,900</td>
</tr>
</tbody>
</table>

Collapse Tests

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, two representative ring samples were loaded up to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test was conducted in accordance with ASTM Standard D5333 laboratory procedure. The test results are presented in the following table.

Table No. B-3, Collapse Test Result

<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-1</td>
<td>5.0-6.5</td>
<td>Sandy Silt (ML)</td>
<td>-3.9</td>
<td>None</td>
</tr>
<tr>
<td>BH-2</td>
<td>3.5-5.0</td>
<td>Sandy Silt (ML)</td>
<td>-1.0</td>
<td>None</td>
</tr>
</tbody>
</table>

Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analyses were performed on two select samples in accordance with the ASTM Standard D422 test method. Grain-size curves are shown in Drawing No. B-1, Grain Size Distribution Results.

Laboratory Maximum Density Tests

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

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

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Optimum Moisture (%)</th>
<th>Maximum Density (lb/cft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>0.0-5.0</td>
<td>Sandy Silt (ML)</td>
<td>133.5</td>
<td>7.0</td>
</tr>
</tbody>
</table>
Direct Shear Tests

One direct shear test was performed on a relatively undisturbed sample in the soaked moisture condition in accordance with the ASTM D3080 procedure. For this test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing Nos. B-3, Direct Shear Test Results, and the following table.

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

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Description</th>
<th>Friction Angle (degrees)</th>
<th>Cohesion (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>3.5-5.0</td>
<td>Sandy Silt (ML)</td>
<td>30</td>
<td>180</td>
</tr>
</tbody>
</table>

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.
### Table: Curves of 100% Saturation for Specific Gravity Equal to:

<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-1</td>
<td>0.0-5.0</td>
<td>Sandy Silt (ML)</td>
<td>D1557 - B</td>
<td>7</td>
<td>133.5</td>
</tr>
</tbody>
</table>

### MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Project No. 14-81-137-02

Arlington Desalter Booster Pump Station and Reservoir
East End of Sterling Avenue, City of Riverside, California
For: Albert A. Webb Associates

Drawing No. B-2
**DIRECT SHEAR TEST RESULTS**

**Arlington Desalter Booster Pump Station and Reservoir**
East End of Sterling Avenue, City of Riverside, California For: Albert A. Webb Associates

| BORING NO. | BH-2 | DEPTH (ft) | 3.5-5.0 |
| DESCRIPTION | Sandy Silt (ML) |
| COHESION (psf) | 180 | FRICTION ANGLE (degrees): | 30 |
| MOISTURE CONTENT (%) | 13.1 | DRY DENSITY (pcf) : | 110.8 |

**NOTE:** Ultimate Strength.
## ATTERBERG LIMITS RESULTS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>●</td>
<td>BH-2</td>
<td>5.0-10.0</td>
<td>24.7</td>
<td>18.5</td>
<td>7</td>
<td>Sandy Silty Clay (CL-ML)</td>
</tr>
</tbody>
</table>

Arlington Desalter Booster Pump Station and Reservoir  
East End of Sterling Avenue, City of Riverside, California  
For: Albert A. Webb Associates  
Project No. 14-81-137-02  
Drawing No. B-4
Appendix C

Liquefaction and Settlement Analyses
APPENDIX C

LIQUEFACTION AND SETTLEMENT ANALYSES

Cone penetrometer test (CPT) data from CPT-1 and CPT-2 were analyzed to evaluate the liquefaction potential at the site. The CPT logs are presented in Appendix A, *Field Exploration*. The liquefaction analysis was performed using LiquefyPro (Civiltech, 2012). A disaggregated mean earthquake magnitude of M6.7 and peak ground acceleration (PGA) of 0.527g, where g is the acceleration due to gravity, were selected for this analysis. The PGA was calculated using the U.S. Seismic Design Map tool (USGS, 2015b) and is based on the California Building Code (CBSC, 2013) and ASCE Standard 7-10 (ASCE, 2013) Equation 11.8-1. A factor of safety against liquefaction of 1.3 was utilized in accordance with California Geological Society Special Publication 117A: *Guidelines for Evaluating and Mitigating Seismic Hazards in California* (CGS, 2008). A depth to groundwater of 15 feet bgs was used based on historical groundwater data.

The potential dynamic settlement at the site was analyzed under current groundwater conditions, with groundwater deeper than 50 feet bgs, and under historic high groundwater conditions, with groundwater 36 feet bgs. The potential differential settlement over a horizontal distance of 50 feet was calculated based on the difference in potential total settlement at CPT-1 and CPT-2, which were located approximately 43 feet apart.

The results of our analyses are presented on Plates C-1 through C-4, and summarized in the following table.

**Table C-1, Liquefaction Analysis Results**

<table>
<thead>
<tr>
<th>Boring Number</th>
<th>Total Depth Explored (feet)</th>
<th>Current Groundwater (&gt;50' bgs)</th>
<th>Historic Groundwater (36' bgs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Depth Explored (feet)</td>
<td>Potential Total Settlement (inches)</td>
<td>Potential Differential Settlement (inches in 50')</td>
</tr>
<tr>
<td>CPT-1</td>
<td>50.5</td>
<td>0.33</td>
<td>0.21</td>
</tr>
<tr>
<td>CPT-2</td>
<td>46.1</td>
<td>0.15</td>
<td></td>
</tr>
</tbody>
</table>

The analysis indicates that dynamic settlement may occur within the site during a large seismic event. Both liquefaction-induced settlement and dry dynamic settlement may occur, depending on the depth to groundwater at the time of the event.
LIQUEFACTION ANALYSIS
1.1 MG Water Reservoir between Sterling Avenue and Highway 91

Hole No. = CPT 1  Water Depth = 51.5 ft  Surface Elev. = 710
Magnitude = 7  Acceleration = 0.527g

Soil Description

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Unit Weight (g/cc)</th>
<th>Fine %</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>0</td>
<td>fine to medium-grained sand, trace clay, red brown</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>olive</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td>fine-grained sand, olive brown</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td>fine to medium-grained, brown</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td>fine-grained, yellow brown</td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
<td>fine-grained, yellow brown</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Shear Stress Ratio

Factor of Safety

Settlement

S = 0.33 in

Shaded Zone has Liquefaction Potential

CivilTech Corporation
Plate C-1
LIQUEFACTION ANALYSIS
1.1 MG Water Reservoir between Sterling Avenue and Highway 91

Hole No.=CPT-2 Water Depth=50.0 ft Surface Elev.=711 Magnitude=7
Acceleration=0.527g

Soil Description

- fine to medium-grained sand, red brown
- olive
- fine-grained sand, olive brown
- fine to medium-grained, brown
- fine-grained, yellow brown

Shear Stress Ratio

Factor of Safety

Settlement

S = 0.15 in.

Shaded Zone has Liquefaction Potential

CivilTech Corporation

Plate C-2
LIQUEFACTION ANALYSIS

1.1 MG Water Reservoir between Sterling Avenue and

Hole No. = CPT 1  Water Depth = 36 ft  Surface Elev. = 710

Magnitude = 7  Acceleration = 0.527g

Soil Description:
- Fine to medium-grained sand, trace clay, red brown
- Olive
- Fine-grained sand, olive brown
- Fine to medium-grained, brown
- Fine-grained, yellow brown

Shear Stress Ratio:
- CRR
- CSR
- fs

Factor of Safety:
- 0.1
- 5

Settlement:
- S = 0.54 in.

CivilTech Corporation  Plate C-3