APPENDIX D
GEOTECHNICAL FEASIBILITY STUDY REPORT
GEOTECHNICAL FEASIBILITY STUDY REPORT
Proposed Public Restroom, Roadway Improvement and Stormwater Sediment Basin Project
Arroyo Seco Canyon
Pasadena, California

Converse Project No. 13-31-199-01
August 23, 2013

PREPARED FOR
Carollo Engineers, Inc.
199 South Los Robles Avenue, Suite 530
Pasadena, CA 91101
August 23, 2013

Ms. Inge Wiersema
Carollo Engineers, Inc.
199 South Los Robles Avenue, Suite 530
Pasadena, CA 91101

Subject: GEOTEchnICAL FEASIBILITY STUDY REPORT
Proposed Public Restroom, Roadway Improvement, and Stormwater Sediment Basins Project
Arroyo Seco Canyon
Pasadena, California
Converse Project No. 13-31-199-01

Dear Ms. Wiersema:

Converse Consultants (Converse) is pleased to present this Geotechnical Feasibility Study Report for the proposed Public Restroom, Roadway Improvement, and Stormwater Sediment Basins Project located at Arroyo Seco Canyon in Pasadena, California. Our services were performed in accordance with our proposal dated June 10, 2013.

The purpose of this study is to perform preliminary engineering geologic and geotechnical explorations to characterize the project sites, evaluate the feasibility of on-site wastewater treatment system at the planned public restroom sites, and provide geotechnical recommendations for restroom foundations and roadway improvement. Please be advised that our geotechnical recommendations for foundations and roadway improvement can be used for structural design. However, the preliminary percolation study for OWTS will not be sufficient for actual design and submission for the LA County review because the design-level information is not available to us at this time.

We appreciate the opportunity to be of service to the Carollo Engineers, Inc. If you should have any questions, please do not hesitate to contact us at (626) 930-1200.

CONVERSE CONSULTANTS

William H. Chu, P.E., G.E.
Senior Vice President/Principal Engineer

Dist: 4/Addressee
MM/SCL/WHC/amm
PROFESSIONAL CERTIFICATION

This feasibility report for the proposed Improvement, and Stormwater Sediment Basins Project located at Arroyo Seco Canyon in Pasadena, California has been prepared by the staff of Converse under the professional supervision of the individuals whose seals and signatures appear hereon. This feasibility report may require additional geotechnical studies and may not contain sufficient information for design and construction.

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

In the event that changes to the property occur, or additional, relevant information about the property is brought to our attention, the conclusions 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.

Sean C. Lin, P.E., G.E.                              Mark Schluter, P.G., C.E.G.
Senior Engineer                                  Senior Geologist

William H. Chu, G.E.
Principal Engineer, Senior Vice President
EXECUTIVE SUMMARY

The following is the summary of our geotechnical study, findings, 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 project consists of three separate areas within a 2-mile stretch along the Arroyo Seco Canyon in the City of Pasadena, California. The elements that require geotechnical analyses consist of a public restroom with onsite wastewater treatment system (OWTS), roadway improvement and associated retaining wall, and stormwater sediment basins.

- Site No. 1 and Site No. 2C are the potential candidate sites for the planned public restroom and OWTS. The planned roadway improvement is located at Site No. 2A. The planned stormwater sediment basins are located at Site No. 3.

- Fourteen (14) exploratory borings (BH-1 through BH-14) were drilled within the project sites from July 8 to July 12, 2013. The borings were advanced using a limited access rig with 12-inch and 24-inch diameter bucket augers, and truck mounted 8-inch diameter hollow stem auger drill rig to depths ranging from 2.5 to 21 feet below the existing ground surface (bgs).

- Borings BH-2, BH-5 and BH-9 through BH-14 were utilized for percolation tests prior to backfill. Percolation test results are presented in Appendix C, Percolation Testing Data.

- Site No. 1 is acceptable to construct the leach lines as onsite wastewater treatment system in accordance with the Los Angeles County requirements based on our preliminary percolation testing.

- Site No. 1 is located in close proximity of a 50-year floodplain. The Arroyo Seco Canyon is subject to periodic flooding following periods of heavy rainfall. Drilled caissons with grade beam system should be used as the restroom foundation. Flood protection measures are recommended for new structures.

- Roadway improvement at Site No. 2A should be supported by a retaining wall with cast-in-drill-hole pile foundations. The original roadway section was washed away by flooding in the Arroyo Seco Canyon. Flood protection measures are recommended for new structures.
- Site No. 2C is not feasible to construct the onsite wastewater treatment system based on our preliminary percolation testing. The site is underlain by shallow hard bedrock.

- The upper 5 feet of soils within Site No. 3 have high to very high percolation rates. It is our opinion that the percolation rates presented on our table demonstrate the good percolation capacity of the onsite soils without considering fine sediment clogging. For planning or design purposes, it is recommended to consider the lowest percolation rates among the tests because the percolation test holes are located at only a few scattered points over a fairly large area. Fine sediment clogging should be also considered into the design and maintenance plan.

- Based on the slope stability analyses, the proposed new Sediment Basin A is located in the area having 1.5 factor of safety or greater for slope stability, and the slope near the creek has factor of safety greater than 1.25, which exceeds the minimum required factor of safety in common geotechnical practice.

- At all of the project sites studied in this report, the onsite materials will contain large amounts of gravels, cobbles and boulders. Based on our field exploration, the earth materials at the site may be excavated with conventional heavy-duty earth moving and trenching equipment in general. Difficult drilling and excavation conditions will be encountered during construction and should be anticipated and other suitable equipment and methods should be used.
# Geotechnical Feasibility Study Report
## Proposed Public Restroom, Roadway Improvement
### And Stormwater Sediment Basins Project
#### Arroyo Seco Canyon
##### Pasadena, California
##### August 23, 2013

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1.0 INTRODUCTION

The project consists of four separate areas within a 2-mile stretch along the Arroyo Seco Canyon in the City of Pasadena, California. The elements that require geotechnical analyses consist of a public restroom with onsite wastewater treatment system (OWTS), roadway improvement and associated retaining wall, and stormwater sediment basins. An aerial view of the site is illustrated on Drawing No. 1, Overall Project Site Plan.

Based on the information provided to us, Site No. 1 and Site No. 2C are the potential candidate sites for the planned public restroom and OWTS. The planned roadway improvement is located at Site No. 2A. The planned stormwater sediment basins are located at Site No. 3. The enlarged maps for these four sites are shown on Drawing Nos. 2a thru 2d.

The purpose of this study is to perform preliminary engineering geologic and geotechnical explorations to characterize the project sites, evaluate the feasibility of onsite wastewater treatment system at the planned public restroom sites, and provide geotechnical recommendations for restroom foundations and roadway improvement. Please be advised that our geotechnical recommendations for foundations and roadway improvement can be used for structural design. However, the preliminary percolation study for OWTS will not be sufficient for actual design and submission for the LA County review because the design-level information is not available to us at this time.

This report for geologic and geotechnical design parameters for the project described herein and is intended for use solely by Carollo Engineers, Inc. This report should not be used as a bidding document but may be made available to the potential contractors for information on faculty data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

2.0 SITE NO. 1 – 1ST CANDIDATE SITE FOR PUBLIC RESTROOM

2.1 Site Description

Site No. 1 is located at the northernmost part of the project, near the National Forest Ranger’s Station. The planned restroom building site is located at a flat ground on the east side of the Gabrieleno Trail near an existing trail monument at the toe of an approximate 20 feet high ascending 2H:1V slope to a terrace pad.
The planned restroom building site is located on the edge of the Arroyo Seco Canyon floodplain and is subject to potential flooding following periods of heavy rainfall.

The planned onsite wastewater treatment system (seepage pit or leach field) area is located on the upper terrace pad east of the planned restroom and north of the Ranger’s Station. The surface conditions consist of unpaved trail, grass and some mature trees.

The restroom site elevation is about 1233 feet above Mean Sea Level (MSL). The onsite wastewater treatment system site elevation is about 1262 feet MSL. The coordinates for the project site are: North latitude: 34.2102 degrees and West longitude: 118.1713 degrees. The project site is depicted on Drawing No. 2a, Site No. 1, Boring Location Map.

2.2 Scope of Work

The scope of our present study includes site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analysis, and preparation of this report. Details of the tasks are addressed in the following sections:

2.2.1 Project Setup and Site Reconnaissance

A Converse geologist conducted a site reconnaissance on June 20, 2013. The purpose of the reconnaissance was to evaluate site conditions with respect to the location of the borings and drill rig accessibility. The Underground Services Alert (USA) was notified on June 28, 2013 within 14 calendar days prior to field exploration.

2.2.2 Subsurface Exploration and Percolation Testing

Three (3) borings, including one boring (BH-1) within the planned restroom area and two borings (BH-2 and BH-2A) within the planned wastewater treatment system area, were drilled to a maximum depth of 7 feet below existing ground surface on July 8 and 9, 2013. The borings were drilled with a limited access drill rig equipped with 12-inch and 24-inch diameter bucket augers for soil sampling. Each boring was visually logged and sampled at regular depth in tervals and at changes in subsurface soils. The borings were backfilled with soil cuttings. All three borings encountered refusals at shallow depths due to numerous cobbles, and boulders larger than 12 inches in diameter. The boring BH-2A was an additional attempt after refusal at BH-2. The locations of borings are shown on the attached Drawing No. 2a, Site No. 1, Boring Location Map.
California Modified Sampler (Ring samples), and bulk soil samples were obtained for laboratory testing. The bore holes were backfilled and compacted with soil cuttings after the completion of field testing.

Boring BH-2 was utilized for percolation tests prior to backfill. Percolation test procedures and test results are further discussed in Section 2.6.1, Preliminary OWTS Feasibility Evaluation. The raw data of percolation testing is presented in Appendix C, *Percolation Testing Data*.

2.2.3 Laboratory Testing

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

- In situ moisture contents and dry densities (ASTM Standard D2216)
- Grain-Size Analysis (ASTM D422)
- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Direct shear (ASTM Standard D3080)
- Consolidation (ASTM Standard D2435)
- Soil corrosivity tests (Caltrans 643, 422, 417 and 532)

The detailed description of the laboratory test methods and test results are presented in Appendix B, *Laboratory Testing Program*.

2.2.4 Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated with respect to the proposed development. Recommendations for foundations, earthwork, and feasibility evaluation of OWTS are provided.

2.3 Subsurface Conditions

2.3.1 Subsurface Soil Profile of Project Site

Based on our exploratory soil boring (BH-1) at the restroom site, stream deposits (Map symbol: Qg) consisting of primarily light brown silty sand with some cobbles and boulders were encountered to a maximum explored depth of 7 feet below existing ground surface (bgs). The stream deposits are generally moderately dense to very dense.
Based on our exploratory soil borings (BH-2 and BH-2A) at the wastewater treatment system site, older alluvium (Map symbol: Qoa) consisting of primarily brown silty sand with some cobbles and boulders was encountered to a maximum explored depth of 5 feet below existing ground surface (bgs). The older alluvium is generally moderately dense to dense. The detailed descriptions of the borings are presented in Appendix A, Field Exploration.

Based on large amount of cobbles and boulders encountered during our exploratory borings, difficult drilling conditions are expected during construction. Therefore it is our opinion that leach lines are more feasible than seepage pits for the onsite wastewater treatment system.

2.3.2 Groundwater

Groundwater was not encountered in our exploratory borings to a maximum depth of 7 feet. In accordance with the Seismic Hazard Zone Report for the Pasadena Quadrangle (CDMG, 1998), the historic highest groundwater level contours are not defined at this location. However, the restroom site is adjacent to existing creek. Seasonal high groundwater is anticipated to be shallow.

2.3.3 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 geologic characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If during construction, subsurface conditions differ significantly from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

2.4 Faulting and Geologic Hazards

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, landslides, and liquefaction.

2.4.1 Fault Surface Rupture and Active Faults

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. Based on Drawing No. 3, Regional Geologic Map (Dibblee, 1989), a splay of Tujunga Fault is
located at approximate 100 feet south of the project site.

2.4.2 Liquefaction

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface. The site is located within a mapped Seismic Hazard Zone for liquefaction (CDMG, 1998) as shown in Drawing No. 4, Seismic Hazard Zones Map.

Based on the results of our subsurface exploration, very dense gravelly sand with cobbles and boulders was encountered underneath the proposed restroom site, it is our professional opinion that the site is not susceptible to liquefaction and seismically-induced settlement to be negligible.

2.4.3 Landslides

The site is not located within a Seismic Hazard Zone for required investigation for earthquake-induced landsliding (CDMG, 1999). The restroom site is relatively flat and the ascending 2H:1V, 20-foot-high slope at the east is covered by well developed vegetation. Based on our field explorations and gradient of slope, it is our opinion that this ascending slope is considered stable statically and seismically. The proposed restroom building should be set back at least 10 feet away from the toe of slope in accordance with CBC 2010.

2.4.4 Flood Zone

Based on the information provided by Carollo Engineers, the restroom site is located near a 50-year floodplain. The effects of flood should be considered in the design of restroom building. Flood protection measures are recommended for new structures.

2.5 Seismic Analysis

2.5.1 CBC Seismic Design Parameters

Seismic parameters based on the 2010 California Building Code are calculated using the United States Geological Survey computer program Seismic Hazards Curves,
Response Parameters and Design Parameters, Version 5.1.0a. The seismic parameters are presented below.

Table No. 1, 2010 CBC Seismic Parameters For Site No. 1

<table>
<thead>
<tr>
<th>Seismic Parameters</th>
<th>Site Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_S$</td>
<td>D</td>
</tr>
<tr>
<td>2.660g</td>
<td></td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
<td>D</td>
</tr>
<tr>
<td>0.976g</td>
<td></td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(1)), $F_a$</td>
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</tr>
<tr>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Site Coefficient (from Table 1613.5.3(2)), $F_v$</td>
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</tr>
<tr>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$</td>
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<tr>
<td>2.660g</td>
<td></td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{M1}$</td>
<td>D</td>
</tr>
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<tr>
<td>Design Spectral Response Acceleration for short period, $S_{DS}$</td>
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<td>1.773g</td>
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</tr>
<tr>
<td>Design Spectral Response Acceleration for 1-second period, $S_{D1}$</td>
<td>D</td>
</tr>
<tr>
<td>0.976g</td>
<td></td>
</tr>
</tbody>
</table>

2.5.2 Deaggregated Seismic Source Parameters

Based on our analyses utilizing the USGS 2008 NSHM P PSHA Interactive Deaggregation web site, the mean and modal earthquake magnitudes for a return time of 2475 years are calculated to be 6.68 and 7.02, respectively. The earthquake magnitude of 7.02 should be considered for seismic analyses at the project site.

2.6 Conclusions and Recommendations

2.6.1 Preliminary OWTS Feasibility Evaluation

Based on large amounts of cobbles and boulders encountered during our exploratory borings, difficult drilling conditions are expected during construction. Therefore it is our opinion that leach lines are more feasible than seepage pits for the onsite wastewater treatment system.

Boring BH-2 at Site No. 1 was utilized to perform percolation testing on July 8 and 9 to evaluate the feasibility of onsite wastewater treatment system (OWTS). The bored hole was cased using a four-inch diameter perforated PVC casing surrounded with filter gravel pack. Water was added to the bore hole until the water level was at the ground surface and allowed to pre-soak for one day. After pre-soak, the hole was filled with water again to 12 inches above the bottom, and allowed adequate time for the water level to drop. As the water level drops, one inch of drop was recorded. The percolation data was presented in Appendix C, Percolation Testing Data.
In accordance with the LA County guidelines, the size of the dispersal field shall be determined by the Ryon Formula utilizing the slowest elapsed time required for the water to drop from the 5th to the 6th inch. We have performed a preliminary capacity estimation based on our percolation test results as presented below:

Assume:
- A 50' x 50' leach field is planned for 100% design capacity
- Leach line consists of 3-foot wide trench with 1 foot of filter material below the perforated pipe
- Five 50-foot long leach lines are installed within leach field

Ryon Formula: 
\[ A = \frac{(T+6.24) \times C}{58} \]
where
- \( A = \) square feet of leach lines = 3'*50'*5 = 750 ft\(^2\)
- \( T = \) time for the 6th inch of water to drain = 11 minutes

Therefore, the calculated maximum septic tank capacity, \( C = 2523 \) gal

Site No. 1 is acceptable to construct the leach lines near Borings 2 for onsite wastewater treatment system in accordance with the LA County requirements. It should be advised that percolation testing was performed at only one location for the current feasibility study, which is not fully in compliance with the minimum three testing locations required by the LA County. A comprehensive percolation testing program should be conducted once the site is selected for the planned public restroom.

2.6.2  Foundation Recommendations for Public Restroom

Based on the results of our literature review, subsurface exploration, laboratory testing, geotechnical analyses, and understanding of the planned site improvements, it is our opinion that the proposed public restroom is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans, specifications, and are followed during site construction.

The restroom site is located in close proximity of a 50-year floodplain. The effects of base flood elevation should be considered in the design of restroom building. We recommended the restroom building be supported by drilled caissons with grade beam system. Caissons should be at least founded at least 4 feet below lowest adjacent final grade into dense soils and at least 24 inches in diameter. Bearing capacity of caisson can be calculated by an allowable skin friction of 350 psf. The allowable value indicated above is obtained by applying a factor of safety of 2.0 to the ultimate value. The actual reinforcement of caisson should be determined by the structural engineer.
As an alternative, conventional spread footings can be used. Isolated footing should be at least 24 inches square, and continuous footings should be 12 inches wide. Footings should be embedded at least 4 feet below the lowest adjacent grade into dense native soil. Conventional footings with the minimum sizes can be designed for a net allowable bearing pressure of 3,500 psf for dead-plus-live loads.

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.35 may be assumed with normal dead load forces. An allowable passive earth pressure of 350 psf per foot of depth up to a maximum of 3,500 psf may be used. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

The static settlement is anticipated to be less than 0.5 inch. Differential settlement is expected to be up to one-half of the total settlement over a 30-foot span.

The above vertical bearing may be increased by 33% for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33% for lateral loading due to wind or seismic forces.

2.6.3 Slab-on-grade

Slabs-on-grade should be supported on compacted fill and have a minimum thickness of four (4) inches nominal for support of normal ground-floor live loads. Minimum reinforcement for slabs-on-grade should be No. 3 reinforcing bars, spaced at 18 inches on-center each way. The thickness and reinforcement of more heavily-loaded slabs will be dependent upon the anticipated loads and should be designed by a structural engineer. A static modulus of subgrade reaction equal to 150 pounds per square inch per inch may be used in structural design of concrete slabs-on-grade.

It is critical that the exposed subgrade soils should not be allowed to desiccate prior to the slab pour. Care should be taken during concrete placement to avoid slab curling. Slabs should be designed and constructed as promulgated by the ACI and Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, a 10-mil-thick moisture retarder/barrier between the bottom of slab and subgrade that meets the performance criteria of ASTM E 1745 Class A material. Retarder/barrier sheets should be overlapped a minimum of six inches, and should be taped or otherwise sealed per the product specifications.
2.6.4 Earth Pressures for Retaining Walls

The following design values can be used for the proposed retaining walls, if any. The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of soil behind the wall, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following earth pressures are recommended for vertical walls with no hydrostatic pressure.

Table No. 2, Lateral Earth Pressures for Retaining Wall Design

<table>
<thead>
<tr>
<th>Backfill Slope (H:V)</th>
<th>Cantilever Wall (triangular pressure distribution) Equivalent Fluid Pressure (pcf)</th>
<th>Restrained Wall (uniform pressure distribution) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>32</td>
<td>23H</td>
</tr>
<tr>
<td>2:1</td>
<td>45</td>
<td>30H</td>
</tr>
</tbody>
</table>

The recommended lateral pressures assume that the walls are fully back-drained to prevent build-up of hydrostatic pressure. Adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. The drainage system should consist of perforated pipe surrounded by a minimum one (1) square feet per lineal foot of free draining, uniformly graded, ¾-inch washed, crushed aggregate, and wrapped in filter fabric such as Mirafi 140N or equivalent. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, rigid ABS (SDR-35) or PVC A-2000, or equivalent, with perforations placed down. Alternatively, a prefabricated drainage composite system such as the Miradrain G100N or equivalent can be used. The subdrain should be connected to solid pipe outlets, with a maximum outlet spacing of 100 feet.

Walls subjected to surcharge loads located within a distance equal to the height of the wall should be designed for an additional uniform lateral pressure equal to one-third or one-half of the anticipated surcharge load for unrestrained or restrained walls, respectively. These values are applicable for backfill placed between the wall stem and an imaginary plane rising 45 degrees from below the edge (heel) of the wall footings.

Although not anticipated, retaining walls greater than 12 feet should be designed to resist additional earth pressure caused by seismic ground shaking. A seismic earth pressure of 15H (psf), based on an inverted triangular distribution, can be used for design of wall.
2.6.5 Soil Corrosivity Evaluation

Based on our review of soil corrosivity test results (see Appendix B), the pH, chloride content and saturated resistivity are not in the corrosive range to ferrous metal. The soluble sulfate concentration is not in the corrosive range to concrete. Mitigation measures to protect concrete in contact with the soils are not anticipated.

A corrosion engineer may be consulted for appropriate mitigation procedures and construction design, if needed. General considerations for corrosion mitigation measures may include the following:

- Steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed.
- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

2.6.6 Site Drainage

Adequate positive drainage should be provided away from the structure foundations to prevent ponding and to reduce percolation of water into the foundation soils. We recommend that any landscape areas immediately adjacent to the foundation shall be designed sloped away from the foundation with a minimum 2 percent slope gradient for at least 10 feet measured perpendicular to the face of the foundation. Impervious surfaces within 10 feet of the structure foundation shall be sloped a minimum of 1 percent away from the structure.

2.6.7 Earthwork and Site Grading

The earthwork anticipated for the restroom building includes foundation excavations and subgrade preparation. To prepare the subgrade underneath the slab, we recommend scarify the subgrade at least 6 inches, moisture conditioned as needed to near optimum moisture content, and compacted to 90 percent relative compaction for slab support. Deeper removal will be needed if soft soil conditions expose at the excavation bottom.

All engineered fill should be placed on competent, scarified and compacted bottom as evaluated by the geotechnical engineer and in accordance with the recommendations presented in this section. Excavated site soils, free of deleterious materials and rock
particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. Any proposed import fill should be evaluated and approved by Converse prior to import to the site. Import fill material should have an expansion index less than 20.

The onsite materials will contain large amount of gravels, cobbles and boulders. Based on our field exploration, the earth materials at the site may be excavated with conventional heavy-duty earth moving and trenching equipment in general. Difficult drilling and excavation conditions should be also anticipated and other suitable equipment and methods should be used.

Prior to compaction, fill materials should be thoroughly mixed and moisture conditioned within two (2) percent above the optimum moisture content. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D1557 test method. The upper 12 inches of subgrade below pavement areas should be compacted to 95 percent relative compaction.

2.6.8 Expansive Soil

The near surface soils have a “Very Low” expansive potential. Mitigation for expansive soil is not considered necessary.

2.6.9 Pipeline Backfill

Any soft and/or unsuitable material encountered at the pipe invert should be removed and replaced with an adequate bedding material. The pipe subgrade should be level, firm, uniform, free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than two (2) inches in the largest dimension, if any, should be removed from the trench bottom and replaced with compacted materials. 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. The bedding zone is defined as that portion of the pipe trench from four inches below the pipe invert to one foot above the top of pipe, in accordance with Section 306-1.2.1 of the Latest Edition of the Standard Specifications for Public Works Construction (SSPWC).
The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than one (1) inch in maximum dimension in the upper 12 inches of subgrade soils and larger than three (3) inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing 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. No more than 30 percent of the backfill volume should be larger than ¾ inch in the largest dimension.

Trench backfill shall be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method. At least the upper twelve (12) inches of trench underlying pavements should be compacted to at least 95 percent of the laboratory maximum dry density.

Trench backfill shall be compacted by mechanical methods, such as sheepfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within two (2) percent of optimum moisture content and then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent. Observation and field tests should be performed by Converse during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall 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 conditions during cut and/or fill operations. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall 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.

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)
Have a Plasticity Index of 10 or less

Any import fill should be tested and approved by the geotechnical representative prior to delivery to the site.

2.6.10 Temporary Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations may be constructed according to the slope ratios presented in the following table:

Table No. 3, Slope Ratios for Temporary Excavation

<table>
<thead>
<tr>
<th>Maximum Depth of Cut (feet)</th>
<th>Maximum Slope Ratio* (horizontal: vertical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 4</td>
<td>vertical</td>
</tr>
<tr>
<td>4 – 8</td>
<td>1:1</td>
</tr>
<tr>
<td>&gt;8</td>
<td>1.5:1</td>
</tr>
</tbody>
</table>

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

Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand should be constructed at a flatter gradient than presented in the table above. Surfaces exposed in slope excavations should be kept moist but not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within five (5) feet of the unsupported excavation edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be
observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

3.0 SITE NO. 2A – ROADWAY REPAIR SITE

3.1 Site Description

Site No. 2A is located at about 1,000 feet southeast of Site No. 1, just south of the Behner Intake. It is our understanding that the existing asphalt paved roadway (Arroyo Seco Canyon Road/Gabrieleno Trail) was eroded for approximate 145 feet in length along the westerly shoulder during 2009-2010 storms and subsequent canyon floods. K-rails are currently in place along the western edge of the roadway. The northern segment of road erosion scarp is about 4 to 5 feet in vertical height and about 90 feet in length along roadway. The scarp exposed older alluvial soil consisting of brown silty sand with cobbles and boulders. The southern segment of road erosion scarp is about 1 foot in height and about 55 feet in length. The southern scarp exposed soil layer about 6 inches in height underlain by very hard, massive granitic rock.

The site is situated at about 1192 feet MSL. The site coordinates are: North latitude: 34.2075 degrees and West longitude: 118.1681 degrees. The project site is depicted on Drawing No. 2b, Site No. 2A, Boring Location Map.

3.2 Scope of Work

The scope of our present study includes site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analysis, and preparation of this report. Details of the tasks are addressed in the following sections:

3.2.1 Project Setup and Site Reconnaissance

A Converse geologist conducted a site reconnaissance on June 20, 2013. The purpose of the reconnaissance was to evaluate site conditions with respect to the location of the borings and drill rig accessibility. The Underground Services Alert (USA) was notified on June 28, 2013 within 14 calendar days prior to field exploration.

3.2.2 Subsurface Exploration

One boring (BH-3) was drilled to 5 feet below existing roadway ground surface on July 9, 2013. The borings were drilled with a limited access drill rig equipped with 12-inch diameter bucket augers for soil sampling. The boring was visually logged.
and sampled at regular depth intervals and at changes in subsurface soils. The boring was backfilled and compacted with soil cuttings and patched with asphalt after completion of drilling. The boring encountered refusal at shallow depths due to very hard granitic bedrock. The locations of borings are shown on the attached *Drawing No. 2b, Site No. 2A, Boring Location Map.*

California Modified Sampler (Ring samples), and bulk soil samples were obtained for laboratory testing. The bore holes were backfilled and compacted with soil cuttings after the completion of field testing.

3.2.3 Laboratory Testing

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

- *In situ* moisture contents and dry densities (ASTM Standard D2216)
- Grain-Size Analysis (ASTM D422)
- Soil corrosivity tests (Caltrans 643, 422, 417 and 532)

The detailed description of the laboratory test methods and test results are presented in Appendix B, *Laboratory Testing Program.*

3.2.4 Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated with respect to the proposed development. Recommendations for retaining walls, foundations, earthwork, and pavement structural section are provided.

3.3 Subsurface Conditions

3.3.1 Subsurface Soil Profile of Project Site

Based on our field observations, older alluvial soil (Map symbol: Qoa) and granitic bedrock (Map symbol: gr) were exposed on the erosion scars underneath the roadway. Based on our exploratory soil boring (BH-3), the explored location is underlain by about 3-foot thick sandy soils over by granitic bedrock. The upper 1 foot of bedrock was highly weathered. The bedrock below the weather zone is massive and very hard. The detailed descriptions of the borings are presented in Appendix A, Field Exploration.
3.3.2 Groundwater

Groundwater was not encountered in our exploratory borings to a maximum depth of 3.5 feet. In accordance with the Seismic Hazard Zone Report for the Pasadena Quadrangle (CDMG, 1998), the historic highest groundwater level contours are not defined at this location. However, the roadway is adjacent to existing stream channel, seasonal high groundwater is anticipated to be shallow.

3.3.3 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 geologic characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If during construction, subsurface conditions differ significantly from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

3.4 Faulting and Geologic Hazards

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, landslides, and liquefaction.

3.4.1 Fault Surface Rupture and Active Faults

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. Based on Drawing No. 3, Regional Geologic Map (Dibblee, 1989), the trace of Tujunga Fault is located at approximate 1,000 feet south of the project site.

3.4.2 Liquefaction

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level is shallow. The site is not located
within a mapped Seismic Hazard Zone for liquefaction (CDMG, 1998) as shown in Drawing No. 4, Seismic Hazard Zones Map.

Based on the results of our subsurface exploration, the site has shallow granitic bedrock, it is our professional opinion that the site is not susceptible to liquefaction and seismically-induced settlement to be negligible.

3.4.3 Landslides

The Granitic bedrock slope east of the site is located within a Seismic Hazard Zone for required investigation for earthquake-induced landsliding (CDMG, 1999). Based on our field observations of bedrock slope, it is our opinion the slope is stable at its current condition. However, small rock fall hazard should be expected due to steepness of the slope.

3.4.4 Flood Zone

Based on the information provided by Carollo Engineers, the roadway repair site is located within a 5-year floodplain. The roadway is located along the Arroyo Seco Canyon floodplain and is subject to potential flooding following periods of heavy rainfall. The retaining wall should be designed considering the inundation and scour by flood events. Flood protection measures are recommended for new structures. Channel armor may be necessary to protect the retaining wall. Maintenance of roadway will be required after flood events.

3.5 Seismic Analysis

3.5.1 CBC Seismic Design Parameters

Seismic parameters based on the 2010 California Building Code are calculated using the United States Geological Survey computer program Seismic Hazards Curves, Response Parameters and Design Parameters, Version 5.1.0a. The seismic parameters are presented below.
Table No. 4, 2010 CBC Seismic Parameters For Site No. 2A

<table>
<thead>
<tr>
<th>Seismic Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>C</td>
</tr>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_S$</td>
<td>2.636g</td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
<td>0.973g</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>1.3</td>
</tr>
<tr>
<td>MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$</td>
<td>2.636g</td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{M1}$</td>
<td>1.265g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for short period, $S_{0S}$</td>
<td>1.757g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration for 1-second period, $S_{01}$</td>
<td>0.843g</td>
</tr>
</tbody>
</table>

Seismic Design Category | D

3.5.2 Deaggregated Seismic Source Parameters

Based on our analyses utilizing the USGS 2008 NSHM P PSHA Interactive Deaggregation website, the mean and modal earthquake magnitudes for a return time of 2475 years are calculated to be 6.89 and 7.02, respectively. The earthquake magnitude of 7.02 should be considered for seismic analyses at the project site.

3.6 Conclusions and Recommendations

3.6.1 Retaining Wall Recommendations

We recommend a training wall along the west shoulder of the roadway be constructed to support and restore the existing roadway. The training wall is expected to be from 3 to 6 feet in height and should be designed for an active pressure in terms of equivalent fluid pressure of 90 pcf, considering both earth and hydrostatic pressure behind the wall.

Walls subjected to surcharge loads located within a distance equal to the height of the wall should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively. These values are applicable for backfill placed between the wall stem and an imaginary plane rising 45 degrees from below the edge (heel) of the wall footings.

Retaining wall should be supported by cast-in-drilled-hole (CIDH) piles embedded at least 8 feet into older alluvial soils (northern segment) or at least 3 feet into granitic bedrock (southern segment). CIDH piles should be at least 18 inches in diameter with 8 feet center-to-center spacing. Pile bearing capacities can be calculated using allowable
skin friction of 400 psf and 800 psf for older alluvium and competent granitic bedrock, respectively. Uplift capacity can be taken as one-half of the downward pile capacity.

A coefficient of friction of 0.4 can be assumed for concrete in contact with firm older alluvium and bedrock. An allowable passive earth pressure in terms of equivalent fluid pressure of 300 pcf and 500 pcf up to a maximum of 4,000 psf can be used for firm older alluvial soils and bedrock, respectively. If pile spacing is greater than 3 times pile diameter, the passive pressure can be doubled.

For any backfills, excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, are suitable for placement as compacted fill. Any proposed import fill should be evaluated and approved by Converse prior to import to the site. Import fill material should have an expansion index less than 20. Prior to compaction, fill materials should be thoroughly mixed and moisture conditioned within two (2) percent above the optimum moisture content. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer. All fill, if not specified otherwise elsewhere, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D1557 test method.

3.6.2 Flexible Pavement Recommendations

The flexible pavement structural section design recommendations were performed in accordance with the method contained in the CALTRANS Highway Design Manual, Chapter 630 without the factor of safety. No specific traffic study was performed to determine the Traffic Index (TI) for the proposed project. The recommended flexible pavement structural sections for various TI conditions are presented in the following table:

<table>
<thead>
<tr>
<th>Design</th>
<th>Subgrade</th>
<th>Design TI</th>
<th>PAVEMENT STRUCTURAL SECTIONS</th>
<th>FULL AC STRUCTURAL SECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-value</td>
<td>Design TI</td>
<td>AC (inches)</td>
<td>AB (inches)</td>
<td>AC (inches)</td>
</tr>
<tr>
<td>50*</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>3</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

* maximum allowable R-value for design
Actual traffic index and traffic load should be determined by either Civil Engineer or Traffic Engineer. The above pavement sections are recommended as a guideline for basic usage of the indicated TI values, and may not be sufficient for actual traffic loading.

Base material shall conform to requirements for a Class 2 Crushed Aggregate Base (CAB) or equivalent (such as crushed miscellaneous base - CMB) and should be placed in accordance with the requirements of the Standard Specifications for Public Works Construction (SSPWC, Latest Edition). asphaltic materials should conform to Section 203-1, "Paving Asphalt," and should be placed in accordance with Section 302-5, "Asphalt Concrete Pavement," of the SSPWC.

3.6.3 Earthwork

The earthwork anticipated for the roadway improvement includes CIDH pile excavations and subgrade preparation. To prepare the subgrade underneath the pavement, we recommend scarify the subgrade at least 6 inches, moisture conditioned as needed to near optimum moisture content, and compacted to 90 percent relative compaction for slab support. Deeper removal will be needed if soft soil conditions expose at the excavation bottom.

All engineered fill should be placed on competent, scarified and compacted bottom as evaluated by the geotechnical engineer and in accordance with the recommendations presented in this section. Excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. Any proposed import fill should be evaluated and approved by Converse prior to import to the site. Import fill material should have an expansion index less than 20.

The onsite materials will contain large amount of gravel s, cobbles, boulders and very hard granitic bedrock. Based on our field exploration, the earth materials at the site may be excavated with conventional heavy-duty earth moving and trenching equipment in general. Difficult drilling and excavation conditions shall be also anticipated and other suitable equipment and methods should be used.

Prior to compaction, fill materials should be thoroughly mixed and moisture content within two (2) percent above the optimum moisture content. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D1557 test method.
4.0 SITE NO. 2C – 2nd CANDIDATE SITE FOR PUBLIC RESTROOM

4.1 Site Description

Site No. 2C is located at a relative flat area on the east side of Gabrieleno Trail just north of the JPL parking area south of a junction of two stream channels. The preliminarily planned restroom building is located on the northern portion of the site, and the onsite wastewater treatment system (OWTS) is located on the southern portion. The site is about 3 feet higher than the existing road and the surface conditions consist of bushes and some mature trees. An approximate 120 feet high ascending granitic bedrock slope to a building pad is located about 60 feet east of boring BH-5. The slope is steeper than 1H:1V.

The site is situated at about 1162 feet MSL. The site coordinates are: North latitude: 34.2102 degrees and West longitude: 118.1713 degrees. The project site is depicted on Drawing No. 2c, Site No. 2C, Boring Location Map.

4.2 Scope of Work

The scope of our present study includes site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analysis, and preparation of this report. Details of the tasks are addressed in the following sections:

4.2.1 Project Setup and Site Reconnaissance

A Converse geologist conducted a site reconnaissance on June 20, 2013. The purpose of the reconnaissance was to evaluate site conditions with respect to the location of the borings and drill rig accessibility. The Underground Services Alert (USA) was notified on June 28, 2013 within 14 calendar days prior to field exploration.

4.2.2 Subsurface Exploration and Percolation Testing

Three (3) borings, including two borings (BH-4 and BH-4A) within the planned restroom area and one boring (BH-5) within the planned wastewater treatment system (leach lines) area, were drilled a maximum 3.5 feet below existing ground surface on July 10, 2013. The borings were drilled with a limited access drill rig equipped with 12-inch and 24-inch diameter bucket augers for soil sampling. Each boring was visually logged and sampled at changes in subsurface soils. The borings were backfilled with soil cuttings after completion of testing. All three borings encountered refusals at shallow depths due to very hard granitic bedrock. The boring BH-4A was an additional attempt after
refusal at BH-4. The locations of borings are shown on the attached Drawing No. 2c, Site No. 2C, Boring Location Map.

California Modified Sampler (Ring samples), and bulk soil samples were obtained for laboratory testing. The bore holes were backfilled and compacted with soil cuttings after the completion of field testing.

Boring BH-5 was utilized for percolation tests prior to backfill. Percolation test procedures and test results are further discussed in Section 4.6.1, Preliminary OWTS Feasibility Evaluation. The raw data of percolation testing is presented in Appendix C, *Percolation Testing Data*.

### 4.2.3 Laboratory Testing

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

- *In situ* moisture contents and dry densities (ASTM Standard D2216)
- Grain-Size Analysis (ASTM D422)
- Soil corrosivity tests (Caltrans 643, 422, 417 and 532)
- R-value

The detailed description of the laboratory test methods and test results are presented in Appendix B, *Laboratory Testing Program*.

### 4.2.4 Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated with respect to the proposed development. Feasibility evaluation of OWTS are provided.

### 4.3 Subsurface Conditions

#### 4.3.1 Subsurface Soil Profile of Project Site

Based on our exploratory soil borings (BH-4, BH-4A and BH-5) at the site, the site is underlain by thin sandy soils over very hard granitic bedrock (Map symbol: gr). The bedrock was encountered at about 6 inches to 2.5 feet below the ground surface. The detailed descriptions of the borings are presented in Appendix A, Field Exploration.
Although the bedrock is considered as an excellent material to support the building foundation, however, it is impermeable and not suitable for the planned leach field.

4.3.2 Groundwater

Groundwater was not encountered in our exploratory borings to a maximum depth of 2.5 feet. In accordance with the Seismic Hazard Zone Report for the Pasadena Quadrangle (CDMG, 1998), the historic highest groundwater level is reportedly at depth of approximately 20 feet at the site.

4.3.3 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 geologic characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If during construction, subsurface conditions differ significantly from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

4.4 Faulting and Geologic Hazards

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, landslides, and liquefaction.

4.4.1 Fault Surface Rupture and Active Faults

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. Based on Drawing No. 3, Regional Geologic Map (Dibblee, 1989), the trace of Tujunga Fault is located at approximate 80 feet south of the project site.

4.4.2 Liquefaction

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level
and loose sands occur within 50 feet of the ground surface. The site is not located within a mapped Seismic Hazard Zone for liquefaction (CDMG, 1998) as shown in Drawing No. 4, *Seismic Hazard Zones Map*.

Based on the results of our subsurface exploration, the site has shallow Grantic bedrock, it is our professional opinion that the site is not susceptible to liquefaction and seismically-induced settlement to be negligible.

4.4.3 Landslides

The grantic bedrock slope east of the site is located within a Seismic Hazard Zone for required investigation for earthquake-induced landsliding (CDMG, 1999). Based on our field observations of bedrock slope, it is our opinion the slope is stable at its current condition. However, small rock fall hazard should be expected due to steepness of the slope.

4.4.4 Flood Zone

The proposed restroom site is located in the Arroyo Seco Canyon near the confluence to two tributary drainage canyons. The site is subject to potential flooding following major storm events and/or wildfires.

4.5 Seismic Analysis

4.5.1 CBC Seismic Design Parameters

Seismic parameters based on the 2010 California Building Code are calculated using the United States Geological Survey computer program *Seismic Hazards Curves, Response Parameters and Design Parameters, Version 5.1.0a*. The seismic parameters are presented below.

**Table No. 6, 2010 CBC Seismic Parameters For Site No. 2C**

<table>
<thead>
<tr>
<th>Seismic Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class B</td>
<td></td>
</tr>
<tr>
<td>Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_S$</td>
<td>2.656g</td>
</tr>
<tr>
<td>Mapped 1-second Spectral Response Acceleration, $S_1$</td>
<td>0.979g</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>1.0</td>
</tr>
<tr>
<td>MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$</td>
<td>2.656g</td>
</tr>
<tr>
<td>MCE 1-second period Spectral Response Acceleration, $S_{M1}$</td>
<td>0.979g</td>
</tr>
</tbody>
</table>
4.5.2 Deaggregated Seismic Source Parameters

Based on our analyses utilizing the USGS 2008 NSHM P PSHA Interactive Deaggregation website, the mean and modal earthquake magnitudes for a return time of 2475 years are calculated to be 6.89 and 7.02, respectively. The earthquake magnitude of 7.02 should be considered for seismic analyses at the project site.

4.6 Conclusions and Recommendations

4.6.1 Preliminary OWTS Feasibility Evaluation

The boring BH-5 was utilized to perform percolation tests on July 10 and 11, 2013 to evaluate the feasibility of onsite wastewater treatment system. Water was added to the bore hole until the water level was at the ground surface on July 10, 2013 and allowed to pre-soak for one day. Based on our observations on July 11, 2013, water still remained in the borehole after the one-day pre-soak. In accordance with the LA County guidelines, Site No. 2C is not feasible for the construction of onsite wastewater treatment system. The site is underlain by shallow, hard bedrock with little permeability.

Since this site is not suitable for the public restroom, further geotechnical recommendation is not provided.

5.0 SITE NO. 3 – STORMWATER SEDIMENT BASIN SITE

5.1 Site Description

Site No. 3 is located at the southernmost part of the project, including the existing JPL parking area and existing sediment basins. The surface conditions consist of asphalt concrete pavement and unpaved ground.

It is understood that the southern portion of the existing JPL parking area will be demolished for the future sediment basin expansion. The construction of basins will consist of excavating approximate 1 foot below the existing grade and constructing about 2-foot-high berms around the basins. Although not finalized, the conceptual plan provided by Carollo is illustrated on Drawing No. 5, Site No. 3, Conceptual Sediment Basin Plan for reference.
5.2 Scope of Work

The scope of our present study includes site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analysis, and preparation of this report. Details of the tasks are addressed in the following sections:

5.2.1 Project Setup and Site Reconnaissance

A Converse geologist conducted a site reconnaissance on June 20, 2013. The purpose of the reconnaissance was to evaluate site conditions with respect to the location of the borings and drill rig accessibility. The Underground Services Alert (USA) was notified on June 28, 2013 within 14 calendar days prior to field exploration.

5.2.2 Subsurface Exploration and Percolation Testing

A total of nine (9) borings were drilled, including 3 borings (BH-6 through BH-8) to a maximum depth of 21 feet near the slopes, 3 borings (BH-9 through BH-11) to about 5 feet within the existing paved parking lot, and 3 borings (BH-12 through BH-14) to about 5 feet within the existing sediment basins. The locations of borings are shown on the attached Drawing No. 2d, Site No. 3, Boring Location Map.

The borings were drilled with a truck-mounted drill rig equipped with 8-inch diameter hollow-stem auger and a 4-inch diameter hand auger for soil sampling and/or percolation testing. Each boring will be visually logged and sampled at regular depth intervals and at changes in subsurface soils. The borings will be backfilled with soil cuttings and patched with asphalt where needed.

During our drilling at Boring BH-7 location, the driller hit a 16-inch water main pipe on July 12, 2013. Prior to using the drill rig, the driller attempted to hand auger the upper 5 feet below ground surface (bgs). However, the hand augering encountered refusal due to big cobbles at about 3 feet bgs. The driller started to use the drill rig to drill the boring, and then hit the water main at about 4 to 5 feet bgs.

This incident created a cavity on the existing roadway and erosion scarps on both sides of roadway. Based on our visual observations, the cavity was about maximum 15 feet in width and 10 feet in depth. The erosion scarp on the east side of roadway created a near vertical ascending slope up to approximately 20 feet in height. This slope exposed terrane deposits consisting of silty sand with cobbles and boulders. Some cobbles and boulders were exposed on the slope that may cause potential rockfall hazard.
Converse revisited the site on July 23, 2013. Based on our observations, the cavity on the road had been backfilled with slurry to about 80% of road width and the road was accessible to the recreational traffic.

5.2.3 Laboratory Testing

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

- *In situ* moisture contents and dry densities (ASTM Standard D2216)
- Grain-Size Analysis (ASTM D422)
- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Direct shear (ASTM Standard D3080)
- Consolidation (ASTM Standard D2435)
- Soil corrosivity tests (Caltrans 643, 422, 417 and 532)

The detailed description of the laboratory test methods and test results are presented in Appendix B, Laboratory Testing Program.

5.2.4 Analyses and Report

Percolation test results and slope stability were analyzed and evaluated with respect to the proposed development. Preliminary earthwork recommendations are presented.

5.3 Subsurface Conditions

5.3.1 Subsurface Soil Profile of Project Site

Based on our exploratory soil borings, stream deposits (Map symbol: Qg) and terrace deposits (Map symbol: Qof) both consisting of primarily light brown silty sand with numerous cobbles and boulders were encountered to a maximum explored depth of 21 feet below existing ground surface (bgs). The stream deposits and terrace deposits are both moderately dense to very dense. The detailed descriptions of the borings are presented in Appendix A, Field Exploration.

Drawing No. 6a and 6b are prepared to illustrate the geologic cross sections of A-A' and B-B' for slope stability analyses.
EXPLANATION
Qg - STREAM DEPOSIT
TD - TOTAL DEPTH

GEOLOGIC CROSS SECTION A-A''
5.3.2 **Groundwater**

Groundwater was not encountered in our exploratory borings to a maximum depth of 21 feet. In accordance with the Seismic Hazard Zone Report for the Pasadena Quadrangle (CDMG, 1998), the historic highest groundwater level is reportedly at depths of approximately 20 feet.

5.3.3 **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 geologic characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If during construction, subsurface conditions differ significantly from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

5.4 **Percolation Testing Results**

The borings BH-9 through BH-14 were utilized to perform percolation tests on July 11 and 12 to evaluate the percolation rates for the design of future sediment basins. Tests were performed using the falling head test method in accordance with Los Angeles County “Low Impact Development Best Management Practice Guideline for Design, Investigation, and Reporting”

The bored hole was cased using a two-inch diameter perforated PVC casing surrounded with filter gravel pack. Water was added to the bore hole until the water level was at the ground surface and allowed to pre-soak for at least 2 hours. After pre-soak, water was added to the bore hole until the water level was at the ground surface. The water level was measured to the nearest 1/100-foot and recorded every 10-minute interval. The results of the percolation tests are tabulated below and in Appendix C, Percolation Testing Data.

**Table No. 7, Percolation Test Results at Site No. 3**

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Location</th>
<th>Depth of Test Hole* (feet)</th>
<th>Lowest Percolation Rate (inches/hour)</th>
<th>Average Percolation Rate (inches/hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-9</td>
<td>JPL Parking</td>
<td>5.0</td>
<td>6.58</td>
<td>7.87</td>
</tr>
<tr>
<td>BH-10</td>
<td>JPL Parking</td>
<td>5.0</td>
<td>11.52</td>
<td>15.90</td>
</tr>
</tbody>
</table>
The upper 5 feet of soils within Site No. 3 have high to very high percolation rates. Converse has reviewed a literature entitled “Seepage, Drainage and Flow Nets” by Cedergren (1989) to verify the percolation results and provide our opinions to the high percolation rates. The followings are some reasons to explain the high percolation rates:

- Young stream deposits from the river channel naturally have higher permeability than other alluvial soils. These sediments are generally loose and unconsolidated.
- Percolation test holes did not have fine sediments.
- Clean water was used for percolation tests.
- The soils surrounding the test holes (especially near surface soil) may be disturbed or loosened during drilling.
- The higher percolation rates near surface increased the average rates.

The soils encountered in the JPL parking lot and existing sediment basins are “Stream Deposits” from the Arroyo Seco Canyon, consisting of primarily gravelly sands with cobbles and boulders, which are excellent permeable materials. The typical permeability rates of gravelly sands range from 2.8 to 280 ft/day, however, the percent of fine sediments accumulated in soil will reduce the permeability down to 0.2 to 3 ft/day according to Cedergren (1989). Our test results are within the typical percolation rate range.

It is our opinion that the percolation rates presented on our table demonstrate the good percolation capacity of the onsite soils without considering fine sediment clogging. For planning or design purposes, it is recommended consider the lowest percolation rates among the tests because the percolation test holes are located at only a few scattered points over a fairly large area. Fine sediment clogging should be also considered into the design and maintenance plan for the stormwater spreading and recharge of the basins.
5.5  **Slope Stability Analysis**

Geologic cross sections A-A’ and B-B’ were analyzed for gross static slope stability by using a computer program SLOPE/W which utilizes various limiting equilibrium methods, including the ordinary slice, Bishop’s, Jabu’s, and Spencer’s method.

To evaluate the influence of the future sediment basin expansion, groundwater level is assumed to be (1) at 20 feet below the ground surface, and (2) groundwater at ground surface, to simulate dry and saturated soil conditions underneath the sediment basins, respectively. The detailed analyses results are presented in Appendix D, Slope Stability Analyses. The summary of slope stability results are presented in the following table:

<table>
<thead>
<tr>
<th>Cross Section</th>
<th>Groundwater Depth (feet)</th>
<th>Minimum Factor of Safety (Static)</th>
<th>Plate No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A – A’</td>
<td>20</td>
<td>1.496</td>
<td>S-A1</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>1.292</td>
<td>S-A2</td>
</tr>
<tr>
<td>B – B’</td>
<td>20</td>
<td>1.487</td>
<td>S-B1</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>1.487</td>
<td>S-B2</td>
</tr>
</tbody>
</table>

As shown in the table above, the slope stability near the existing stream channel (Cross Section A-A’) has a factor of safety greater than 1.496 at current condition. When the proposed new sediment basin with groundwater at surface is assumed, the factor of safety reduces to 1.292. The factors of safety in both cases remain above 1.0, which indicates this slope does not expose immediate instability. In general practices, most structures constructed on slopes require a minimum factor of safety of 1.5, and temporary grading requires a minimum factor of safety of 1.25. The proposed new Sediment Basin A is located in the area having 1.5 factor of safety, and the slope near the stream channel has factor of safety greater than 1.25 which meet the minimum requirements. It should be advised that this slope will be eroded by the stream from time to time, and the slope stability will be also changed by the slope profile.

The slope stability of the easterly slope (Cross Section B-B’) has a factor of safety of 1.487 at current condition. When the proposed new sediment basin with groundwater at surface is assumed, the factor of safety remains the same, which suggests the saturation of subsurface soil will not impact the current slope stability.
5.6 Earthwork Recommendations

5.6.1 Earthwork and Site Grading

To construct the planned sediment basins, the anticipated earthwork and site grading includes excavations of basins and constructing berms. All berms should be constructed with a slope gradient less than 2H:1V.

All engineered fill should be placed on competent, scarified and compacted bottom as evaluated by the geotechnical engineer and in accordance with the recommendations presented in this section. Excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. Any proposed import fill should be evaluated and approved by Converse prior to import to the site. Import fill material should have an expansion index less than 20.

The onsite materials will contain large amount of gravels, cobbles and boulders. Based on our field exploration, the earth materials at the site may be excavated with conventional heavy-duty earth moving and trenching equipment in general. Difficult drilling and excavation conditions should be also anticipated and other suitable equipment and methods should be used.

Prior to compaction, fill materials should be thoroughly mixed and moisture conditioned within two (2) percent above the optimum moisture content. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D1557 test method. The upper 12 inches of subgrade below pavement areas should be compacted to 95 percent relative compaction.

5.6.2 Expansive Soil

The near surface soils have a “Very Low” expansive potential. Mitigation for expansive soil is not considered necessary.

5.6.3 Pipeline Backfill

Any soft and/or unsuitable material encountered at the pipe invert should be removed and replaced with an adequate bedding material. The pipe subgrade should be level, firm, uniform, free of loose materials and properly graded to provide uniform bearing.
and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than two (2) inches in the largest dimension, if any, should be removed from the trench bottom and replaced with compacted materials. 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. The bedding zone is defined as that portion of the pipe trench from four inches below the pipe invert to one foot above the top of pipe, in accordance with Section 306-1.2.1 of the Latest Edition of the Standard Specifications for Public Works Construction (SSPWC).

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than one (1) inch in maximum dimension in the upper 12 inches of subgrade soils and larger than three (3) inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing 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. No more than 30 percent of the backfill volume should be larger than $\frac{3}{4}$ inch in the largest dimension.

Trench backfill shall be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method. At least the upper twelve (12) inches of trench underlying pavements should be compacted to at least 95 percent of the laboratory maximum dry density.

Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill material shall be brought to within two (2) percent of optimum moisture content and then placed in horizontal layers if the expansion index is less than or equal to 30. Should the expansion index be greater than 30, backfill materials shall be brought to approximately 2 percent above optimum moisture content. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent. Observation and field tests should be performed by Converse during construction to confirm that the required degree of
compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall 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 conditions during cut and/or fill operations. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall 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.

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)
- Have a Plasticity Index of 10 or less

Any import fill should be tested and approved by the geotechnical representative prior to delivery to the site.

5.6.4 Temporary Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations may be constructed according to the slope ratios presented in the following table:

Table No. 9. Slope Ratios for Temporary Excavation at Site No. 3

<table>
<thead>
<tr>
<th>Maximum Depth of Cut (feet)</th>
<th>Maximum Slope Ratio* (horizontal: vertical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 4</td>
<td>vertical</td>
</tr>
<tr>
<td>4 – 8</td>
<td>1:1</td>
</tr>
<tr>
<td>&gt;8</td>
<td>1.5:1</td>
</tr>
</tbody>
</table>

*Slope ratio assumed to be uniform from top to toe of slope.
Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand should be constructed at a flatter gradient than presented in the table above. Surfaces exposed in slope excavations should be kept moist but not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within five (5) feet of the unsupported excavation edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

6.0 SLOPE REPAIR RECOMMENDATIONS

The erosion scarps caused by the broken water pipe incident on both sides of the existing roadway (Arroyo Seco Canyon Road) will require repairs to support the roadway.

For the erosion scar below the roadway, we recommend soldier piles with wood lagging be used as the retaining system to support the roadway. The soldier piles should be embedded at least 8 feet below the lowest adjacent grade. The actual embedment depth should be determined by the design engineer. The pile capacity can be calculated using allowable skin friction of 350 psf. An allowable passive resistance in the terms of equivalent fluid pressure of 300 pcf can be used for lateral design. Passive resistance can be doubled if pile spacing is greater than 3 time diameter. The retaining wall should have proper subdrain or weepholes. Loose soils and debris washed down to the parking lot fence should be removed and cleared. Portions of the fence may have to be repaired and/or replaced.

For the erosion scarp on the ascending slope east of the roadway, we recommend a buttress system be constructed to increase the slope stability and reduce rock fall hazard. Gabions constructed to a 1H:1V slope ratio to a minimum height of 8 feet can be used as the buttress system, any oversized rocks with potential rock fall hazard should be removed from the slope face.
7.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

This report has been partially prepared to aid in the foundation plans and specifications, and to assist the architect, civil and structural engineers in the design of the proposed structures. It is recommended that this office be provided an opportunity to review final design drawings and specifications to verify that the recommendations of this report have been properly implemented.

Recommendations presented herein are based upon the assumption that adequate earthwork monitoring will be provided by the geotechnical engineer. Footing excavations should be observed by the geotechnical engineer prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials. Trench backfill should be placed and compacted with observation and field density testing provided by this office.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor’s operations, and cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any recommended actions presented herein to be unsafe.

8.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory studies, combined with an interpolation and extrapolation of soil conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

Design recommendations given in this report are based on the assumption that the earthwork and site grading 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 final site grading and 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.
9.0 REFERENCES


CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA DIVISION OF MINES AND GEOLOGY, Seismic Hazard Zones Map of the Pasadena 7.5 Minute Quadrangle, Los Angeles County, March, 1999.


APPENDIX A

FIELD EXPLORATION
APPENDIX A

FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface conditions were noted, and the approximate locations of the boring were determined. The exploratory borings were approximately located using existing boundary and other features as a guide and should be considered accurate only to the degree implied by the method used. The various field study methods performed are discussed below.

Exploratory Borings

Fourteen (14) exploratory borings (BH-1 through BH-14) were drilled within the project sites from July 8 to July 12, 2013. The borings were advanced using a limited access rig with 12-inch and 24-inch diameter bucket augers, and truck mounted 8-inch diameter hollow stem auger drill rig to depths ranging from 2.5 to 21 feet below the existing ground surface (bgs). Every boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils.

California Modified Sampler (Ring samples), Standard Penetration Test samples, and bulk soil samples were obtained for laboratory testing. Standard Penetration Tests (SPTs) were performed in selected borings at selected intervals using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The bore holes were backfilled and compacted with soil cuttings by reverse spinning of the auger following the completion of drilling and patched with asphalt.

Borings BH-2, BH-5, and BH-9 through BH-14 were utilized for percolation tests prior to backfill. Percolation test results are presented in Appendix C, Percolation Testing Data.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. Changes in material conditions that occur between driven samples are indicated in the logs at the top of the next drive sample. A key to soil symbols and terms is presented as Drawing No. A-1, Soil Classification Chart. The log of the exploratory boring is presented in Drawing Nos. A-2 through A-15, Log of Borings.
### SOIL CLASSIFICATION CHART

<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>SYMBOLS</th>
<th>TYPICAL DESCRITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE GRAINED SOILS</td>
<td>GRAVELS AND GRAVELLY SOILS</td>
<td>CLEAN GRAVELS (LITTLE OR NO FINES)</td>
</tr>
<tr>
<td></td>
<td>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE</td>
<td>CLEAN GRAVELS (LITTLE OR NO FINES)</td>
</tr>
<tr>
<td></td>
<td>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE</td>
<td>GRAVELS WITH FINES (APPROXIMATE AMOUNT OF FINES)</td>
</tr>
<tr>
<td></td>
<td>SAND AND SANDY SOILS</td>
<td>CLEAN SANDS (LITTLE OR NO FINES)</td>
</tr>
<tr>
<td></td>
<td>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE</td>
<td>SANDS WITH FINES (APPROXIMATE AMOUNT OF FINES)</td>
</tr>
<tr>
<td></td>
<td>FINE GRAINED SOILS</td>
<td>SILTS AND CLAYS LIQUID LIMIT LESS THAN 50</td>
</tr>
<tr>
<td></td>
<td>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE</td>
<td>SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50</td>
</tr>
<tr>
<td></td>
<td>HIGHLY ORGANIC SOILS</td>
<td>PT</td>
</tr>
</tbody>
</table>

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

### BORING LOG SYMBOLS

#### SAMPLE TYPE
- STANDARD PENETRATION TEST
- DRIVE SAMPLE
- DRIVE SAMPLE
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

#### LABORATORY TESTING ABBREVIATIONS

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>STRENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Results shown in Appendix B)</td>
<td>p</td>
</tr>
<tr>
<td>Pocket Penetrometer</td>
<td>p</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>ds</td>
</tr>
<tr>
<td>Direct Shear (single point)</td>
<td>ds'</td>
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<tr>
<td>Unconfined Compression</td>
<td>uc</td>
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<tr>
<td>Triaxial Compression</td>
<td>tx</td>
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<td>Vane Shear</td>
<td>vs</td>
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<tr>
<td>Consolidation</td>
<td>c</td>
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<tr>
<td>Collapse Test</td>
<td>col</td>
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<tr>
<td>Resilience (R) Value</td>
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<tr>
<td>Chemical Analysis</td>
<td>ca</td>
</tr>
<tr>
<td>Electrical Resistivity</td>
<td>er</td>
</tr>
</tbody>
</table>

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

Converse Consultants

Project Name: ARROYO SECO CANYON PROJECT
Location: PASADENA, CALIFORNIA

Project No.: 13-31-199-01
Drawing No.: A-1
**Log of Boring No. BH-1**

Dates Drilled: 7/9/2013  
Logged by: MDR  
Checked By: SCL  
Equipment: 12" AUGER BUCKET  
Driving Weight and Drop: 866 lb Kelly Bar / 30"  
Ground Surface Elevation (ft): N/A  
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.

**STREAM DEPOSIT (Qg):**
- GRAVELLY SAND WITH SILT (SP-SM): fine to medium-grained, some cobbles up to 12" in maximum dimension, light brown.
- fine to coarse-grained, some gravel's up to 3" in maximum dimension, light brown, cobbles, boulders.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>BLOWS/FOOT</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
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<tbody>
<tr>
<td>5</td>
<td></td>
<td>DRIVE</td>
<td>BULK</td>
<td></td>
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<td></td>
<td>32</td>
<td>12</td>
<td>117</td>
</tr>
</tbody>
</table>


**SCALE: 1"=5' (H=V)**  
**SKETCH**

---

Converse Consultants  
ARROYO SECO CANYON PROJECT  
PASADENA, CALIFORNIA  
Project Name: ARROYO SECO CANYON PROJECT  
Project No.: 13-31-199-01  
Drawing No.: A-2
Log of Boring No. BH-2

Dates Drilled: 7/8/2013
Logged by: MDR
Checked By: SCL
Equipment: 12" & 24" AUGER BUCKET
Driving Weight and Drop: 866 lb Kelly Bar / 30"
Ground Surface Elevation (ft): N/A
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.

OLDER ALLUVIUM (Qoa):
SILTY SAND TO SANDY SILT (SM/ML): fine to coarse-grained, some gravels up to 3" in maximum dimension, brown.
-at 3' encountered cobbles, changed to 12" diameter drill bit, difficult drilling with rocks.

End of boring at 5 feet due to refusal of cobbles
Groundwater not encountered during drilling.
Borehole utilized for percolation testing
Borehole backfilled with soil cuttings after completion of percolation testing on 7-9-13.

SCALE: 1"=5' (H=V)
Total Depth=5'

Converse Consultants
Project Name
ARROYO SECO CANYON PROJECT
PASADENA, CALIFORNIA
Project No. 13-31-199-01
Drawing No. A-3a
Log of Boring No. BH-2A

Dates Drilled: 7/9/2013
Logged by: MDR
Checked By: SCL
Equipment: 12" AUGER BUCKET
Driving Weight and Drop: 866 lb Kelly Bar / 30"
Ground Surface Elevation (ft): N/A
Depth to Water (ft): NOT ENCOUNTERED

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

**OLDER ALLUVIUM (Qoa):**
Silty Sand (SM): fine to coarse-grained, little gravels up to 3" in maximum dimension, few cobbles up to 2' in maximum dimension, brown.

End of boring at 3.5 feet due to refusal of bedrock
Groundwater not encountered during drilling.
Borehole utilized for percolation testing
Borehole backfilled with soil cuttings after completion of drilling on 7-8-13.

**SCALE: 1"=5' (H=V)**

**SKETCH**

```
  0
  \   \  SM
  \    \  |  Boulder
  \     \  |
  \      \ | Total Depth=3.5'
  \   \   |
  \  \   |
  \   \ |
  \    \|
  \     5|
  \      |
  \     10|
```

---

Converse Consultants
Project Name: ARROYO SECO CANYON PROJECT
Pasadena, California
Project No. 13-31-199-01
Drawing No. A-3b
**Log of Boring No. BH-3**

**Dates Drilled:** 7/9/13  
**Logged by:** MDR  
**Checked By:** SCL

**Equipment:** 12" AUGER BUCKET  
**Driving Weight and Drop:** 866 lb Kelly Bar / 30"

**Ground Surface Elevation (ft):** N/A  
**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.

**3" ASPHALT WITH NO BASE**

**SILTY SAND(SM):** fine-grained, gravels up to 2" in maximum dimension, brown.

**GRANITIC BEDROCK (gr):** weathered at upper 1 ft, very hard, gray, drilling refusal, hard, intact.

End of boring at 5 feet due to refusal of cobbles  
Groundwater not encountered during drilling.  
Borehole utilized for percolation testing  
Borehole backfilled with soil cuttings after completion of drilling on 7-9-13.

---

**SCALE: 1"=5' (H=V)**

**SKETCH**

- Asphalt
- SM
- gr

Total Depth=5'

---

Converse Consultants

**Project Name:** ARROYO SECO CANYON PROJECT  
**Project No.:** 13-31-199-01  
**Drawing No.:** A-4
**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.

- **Silty Sand (SM):** medium to coarse-grained, some gravels up to 3" in maximum dimension, light brown.
- **Granitic Bedrock (gr):** medium weathered at upper 1 ft, moderately to very hard, gray.

End of boring at 3.5 feet due to refusal from bedrock. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings on 7-10-13.

---

**Sketch**

Scale: 1" = 5' (H=V)

- SM
- gr

Total Depth = 3.5'
Log of Boring No. BH-4

Dates Drilled: 7/10/2013
Logged by: MDR
Checked By: SCL
Equipment: 24" AUGER BJCKET
Driving Weight and Drop: 866 lb Kelly Bar / 30"
Ground Surface Elevation (ft): N/A
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></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>
</tr>
</tbody>
</table>

**SILTY SAND (SM):** medium to coarse-grained, some gravels up to 3" in maximum dimension, few cobbles up to 8" in maximum dimension, light brown.

**GRANITIC BEDROCK (gr):** medium weathered at upper 1 ft, moderately hard to very hard, gray.
End of boring at 2.5 feet due to refusal from bedrock.
Groundwater not encountered during drilling.
Borehole backfilled with soil cuttings on 7-10-13.

<table>
<thead>
<tr>
<th>SCALE: 1&quot;=5' (H=V)</th>
<th>SKETCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>SM</td>
</tr>
<tr>
<td>10</td>
<td>gr</td>
</tr>
<tr>
<td>Total Depth=2.5'</td>
<td></td>
</tr>
</tbody>
</table>

Converse Consultants
Project Name: ARROYO SECO CANYON PROJECT
Project No.: 13-31-199-01
Drawing No.: A-5a

PASADENA, CALIFORNIA
Log of Boring No. BH-5

Dates Drilled: 7/10/2013
Logged by: MDR
Checked By: SCL
Equipment: 12" & 24" AUGER BUCKET
Driving Weight and Drop: 866 lb Kelly Bar / 30"
Ground Surface Elevation (ft): N/A
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.

GRAVELLY SAND WITH SILT (SP-SM): fine to coarse-grained, some gravels up to 3" in maximum dimension, little cobbles up to 8" in maximum dimension, light brown/gray brown.

GRANITIC BEDROCK (gr): very hard, moderately fractured, slightly weathered, gray.
End of boring at 5 feet due to refusal of bedrock
Groundwater not encountered during drilling.
Borehole utilized for percolation testing
Borehole backfilled with soil cuttings after completion of percolation testing on 7-11-13.

SCALE: 1"=5' (H=V)

SKETCH

Total Depth=3.5'

Converse Consultants
Project Name
ARROYO SECQ CANYON PROJECT
PASADENA, CALIFORNIA

Project No. 13-31-199-01
Drawing No. A-6
### Log of Boring No. BH-6

**Dates Drilled:** 7/11/2013  
Logged by: JR  
Checked By: SCL

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

**Ground Surface Elevation (ft):** N/A  
**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>DRIVE</th>
<th>BULK</th>
<th>BLOWSSIF</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5&quot;</td>
<td>ASPHALT OVER 7&quot; BASE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>STREAM DEPOSIT (Qg):</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SILTY SAND (SM): fine-grained, some gravel up to 2&quot; in maximum dimension, brown.</td>
<td></td>
<td></td>
<td></td>
<td>7/11/12 3</td>
<td>117</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-some gravel up to 2&quot; in maximum dimension, less silt, yellow brown</td>
<td></td>
<td></td>
<td></td>
<td>13/27/26 1</td>
<td>119</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-fine-grained, trace gravel up to 3&quot; in maximum dimension, orange brown</td>
<td></td>
<td></td>
<td></td>
<td>16/15/15 9</td>
<td>100</td>
<td>ds</td>
</tr>
<tr>
<td></td>
<td>-cobble layer, possible boulders</td>
<td></td>
<td></td>
<td></td>
<td>18/28/50(5&quot;) 1</td>
<td>129</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>SAND WITH SILT (SP-SM): fine-grained, some gravel up to 2&quot; in maximum dimension.</td>
<td></td>
<td></td>
<td></td>
<td>50(6&quot;)</td>
<td>dist.</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>-cobble layer, possible boulders</td>
<td></td>
<td></td>
<td></td>
<td>50(3&quot;)</td>
<td>dist.</td>
<td></td>
</tr>
</tbody>
</table>

Log of Boring No. BH-7

Dates Drilled: 7/12/2013  Logged by: JR  Checked By: SCL
Equipment: 8" HOLLOW STEM AUGER  Driving Weight and Drop:
Ground Surface Elevation (ft): N/A  Depth to Water (ft): NOT ENCOUNTRED

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DRIVE</td>
<td>BULK</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.

4" ASPHALT WITH NO BASE

**FILL (AF):**
GRAVELLY SAND WITH SILT(SP-SM); fine to coarse-grained, some gravels up to 3" in maximum dimension, gray brown.

End of boring at 5 feet due to encounter of water main pipe.
The road erosion and excavation created by the broken pipe and repair were backfilled with cement slurry.

Converse Consultants
ARROYO SECO CANYON PROJECT
PASADENA, CALIFORNIA

Project Name
Project No. 13-31-199-01  Drawing No. A-8

| PROJECT ID: 13-31-199-01, Template: LOG |
Log of Boring No. BH-8

Dates Drilled: 7/12/2013  Logged by: JR  Checked By: SCL
Equipment: 8" HOLLOW STEM AUGER  Driving Weight and Drop: 140 lbs / 30 in
Ground Surface Elevation (ft): N/A  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>FILL (AF): SILTY SAND (SM): fine-grained, some gravels up to 3&quot; in maximum dimension, light brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>STREAM DEPOSIT (Qg): GRAVELLY SAND (SP): fine to coarse-grained, gravels up to 3&quot; in maximum dimension, light brown.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- cobble layer, possible boulders</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>SAND WITH SILT (SP-SM): fine to coarse-grained, gravels up to 2&quot; in maximum dimension.</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>- cobble layer, possible boulders</td>
</tr>
</tbody>
</table>

End of boring at 19 feet due to refusal of cobbles. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings on 7-12-13.
Log of Boring No. BH-9

Dates Drilled: 7/11/2013  Logged by: JR  Checked By: SCL
Equipment: 8" HOLLOW STEM AUGER  Driving Weight and Drop: 140 lbs / 30 in
Ground Surface Elevation (ft): N/A  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>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWSF/T</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
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</thead>
<tbody>
<tr>
<td>5</td>
<td>5&quot; ASPHALT OVER 8&quot; BASE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>STREAM DEPOSIT (Qg):</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SAND WITH SILT (SP-SM): fine to coarse-grained, some</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>gravels up to 3&quot; in maximum dimension, gray brown.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>GRAVELLY SAND (SP): fine-grained, light yellow brown.</td>
<td>27/43/50(4&quot;)</td>
<td>2</td>
<td>124</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 6.5 feet.
Groundwater not encountered during drilling.
Borehole utilized for percolation testing.
Borehole backfilled with soil cuttings and patched with asphalt after completion of percolation test on 7-11-13.
# Log of Boring No. BH-10

### Dates Drilled: 7/11/2013

### Logged by: JR

### Checked By: SCL

### Equipment: 8" HOLLOW STEM AUGER

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

### Ground Surface Elevation (ft): N/A

### 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.

### 4" ASPHALT WITH NO BASE

**STREAM DEPOSIT (Qs):**

- GRAVELLY SAND WITH SILT (SP-SM): fine to coarse-grained, some gravels up to 3" in maximum dimension, gray brown.
- Little gravels up to 3" in maximum dimension, few silt

---

**SAMPLES**

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

**DRIVE**

<table>
<thead>
<tr>
<th>Date</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
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</thead>
<tbody>
<tr>
<td>9/23/21</td>
<td>2</td>
<td>137</td>
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</tbody>
</table>

**BULK**

<table>
<thead>
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<th>Date</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
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</thead>
<tbody>
<tr>
<td>12/15/17</td>
<td>4</td>
<td>100</td>
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### SUMMARY OF SUBSURFACE CONDITIONS

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**4.5" ASPHALT WITH NO BASE**

- **STREAM DEPOSIT (Qg):** Silty sand (SM): fine-grained, some gravels up to 2.5" in maximum dimension, olive brown.

**SAND WITH SILT (SP-SM):** Fine to medium-grained, brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>DRIVE</th>
<th>BULK</th>
<th>BLOWS/FT</th>
<th>DRY UNIT W.T.</th>
<th>OTHER</th>
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</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 6.5 feet.

Groundwater not encountered during drilling.

Borehole utilized for percolation testing.

Borehole backfilled with soil cuttings and patched with asphalt after completion of percolation test on 7-11-13.
**Log of Boring No. BH-12**

Dates Drilled: 7/12/2013  Logged by: JR  Checked By: SCL

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

Ground Surface Elevation (ft): N/A  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.

**STREAM DEPOSIT (Qg):**
SAND WITH SILT (SP-SM): fine to coarse-grained, gravels up to 3" in maximum dimension, light brown.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>SAMPLES</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>DRIVE</td>
<td>BULK</td>
<td>MOISTURE (%)</td>
<td>DRY UNIT WT. (pcf)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>46/50(3&quot;)</td>
<td>1</td>
<td>138</td>
<td></td>
</tr>
</tbody>
</table>

End of boring at 5.75 feet. Groundwater not encountered during drilling. Borehole utilized for percolation testing. Borehole backfilled with soil cuttings after completion of percolation test on 7-12-13.
Log of Boring No. BH-13

Dates Drilled: 7/12/2013  Logged by: JR  Checked By: SCL

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

Ground Surface Elevation (ft): N/A  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></td>
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<td>This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>STREAM DEPOSIT (Qg):</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td>GRAVELLY SAND WITH SILT(SP-SM): fine to</td>
</tr>
<tr>
<td></td>
<td></td>
<td>coarse-grained, gravels up to 3&quot; in maximum dimension, cobbles up to 12&quot; in maximum dimension.</td>
</tr>
</tbody>
</table>


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Converse Consultants

ARROYO SECO CANYON PROJECT
PASADENA, CALIFORNIA

Project Name
Project No. 13-31-199-01
Drawing No. A-14
### Log of Boring No. BH-14

**Dates Drilled:** 7/12/2013

**Logged by:** JR

**Checked By:** SCL

**Equipment:** 8" HOLLOW STEM AUGER

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

**Ground Surface Elevation (ft):** N/A

**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>SAMPLES</th>
<th>BLOWS/FT</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>OTHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**STREAM DEPOSIT (Qq):**
SILOY SAND (SM): coarse-grained, gravels and cobbles up to 10" in maximum dimension, light gray brown.

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

End of boring at 6.5 feet.
Groundwater not encountered during drilling.
Borehole utilized for percolation testing.
Borehole backfilled with soil cuttings after completion of percolation test on 7-12-13.

---

Converse Consultants

Project Name: ARROYO SECO CANYON PROJECT

Project No.: 13-31-199-01

Drawing No.: A-15

---

Project ID: 13-31-199-01.GFJ; Template: LOG
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 relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, Field Exploration. The following is a summary of the laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests, performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the in situ dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, Field Exploration.

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analyses were performed on nine (9) selected samples. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curve is shown in Drawing No. B-1a and B-1b, Grain Size Distribution Results.

Maximum Density Test

One (1) representative bulk sample was tested in the laboratory to determine the maximum dry density and optimum moisture content. The tests were conducted in accordance with the ASTM Standard D1557 laboratory procedure. The test results are presented in Drawing No. B-2, Moisture-Density Relationship Results.

Direct Shear

Direct shear tests were performed on four (4) relatively undisturbed in-situ samples. For each test, three 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 sample was then sheared at a constant strain rate of 0.01 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data,
including sample density and moisture content, see Drawing No. B-3a through B-3d, *Direct Shear Test Results*.

**Table No. B-1, Direct Shear Test Results**

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (feet)</th>
<th>Soil Classification</th>
<th>Ultimate Strength Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Friction Angle</td>
</tr>
<tr>
<td>BH-1</td>
<td>3</td>
<td>Gravelly Sand with Silt (SP-SM)</td>
<td>33</td>
</tr>
<tr>
<td>BH-3</td>
<td>3</td>
<td>Weathered Granitic Bedrock (gr)</td>
<td>34</td>
</tr>
<tr>
<td>BH-6</td>
<td>7</td>
<td>Silty Sand (SM)</td>
<td>33</td>
</tr>
<tr>
<td>BH-8</td>
<td>10</td>
<td>Sand with Silt (SP-SM)</td>
<td>33</td>
</tr>
</tbody>
</table>

**Consolidation**

Consolidation tests were performed on one (1) relatively undisturbed in-situ sample. Data obtained from this test procedure was used to evaluate the settlement characteristics of the foundation soils under load. Preparation for this test involved trimming the sample and placing the one-inch high brass ring into the test apparatus, which contained porous stones, both top and bottom, to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. The sample was tested at field and submerged conditions. The test results, including sample density and moisture content, are presented in Drawing No. B-4, *Consolidation Test Results*.

**Soil Corrosivity**

Three (3) representative soil samples were tested to evaluate minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by Environmental Geotechnology Laboratory, Inc. (EGL), located in Arcadia, California. The test results received from EGL are included in the following table:
Table No. B-2, Corrosivity Test Results

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Sample Depth (feet)</th>
<th>pH (Caltrans 643)</th>
<th>Soluble Chlorides (Caltrans 422) ppm</th>
<th>Soluble Sulfate (Caltrans 417) (%)</th>
<th>Saturated Resistivity (Caltrans 643) Ohm-cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>0 – 3</td>
<td>7.15</td>
<td>85</td>
<td>0.01</td>
<td>22,000</td>
</tr>
<tr>
<td>BH-3</td>
<td>0 – 3</td>
<td>8.15</td>
<td>75</td>
<td>0.002</td>
<td>23,000</td>
</tr>
<tr>
<td>BH-5</td>
<td>0 – 2.5</td>
<td>6.01</td>
<td>75</td>
<td>0.001</td>
<td>23,000</td>
</tr>
</tbody>
</table>

**R-value**

One (1) representative bulk soil sample was tested for resistance value (R-value) in accordance with ASTM D2844 Standard. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table:

Table No. B-3, R-value Test Result

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth, (feet)</th>
<th>Soil Classification</th>
<th>Measured R-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-3</td>
<td>0 – 3</td>
<td>Weathered Grantic Bedrock (gr)</td>
<td>74</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.
### GRAIN SIZE DISTRIBUTION RESULTS

**Converse Consultants**

**Project Name:** ARROYO SECO CANYON PROJECT  
**Location:** PASADENA, CALIFORNIA

---

**Table: Grain Size Distribution**

<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-1</td>
<td>0-3</td>
<td>GRAVELLY SAND WITH SILT (SP-SM)</td>
<td></td>
<td></td>
<td></td>
<td>0.80</td>
<td>18.70</td>
</tr>
<tr>
<td>BH-2</td>
<td>0-5</td>
<td>SANDY SILT WITH SILTY SAND (SM/ML)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-3</td>
<td>0-3</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-5</td>
<td>0-2.5</td>
<td>GRAVELLY SAND WITH SILT (SP-SM)</td>
<td></td>
<td></td>
<td></td>
<td>1.29</td>
<td>20.70</td>
</tr>
<tr>
<td>BH-6</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>0-3</td>
<td>25</td>
<td>2.081</td>
<td>0.43</td>
<td>0.111</td>
<td>28.0</td>
<td>64.7</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>BH-2</td>
<td>0-5</td>
<td>9.5</td>
<td>0.082</td>
<td></td>
<td></td>
<td>1.0</td>
<td>40.8</td>
<td>58.2</td>
<td></td>
</tr>
<tr>
<td>BH-3</td>
<td>0-3</td>
<td>19</td>
<td>1.365</td>
<td>0.315</td>
<td></td>
<td>10.0</td>
<td>75.3</td>
<td>14.7</td>
<td></td>
</tr>
<tr>
<td>BH-5</td>
<td>0-2.5</td>
<td>25</td>
<td>1.848</td>
<td>0.462</td>
<td>0.089</td>
<td>22.0</td>
<td>69.7</td>
<td>8.3</td>
<td></td>
</tr>
<tr>
<td>BH-6</td>
<td>0-5</td>
<td>25</td>
<td>0.649</td>
<td>0.086</td>
<td></td>
<td>17.0</td>
<td>55.6</td>
<td>27.4</td>
<td></td>
</tr>
</tbody>
</table>
**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-10</td>
<td>0-5</td>
<td>GRAVELLY SAND WITH SILT (SP-SM)</td>
<td>0.85</td>
<td>47.55</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-11</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td>0.81</td>
<td>22.11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-12</td>
<td>0-5</td>
<td>SAND WITH SILT (SP-SM)</td>
<td>1.47</td>
<td>13.69</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-13</td>
<td>0-3</td>
<td>GRAVELLY SAND WITH SILT (SP-SM)</td>
<td>0.80</td>
<td>18.70</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-14</td>
<td>0-5</td>
<td>SILTY SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-10</td>
<td>0-5</td>
<td>25.4</td>
<td>3.138</td>
<td>0.42</td>
<td>34.0</td>
<td>54.9</td>
<td>11.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-11</td>
<td>0-5</td>
<td>19</td>
<td>1.369</td>
<td>0.262</td>
<td>17.0</td>
<td>70.6</td>
<td>12.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH-12</td>
<td>0-5</td>
<td>19</td>
<td>1.226</td>
<td>0.403</td>
<td>0.09</td>
<td>12.0</td>
<td>79.4</td>
<td>8.6</td>
<td></td>
</tr>
<tr>
<td>BH-13</td>
<td>0-3</td>
<td>25</td>
<td>2.081</td>
<td>0.43</td>
<td>0.111</td>
<td>28.0</td>
<td>64.7</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>BH-14</td>
<td>0-5</td>
<td>19</td>
<td>0.297</td>
<td>0.43</td>
<td>0.111</td>
<td>10.0</td>
<td>54.0</td>
<td>36.0</td>
<td></td>
</tr>
</tbody>
</table>

**Converse Consultants**

Project Name: ARROYO SECO CANYON PROJECT
Pasadena, California

Project No.: 13-31-199-01
Drawing No.: B-1b
NOTE: Ultimate Strength.
DIRECT SHEAR TEST RESULTS

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-3</th>
<th>DEPTH (ft)</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESCRIPTION</td>
<td>WEATHERED BEDROCK (gr)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>COHESION (psf)</td>
<td>400</td>
<td>FRICTION ANGLE (degrees)</td>
<td>34</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>15.4</td>
<td>DRY DENSITY (pcf)</td>
<td>117</td>
</tr>
</tbody>
</table>

NOTE: Ultimate Strength.
NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>BH-6</th>
<th>DEPTH (ft)</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESCRIPTION</td>
<td>SILTY SAND (SM)</td>
<td>COHESION (psf)</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FRICTION ANGLE (degrees)</td>
<td>33</td>
</tr>
<tr>
<td>MOISTURE CONTENT (%)</td>
<td>9.4</td>
<td>DRY DENSITY (pcf)</td>
<td>100.2</td>
</tr>
</tbody>
</table>
### DIRECT SHEAR TEST RESULTS

**BORING NO.:** BH-8  
**DEPTH (ft):** 10  
**DESCRIPTION:** SAND WITH SILT (SP-SM)  
**COHESION (psf):** 200  
**FRICTION ANGLE (degrees):** 33  
**MOISTURE CONTENT (%):** 15.3  
**DRY DENSITY (pcf):** 107.4

**NOTE:** Ultimate Strength.

---

**Converse Consultants**  
**Project Name:** ARROYO SECO CANYON PROJECT  
**Project No.:** 13-31-199-01  
**Drawing No.:** B-3d  
**Location:** PASADENA, CALIFORNIA
**CONSOLIDATION TEST RESULTS**

BORING NO. : BH-1  
DESCRIPTION : SILTY SAND (SM)  
DEPTH (ft) : 5  

<table>
<thead>
<tr>
<th>MOISTURE CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>PERCENT SATURATION</th>
<th>VOID RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL</td>
<td>11.8</td>
<td>116.8</td>
<td></td>
</tr>
<tr>
<td>FINAL</td>
<td>18.1</td>
<td>116.8</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER
Percolation Testing

Job Name: Arroyo Seco Canyon Project
Job No.: 13-31-199-01
Location: Site No. 1 - Leach field area
Test Date: July 9, 2013

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement at The Bottom 12 inches</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time Interval (ΔT)</td>
</tr>
<tr>
<td>Initial Time Tᵢ</td>
<td>Final Time Tᵢ</td>
</tr>
<tr>
<td>Presoak 7/8/2013</td>
<td>7/9/2013 10:05 AM</td>
</tr>
<tr>
<td>Percolation Test</td>
<td></td>
</tr>
<tr>
<td>11:40:00 AM</td>
<td>11:45:00 AM</td>
</tr>
<tr>
<td>11:45:00 AM</td>
<td>11:52:00 AM</td>
</tr>
<tr>
<td>11:52:00 AM</td>
<td>11:57:00 AM</td>
</tr>
<tr>
<td>11:57:00 AM</td>
<td>12:05:00 PM</td>
</tr>
<tr>
<td>12:05:00 PM</td>
<td>12:15:00 PM</td>
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<tr>
<td>12:15:00 PM</td>
<td>12:25:00 PM</td>
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<tr>
<td>12:37:00 PM</td>
<td>12:42:00 PM</td>
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<td>12:42:00 PM</td>
<td>12:49:00 PM</td>
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<td>12:49:00 PM</td>
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<td>12:55:00 PM</td>
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<tr>
<td>1:02:00 PM</td>
<td>1:09:00 PM</td>
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<td>1:09:00 PM</td>
<td>1:20:00 PM</td>
</tr>
<tr>
<td>1:44:00 PM</td>
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<td>1:52:00 PM</td>
<td>1:59:00 PM</td>
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<td>1:55:00 PM</td>
<td>2:06:00 PM</td>
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<tr>
<td>2:06:00 PM</td>
<td>2:13:00 PM</td>
</tr>
<tr>
<td>2:13:00 PM</td>
<td>2:21:00 PM</td>
</tr>
<tr>
<td>2:21:00 PM</td>
<td>2:31:00 PM</td>
</tr>
</tbody>
</table>

Percolation Testing

Job Name: Arroyo Seco Canyon Project
Job No.: 13-31-199-01
Location: Site No. 2C - Leach field area
Test Date: July 11, 2013

<table>
<thead>
<tr>
<th>Test Boring No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-5</td>
</tr>
<tr>
<td>Depth of Boring ($d_b$): 3.5 feet</td>
</tr>
<tr>
<td>Diameter of Boring (D): 1.00 foot</td>
</tr>
<tr>
<td>Test Performer: MDR</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement at The Bottom 12 inches</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time $T_i$</td>
<td>Final Time $T_f$</td>
<td>Time Interval $\Delta T$ (minutes)</td>
</tr>
<tr>
<td>Presoak</td>
<td>7/10/2013 7:30 AM</td>
<td>7/11/2013 10:40 AM</td>
</tr>
<tr>
<td>Percolation Test</td>
<td>10:47:00 AM</td>
<td>11:24:00 AM</td>
</tr>
<tr>
<td></td>
<td>11:24:00 AM</td>
<td>12:45:00 PM</td>
</tr>
</tbody>
</table>

# Percolation Testing

**Job Name:** Arroyo Seco Canyon Project  
**Job No.:** 13-31-199-01  
**Location:** Site No. 3 - JPL Parking Lot  
**Test Date:** July 11, 2013

![Percolation Diagram](attachment:image.png)

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time</td>
<td>Final Time</td>
<td>Time Interval</td>
<td>Initial depth to water</td>
</tr>
<tr>
<td>$T_i$</td>
<td>$T_f$</td>
<td>$\Delta T$ (hr)</td>
<td>$d_i$ (feet)</td>
</tr>
<tr>
<td>Presoak</td>
<td>9:28 AM</td>
<td>11:28 AM</td>
<td>2.00</td>
</tr>
<tr>
<td>Percolation Test</td>
<td>11:35:00 AM</td>
<td>11:45:00 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:45:00 AM</td>
<td>11:55:00 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:55:00 AM</td>
<td>12:05:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:09:00 PM</td>
<td>12:19:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:19:00 PM</td>
<td>12:29:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:29:00 PM</td>
<td>12:39:00 PM</td>
<td>0.17</td>
</tr>
<tr>
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<td>12:41:00 PM</td>
<td>12:51:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:51:00 PM</td>
<td>1:01:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>1:01:00 PM</td>
<td>1:11:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>1:14:00 PM</td>
<td>1:24:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>1:24:00 PM</td>
<td>1:34:00 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>1:34:00 PM</td>
<td>1:44:00 PM</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Note: Reduction Factor, $R_f = \frac{(2^d - \Delta d)D}{+1}$

**Lowest Percolation Rate**: 6.58 inch/hr  
**Average Percolation Rate**: 7.87 inch/hr

## Percolation Testing

**Job Name:** Arroyo Seco Canyon Project  
**Job No.:** 13-31-199-01  
**Location:** Site No. 3 - JPL Parking Lot  
**Test Date:** July 11, 2013

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time</td>
<td>Final Time</td>
<td>Time Interval</td>
<td>Initial depth to water</td>
</tr>
<tr>
<td>$T_i$</td>
<td>$T_f$</td>
<td>$\Delta T$</td>
<td>$d_1$</td>
</tr>
<tr>
<td>Presoak</td>
<td>8:59 AM</td>
<td>11:00 AM</td>
<td>2.00</td>
</tr>
</tbody>
</table>

### Percolation Test

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time</td>
<td>Final Time</td>
<td>Time Interval</td>
<td>Initial depth to water</td>
</tr>
<tr>
<td>11:11:00 AM</td>
<td>11:21:00 AM</td>
<td>0.17</td>
<td>0.00</td>
</tr>
<tr>
<td>11:21:00 AM</td>
<td>11:31:00 AM</td>
<td>0.17</td>
<td>2.60</td>
</tr>
<tr>
<td>11:31:00 AM</td>
<td>11:41:00 AM</td>
<td>0.17</td>
<td>3.69</td>
</tr>
<tr>
<td>11:42:00 AM</td>
<td>11:52:00 AM</td>
<td>0.17</td>
<td>0.00</td>
</tr>
<tr>
<td>11:52:00 AM</td>
<td>12:02:00 PM</td>
<td>0.17</td>
<td>2.47</td>
</tr>
<tr>
<td>12:02:00 PM</td>
<td>12:12:00 PM</td>
<td>0.17</td>
<td>3.59</td>
</tr>
<tr>
<td>12:13:00 PM</td>
<td>12:23:00 PM</td>
<td>0.17</td>
<td>0.00</td>
</tr>
<tr>
<td>12:23:00 PM</td>
<td>12:33:00 PM</td>
<td>0.17</td>
<td>2.58</td>
</tr>
<tr>
<td>12:33:00 PM</td>
<td>12:43:00 PM</td>
<td>0.17</td>
<td>3.65</td>
</tr>
<tr>
<td>12:44:00 PM</td>
<td>12:54:00 PM</td>
<td>0.17</td>
<td>0.00</td>
</tr>
<tr>
<td>12:54:00 PM</td>
<td>1:04:00 PM</td>
<td>0.17</td>
<td>2.43</td>
</tr>
<tr>
<td>1:04:00 PM</td>
<td>1:14:00 PM</td>
<td>0.17</td>
<td>3.55</td>
</tr>
</tbody>
</table>

**Note:** Reduction Factor, $R_f = \frac{(2^2d_1 - \Delta d)}{D + 1}$

**Lowest Percolation Rate** = 11.52 inch/hr  
**Average Percolation Rate** = 15.90 inch/hr

## Percolation Testing

**Job Name:** Arroyo Seco Canyon Project  
**Job No.:** 13-31-199-01  
**Location:** Site No. 3 - JPL Parking Lot  
**Test Date:** July 11, 2013

<table>
<thead>
<tr>
<th>Test Boring No.</th>
<th>BH-11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Boring ($d_b$):</td>
<td>5.0 feet</td>
</tr>
<tr>
<td>Diameter of Boring ($D$):</td>
<td>0.67 feet</td>
</tr>
<tr>
<td>Test Performer:</td>
<td>JR</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time ($T_f$)</td>
<td>Final Time ($T_f$)</td>
<td>Time interval ($\Delta T$)</td>
<td>Initial depth to water ($d_1$)</td>
</tr>
<tr>
<td><strong>Presoak</strong></td>
<td>8:00 AM</td>
<td>10:00 AM</td>
<td>2.00</td>
</tr>
<tr>
<td><strong>Percolation Test</strong></td>
<td>10:00 AM</td>
<td>10:40 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>10:40 AM</td>
<td>10:50 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:00 AM</td>
<td>11:06 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:20 AM</td>
<td>11:27 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:27 AM</td>
<td>11:37 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:37 AM</td>
<td>11:47 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:48 AM</td>
<td>11:58 AM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>11:58 AM</td>
<td>12:08 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:08 PM</td>
<td>12:16 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:16 PM</td>
<td>12:26 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:26 PM</td>
<td>12:36 PM</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>12:36 PM</td>
<td>12:49 PM</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Note: Reduction Factor, $R_f = (2^2d - \Delta d)/D + 1$

Lowest Percolation Rate = 4.50 inch/hr  
Average Percolation Rate = 6.65 inch/hr

## Percolation Testing

**Job Name:** Arroyo Seco Canyon Project  
**Job No.:** 13-31-199-01  
**Location:** Site No. 3 - Sludge Basin 2  
**Test Date:** July 12, 2013  
**Test Boring No.:** BH-12  
**Depth of Boring (d_b):** 5.0 feet  
**Diameter of Boring (D):** 0.67 feet  
**Test Performer:** MDR

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time</td>
<td>Final Time</td>
<td>Time Interval (hr)</td>
<td>Initial depth to water (feet)</td>
</tr>
<tr>
<td>Presoak</td>
<td></td>
<td></td>
<td>d_1</td>
</tr>
<tr>
<td>10:20 AM</td>
<td>3:20 PM</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Percolation Test</td>
<td></td>
<td></td>
<td>3:46:00 AM</td>
</tr>
<tr>
<td>3:51:00 AM</td>
<td>3:56:00 AM</td>
<td>0.08</td>
<td>3.25</td>
</tr>
<tr>
<td>4:02:00 PM</td>
<td>4:07:00 PM</td>
<td>0.08</td>
<td>0.00</td>
</tr>
<tr>
<td>4:07:00 PM</td>
<td>4:12:00 PM</td>
<td>0.08</td>
<td>3.29</td>
</tr>
<tr>
<td>4:15:00 PM</td>
<td>4:20:00 PM</td>
<td>0.08</td>
<td>0.00</td>
</tr>
<tr>
<td>4:20:00 PM</td>
<td>4:25:00 PM</td>
<td>0.08</td>
<td>3.25</td>
</tr>
<tr>
<td>4:28:00 AM</td>
<td>4:33:00 AM</td>
<td>0.08</td>
<td>0.00</td>
</tr>
<tr>
<td>4:33:00 AM</td>
<td>4:38:00 AM</td>
<td>0.08</td>
<td>3.25</td>
</tr>
</tbody>
</table>

**Note:** Reduction Factor, \( R_f = (2 \times d_1 - Δd) / D + 1 \)

**Lowest Percolation Rate:** 4.70 inch/hr  
**Average Percolation Rate:** 24.09 inch/hr

## Percolation Testing

**Job Name:** Arroyo Seco Canyon Project  
**Job No.:** 13-31-199-01  
**Location:** Site No. 3 - Basin 8  
**Test Date:** July 12, 2013  

### Test Boring No.  
BH-13  
Depth of Boring \((d_b)\): 2.8 feet  
Diameter of Boring \((D)\): 0.67 feet  
Test Performer: MDR

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Time</td>
<td>Final Time</td>
<td>Time Interval ((\Delta T))</td>
<td>Initial depth to water ((d_1)) (feet)</td>
</tr>
<tr>
<td>-----------------</td>
<td>--------------------------</td>
<td>--------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td><strong>Presoak</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7:30 AM</td>
<td>9:00 AM</td>
<td>1.50</td>
<td>0.00</td>
</tr>
<tr>
<td><strong>Percolation Test</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9:01:00 AM</td>
<td>9:04:00 AM</td>
<td>0.05</td>
<td>0.00</td>
</tr>
<tr>
<td>9:04:00 AM</td>
<td>9:06:00 AM</td>
<td>0.03</td>
<td>2.29</td>
</tr>
<tr>
<td>9:06:00 AM</td>
<td>9:08:00 AM</td>
<td>0.03</td>
<td>2.50</td>
</tr>
<tr>
<td>9:21:00 AM</td>
<td>9:24:00 AM</td>
<td>0.05</td>
<td>0.00</td>
</tr>
<tr>
<td>9:24:00 AM</td>
<td>9:25:00 AM</td>
<td>0.03</td>
<td>2.25</td>
</tr>
<tr>
<td>9:33:00 AM</td>
<td>9:36:00 AM</td>
<td>0.05</td>
<td>0.00</td>
</tr>
<tr>
<td>9:36:00 AM</td>
<td>9:38:00 AM</td>
<td>0.03</td>
<td>2.21</td>
</tr>
<tr>
<td>9:52:00 AM</td>
<td>9:55:00 AM</td>
<td>0.05</td>
<td>0.00</td>
</tr>
<tr>
<td>9:55:00 AM</td>
<td>9:57:00 AM</td>
<td>0.03</td>
<td>2.20</td>
</tr>
</tbody>
</table>

Note: Reduction Factor, \(R_f = (2d_2 - \Delta d)/D + 1\)

**Lowest Percolation Rate** = 27.51 inch/hr  
**Average Percolation Rate** = 58.65 inch/hr

## Percolation Testing

**Job Name:** Arroyo Seco Canyon Project  
**Job No.:** 13-31-199-01  
**Location:** Site No. 3 - Basin 13  
**Test Date:** July 12, 2013  

| Test Boring No. | BH-14  | Depth of Boring ($d_o$): | 5.0 feet  | Diameter of Boring (D): | 0.67 feet  
| Test Performer: | MDR  |

<table>
<thead>
<tr>
<th>Time of Testing</th>
<th>Water Level Measurement</th>
<th>Water Level Calculations</th>
<th>Percolation Rate Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Initial Time</strong></td>
<td><strong>Final Time</strong></td>
<td><strong>Time Interval ($\Delta T$)</strong></td>
<td><strong>Initial depth to water ($d_i$)</strong></td>
</tr>
<tr>
<td>$T_i$</td>
<td>$T_f$</td>
<td>(hr)</td>
<td>(feet)</td>
</tr>
<tr>
<td><strong>Presoak</strong></td>
<td>8:00 AM</td>
<td>10:30 AM</td>
<td>2.50</td>
</tr>
<tr>
<td><strong>Percolation Test</strong></td>
<td>10:34:00 AM</td>
<td>10:39:00 AM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>10:39:00 AM</td>
<td>10:44:00 AM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>10:55:00 AM</td>
<td>11:00:00 AM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>11:00:00 AM</td>
<td>11:05:00 AM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>11:22:00 AM</td>
<td>11:27:00 AM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>11:27:00 AM</td>
<td>11:32:00 AM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>3:10:00 PM</td>
<td>3:15:00 PM</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>3:15:00 PM</td>
<td>3:20:00 PM</td>
<td>0.08</td>
</tr>
</tbody>
</table>

**Note:** Reduction Factor, $R_f = (2^*d_i - \Delta d)/D + 1$

**Lowest Percolation Rate = 16.96 inch/hr**  
**Average Percolation Rate = 28.07 inch/hr**

APPENDIX D

SLOPE STABILITY ANALYSES
Description: Gross Slope Stability Analysis - Geologic Cross Section A-A'
Comments: 13-31-199-01 - Arroyo Seco Canyon Project
File Name: 13-31-199-01_a1.slp
Last Saved Date: 8/20/2013
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius

Case 1: Groundwater level at 20' below grade
Soil Shear Strength
Qg
unit weight = 125 pcf
cohesion= 200 psf
friction= 33 degrees

Scale: 1"=50'
Plate S-A1
Case 2: Groundwater level at 0' below parking lot grade

Soil Shear Strength

Qg

unit weight = 125 pcf

cohesion = 200 psf

friction = 33 degrees
FILEINFO
SLOPEW 5.04
TITLE
Gross Slope Stability Analysis - Geologic Cross Section A-A'
13-31-199-01 - Arroyo Seco Canyon Project
DASTAMP 8/20/2013
TIMESTAMP 8:23:45 AM
ANALYSIS
  2  2  1 +6.2400e+001  1  0  0
CONVERGE
  30 +1.0000e-002  1000 +0.0000e+000  0  0  0
SIDE
  1
LAMBDA
+0.0000e+000 +9.9900e+002 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
SOIL
  1 +1.2500e+002 +2.0000e+002 +3.3000e+001 +0.0000e+000 +0.0000e+000 +0.0000e+000
+0.0000e+000  1  0
 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
+0.0000e+000
 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
+0.0000e+000
Stream Deposit (Qg)
SFUNCTION  0
AFUNCTION  0
POINT  12
  1 +0.0000e+000 +1.0930e+003
  2 +1.6000e+001 +1.0930e+003
  3 +2.9690e+001 +1.0930e+003
  4 +3.6981e+001 +1.0988e+003
  5 +4.4165e+001 +1.1107e+003
  6 +9.4743e+001 +1.1169e+003
  7 +3.4068e+002 +1.1166e+003
  8 -4.7258e+001 +1.2022e+003
  9 -4.6392e+001 +1.1180e+003
 10 +4.1597e+001 +1.1177e+003
 11 +9.5587e+001 +1.0955e+003
 12 +3.3965e+002 +1.0972e+003
LINE  1
  1 7
  1
  2
  3
  4
  5
  6
TENSION
  0 +6.2400e+001 +0.0000e+000 +0.0000e+000  0
GRID
  9 10 8 6 6 0 +0.0000e+000 0 +0.0000e+000
RADIUS
  3 3 3 3 0 3 3
AXIS
  0
LIMIT
FILEINFO
SLOPEW 5.04
TITLE
Gross Slope Stability Analysis - Geologic Cross Section A-A'
13-31-199-01 - Arroyo Seco Canyon Project
DATESTAMP 8/20/2013
TIMESTAMP 8:34:20 AM
ANALYSIS
    2   2   1  +6.2400e+001  1  0  0
CONVERGE
    30  +1.0000e+002  1000  +0.0000e+000  0  0  0
SIDE
    1
LAMBDAX
  +0.0000e+000  +9.9900e+002  +0.0000e+000  +0.0000e+000  +0.0000e+000  +0.0000e+000
SOIL
    1
  +1.2500e+002  +2.0000e+002  +3.3000e+001  +0.0000e+000  +0.0000e+000
  +0.0000e+000  1  0
  +0.0000e+000  +0.0000e+000  +0.0000e+000  +0.0000e+000  0  0
  +0.0000e+000  +0.0000e+000  +0.0000e+000  +0.0000e+000  +0.0000e+000
  +0.0000e+000
Stream Deposit (Qg)
SFUNCTIN  0
AFUNCTION  0
POINT
    12
  +1.0000e+000  +1.0930e+003
  +1.6000e+001  +1.0930e+003
  +2.9690e+001  +1.0930e+003
  +3.6981e+001  +1.0988e+003
  +4.4165e+001  +1.1107e+003
  +9.4743e+001  +1.1169e+003
  +3.4068e+002  +1.1166e+003
  +4.7258e+001  +1.2022e+003
  +4.6392e+001  +1.1180e+003
  +4.1597e+001  +1.1177e+003
  +9.6972e+001  +1.1164e+003
  +3.3975e+002  +1.1163e+003
LINE
    1
    1
    1
    2
    3
    4
    5
    6
    7
TENSION
  +6.2400e+001  +0.0000e+000  +0.0000e+000
GRID
  9  10  8  6  6  0  +0.0000e+000  0  +0.0000e+000
RADIUS
  3  3  3  3  0  3  3
AXIS
  0
LIMIT
0 : 0.0000e+000 1.34068e+002
SLIP 0

BLOCK
 0 0 0 0 0 +1.3500e+002 +1.3500e+002 0 0
 0 0 0 0 0 +4.5000e+001 +4.5000e+001 0 0

PORU 1
 1 +0.0000e+000 0 +0.0000e+000

PBRAR 1
 1 +0.0000e+000 0 +0.0000e+000

PIEZ 1 +0.0000e+000 0

PCON 0
POGH 0
POGP 0
POGR 0

PORA 1
 1 +0.0000e+000

LOAD 0
ANCHOR 0
PBOUNDARY 0
SEISMIC
+0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
INTEGRATION

-1

-1
ENGINEERING
FT
MATCOLOR 1
 1 255 255 123
Description: Gross Slope Stability Analysis - Geologic Cross Section B-B'
Comments: 13-31-199-01 Arroyo Seco Canyon Project
File Name: 13-31-199-01_b2.slip
Last Saved Date: 8/20/2013
Analysis Method: Spencer
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius

Case 2: Groundwater level at 0' below parking lot grade
Soil Shear Strength
Qg
unit weight=125 pcf
cohesion=250
friction=33 degrees

New Basin E
BH-6
BH-7
Groundwater level

Scale: 1" = 50'

Plate S-B2
FILEINFO
SLOPEW 5.04
TITLE
Gross Slope Stability Analysis - Geologic Cross Section B-B'
13-31-199-01 Arroyo Seco Canyon Project
DATESTAMP 8/20/2013
TIMESTAMP 9:03:04 AM
ANALYSIS
2 2 1 +6.2400e+001 1 0 C
CONVERGE
30 +1.0000e-002 1000 +0.0000e+000 0 0 0
SIDE
1
LAMBDAL
+0.0000e+000 +9.9900e+002 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
SOIL 1
+1.2500e+002 +2.5000e+002 +3.4000e+001 +0.0000e+000 +0.0000e+000
+0.0000e+000 1 0
+0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 0 0
+0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
+0.0000e+000
Stream Deposit
SFUNCTION 0
AFUNCTION 0
POINT 16
1 +0.0000e+000 +1.1111e+003
2 +9.1000e+001 +1.1160e+003
3 +1.2100e+002 +1.1160e+003
4 +2.1004e+002 +1.1568e+003
5 +2.2700e+002 +1.1580e+003
6 +2.8300e+002 +1.2190e+003
7 +3.6266e+002 +1.2331e+003
8 +3.8088e+002 +1.2331e+003
9 +7.0000e+001 +1.3450e+003
10 +7.0024e+001 +1.2455e+003
11 +1.7800e+002 +1.2460e+003
12 +0.0000e+000 +1.0960e+003
13 +1.0000e+002 +1.0960e+003
14 +2.0000e+002 +1.0960e+003
15 +3.0000e+002 +1.0960e+003
16 +3.7981e+002 +1.0953e+003
LINE 1
1 8
2
3
4
5
6
7
8
TENSION
0 +6.2400e+001 +0.0000e+000 +0.0000e+000 0
GRID
10 11 9 6 6 0 +0.0000e+000 0 +0.0000e+000
FILEINFO
SLOPEW 5.04
TITLE
Gross Slope Stability Analysis - Geologic Cross Section B-B'
13-31-199-01 Arroyo Seco Canyon Project
DATESTAMP 8/20/2013
TIMESTAMP 9:13:59 AM
ANALYSIS
  2 2 1 +6.2400e+001 1 )]
CONVERGE
  30 +1.0000e-002 1000 +0.0000e+000 0 0 0
SIDE
  1
LAMBDA
+0.0000e+000 +9.9900e+002 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
SOIL
  1 +1.2500e+002 +2.5000e+002 +3.4000e+001 +0.0000e+000 +0.0000e+000
+0.0000e+000 1 0
+0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 0 0
+0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
Stream Deposit
SFUNCTION 0
AFUNCTION 0
POINT
  16
  1 +0.0000e+000 +1.1111e+003
  2 +9.1000e+001 +1.1160e+003
  3 +1.2100e+002 +1.1160e+003
  4 +2.1004e+002 +1.1568e+003
  5 +2.2700e+002 +1.1580e+003
  6 +2.8300e+002 +1.2190e+003
  7 +3.6266e+002 +1.2331e+003
  8 +3.8088e+002 +1.2331e+003
  9 +7.0000e+001 +1.3450e+003
 10 +7.0024e+001 +1.2455e+003
 11 +1.7800e+002 +1.2460e+003
 12 -1.6248e+001 +1.1110e+003
 13 +7.9046e+001 +1.1151e+003
 14 +1.2100e+002 +1.1150e+003
 15 +2.3000e+002 +1.0960e+003
 16 +3.7981e+002 +1.0953e+003
LINE
  1
  8
  1
  2
  3
  4
  5
  6
  7
  8
TENSION
  0 +6.2400e+001 +0.0000e+000 +0.0000e+000 0
GKLV
  10 11 9 6 6 0 +0.0000e+000 0 +0.0000e+000
RADIUS
  3 3 3 3 0 3 3
AXIS
  0
LIMIT
  U *0.0000e+000 *3.8088e+002
SLIP 0
BLOCK
  0 0 0 0 0 *1.3500e+002 *1.3500e+002 O 0
  0 0 0 0 0 *4.5000e+001 *4.5000e+001 O 0
PORU 1
  1 +0.0000e+000 0 +0.0000e+000
PBBAR 1
  1 +0.0000e+000 0 +0.0000e+000
PIEZ 1 +0.0000e+000 0
  1 5 1
  12
  13
  14
  15
  16
PCON 0
POGH 0
POGP 0
POGR 0
PORA 1
  1 +0.0000e+000
LOAD 0
ANCHOR 0
PBOUNDARY 0
SEISMIC
  +0.0000e+000 +0.0000e+000 +0.0000e+000 +0.0000e+000
INTEGRATION

-1

-1
ENGINEERING
FT
MATLCOLOR 1
  1 255 255 128