GEOTECHNICAL INVESTIGATION REPORT
14th Street Water Quality Regional Facility
City of Upland, California

October 26, 2010

Converse Project No. 10-81-227-01

October 26, 2010

Mr. Zack Isnasious, P.E.
Principal Engineer
AECOM Water
1131 West Sixth Street, Suite 350
Ontario, CA 91762

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
14th Street Water Quality Regional Facility
City of Upland, California
Converse Project No. 10-81-227-01

Dear Mr. Isnasious:

Converse Consultants (Converse) is pleased to submit this Geotechnical Investigation Report for the proposed 14th Street Water Quality Regional Facility in the City of Upland, California. This report was prepared in accordance with our proposal dated June 11, 2010 and your Task Order Number 03 dated June 23, 2010 to Subconsultant General Agreement dated March 23, 2009.

Based on our field investigation, laboratory data and analysis, the proposed project is feasible from a geotechnical standpoint provided recommendations presented in this report are incorporated in the design and construction.

We appreciate the opportunity to be of continued service to AECOM Water. If you should have any questions, please do not hesitate to contact us at (909) 796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, Ph.D., G. E.
Regional Manager/Principal Engineer

Dist: 4/Addressee

HSQ/HS/SM/mjr

PROFESSIONAL CERTIFICATION

This report has been prepared by the staff of Converse Consultants under the professional supervision of the individuals whose seals and signatures appear hereon.

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.

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EXECUTIVE SUMMARY

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

- The project consists of construction of a desilting/water quality basin. The basin will store approximately 106 acre feet of water as emergency storm water storage to elevation 1,440 feet above mean sea level. The basin sides vary from 2:1 to 4:1 horizontal:vertical (H:V).

- The project will consist of construction of approximately 1200 linear feet of 84-inch diameter storm drain along 14th Street from Mountain Avenue to the basin. The depth of the invert at the lowest point will be about 25 feet below existing surface (bgs). On Mountain Avenue north of 14th Street, the project consists of approximately 1,000 linear feet of 84-inch diameter storm drain with depth to invert at about 15 feet bgs. On Benson Avenue, from the basin to the 13th Street, the project consists of approximately 1,100 linear feet of either 42-inch or 66-inch diameter storm drain with depth to invert at about 16 feet bgs.

- Our scope of work included the following tasks: project set-up, site reconnaissance, subsurface field exploration, laboratory testing, engineering analysis, and preparation of this report.

- Seven (7) exploratory borings (BH-1 through BH-7) were drilled on July 7, 2010. Borings BH-1 and BH-2 were drilled on the 14th Street, boring BH-3 on Benson Avenue, and boring BH-4 on 5th Avenue. Borings BH-5 through BH-7 were drilled on the desilting/water quality basin location. Borings were planned to be drilled to a maximum depth of 25 feet bgs in order to observe the soils near the planned bottom elevation of the basin and the planned pipeline elevations. Six of the borings encountered refusal on oversized rock at depths of 2 to 4 feet bgs. Boring BH-4 encountered an unknown utility at 6.5 feet bgs. To supplement subsurface information obtained from the borings, two exploratory test pits were excavated on July 13, 2010 to a depth of nine (9) feet bgs at the desilting/water quality basin site. Our engineer and geologist visually logged the subsurface conditions encountered in the exploratory borings and test pits at the time of drilling and collected soil samples.

- Based on our field observation and site exploration data, the proposed storm drain alignment is underlain by alluvial deposits of sand, gravel, cobbles and boulders to the maximum depth explored of nine (9) feet bgs. The basin location is underlain by fill material comprised of mixtures of silty sand, gravel, cobbles and boulders to a depth of about five (5) feet. The fill material is underlain by alluvial deposits of sand, gravel and cobbles to the maximum explored depth of nine (9) feet bgs. Based on regional mapping and observations from nearby sites, the subsurface conditions at the planned pipe and basin elevations are anticipated to be very similar to the conditions encountered in the upper 9 feet.

- There are no active faults projecting toward or crossing the proposed alignment. The proposed alignment is not situated within a currently designated State of California Earthquake Fault Zone. The alignment is, however, located in a seismically active zone. Ground shaking from earthquakes associated with nearby and distant faults may occur during the lifetime of the project.

- The seismic coefficients derived from 2007 California Building Code (CBC) are presented in the text of this report.

- The potential for seismic hazards due to the secondary effects of earthquakes including surface fault rupture, surface manifestations of soil liquefaction, seismically induced differential settlement, lateral spreading, landslides, and earthquake-induced flooding is considered to be low.

- During our exploration and laboratory testing no odors or other evidence of contaminated soils and/or hazardous materials were noticed based on visual observations. It should be noted, our scope of work did not include any environmental sampling and testing.

- Groundwater was not encountered in any of the borings or test pits up to a maximum of 9 feet below the existing ground surface. Historical groundwater data indicates groundwater is below 90 feet bgs. Groundwater is not expected to be encountered during the construction of this project.

- The probability of wide area of soil beneath the basin site to be fully saturated and seismic event happening at the same time is low. The site contains significant quantity of cobbles and boulders. Hence, the probability of developing significant pore water pressure during seismic event is considered to be low for the soils beneath the basin site. Hence, the potential for liquefaction induced damage is considered to be low.

- Sand Equivalent (SE) testing was performed in accordance with the ASTM Standard 2419 on one (1) representative soil sample, indicated a SE value of 64.

- Based on the corrosivity test conducted on one representative soil sample, the site soil is not deleterious to concrete. The measured value of the minimum electrical resistivity when saturated was 12,400 Ohm-cm which indicates that the site soil has "Mildly Corrosive" potential to ferrous metals in contact with the soil. A corrosion engineer should be consulted for detailed mitigation measures for corrosion against ferrous metals in contact with the soil.

- One direct shear test was performed on a representative sample according to ASTM Standard D2435. The result of the direct shear test indicates the soil has moderate shear strength.

- Based on the result of two percolation tests performed on the test pits excavated at the desilting/water quality basin location, the average infiltration rate for native ground without siltation and vegetation is approximately 8 inches per hour. However, over time, due to siltation and vegetation growth, the infiltration rate will decrease.

- Subsurface materials for this project site should, in general, be excavatable with conventional heavy-duty equipment. Significant amounts of oversized materials consisting of gravel, cobbles, and boulders up to 24 inches, and occasionally larger, are expected to be encountered. These materials will require special handling and disposal.

- The proposed storm drain pipeline may be constructed using sloped excavations or excavations supported by shoring. Temporary shoring will be required where sloped excavations are not feasible due to space limitations of existing streets, underground utilities and nearby structures.

- Allowable net bearing pressure of 3,000 pounds per square foot may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 24 inches wide.

- Inlet and outlet structures for the basin may be supported on continuous (strip) spread footings. Actual footing size should be determined based on structural requirements. Continuous spread footings should be at least 18 inches wide. The depth of embedment below lowest adjacent soil grade of footings should be at least 18 inches. Footings located on the floor of the basin should be founded on the native soils, scarified; moisture conditioned to near optimum moisture content, and compacted to a minimum of 90 percent of the laboratory maximum dry density. An allowable net bearing capacity of 3,000 pounds per square foot (psf) may be used.

- The static settlement of structures supported on strip and/or spread footings in the range of ¼-inch to ½-inch should be anticipated. Due to the granular nature of the subsurface soils, most of the settlement should occur during construction. The differential settlement may be taken as equal to about one half of the total settlement over a horizontal distance of 50 feet

- Resistance to lateral loads can be assumed to be provided by combination of friction acting at the base of foundations and passive earth pressure. A coefficient of friction of 0.4 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 300 psf per foot of depth may be used for resistance against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 3,000 psf.

- Earthwork should be performed in accordance with recommendations presented in this report and with applicable City of Upland specifications. All backfill material should be compacted to a minimum of 90 percent of the laboratory maximum dry density. The top 12 inches of street subgrade should be compacted to 95 percent of the laboratory maximum dry density. The moisture content of compacted soils should be kept within ± three (3) percent of optimum moisture content. Rocks larger than three (3) inches in the largest dimension should not be placed as trench backfill.

- The depth of cut to create the planned desilting/water quality basin will be in the range of 25 feet at the southwest end and 35 to 45 feet at the northeast end. The presence of a non-cohesive mixture of sand, gravel and cobble might cause instability of the excavated slopes.

- Final slopes for the basin should have a 2:1 (H:V) or flatter slope ratio. It is our understanding that the basin slopes are generally 4:1 (H:V) with small areas at 2:1 (H:V). The slopes should be observed by the geotechnical consultant during grading to determine whether loose sand, excessive oversize rock, or other unfavorable conditions are present at the slope face. If such conditions are observed in areas where the slope is steeper than 4:1 (H:V), additional compaction, a replacement fill slope, or other mitigation measures may be recommended.

- In order to maintain the anticipated infiltration rate, compaction of the basin floor should be limited. The size of the earth working equipment used to excavate the bottom 2 feet of the basin should be limited to the minimum size required to efficiently excavate the encountered soils. If the density of the basin floor after excavation exceeds the in-situ density of nearby undisturbed soils, the basin floor may be scarified to a depth of 24 inches to restore permeability. Alternatively, 24 inches of soil may be excavated from the basin floor and replaced without additional compactive effort.

- For desilting/water quality basin, rock or vegetation should be used to protect the basin inlet and slopes against erosion. The outflow from the basin should be provided with outlet protection to prevent erosion and scouring of the embankment and channel. Structures should be placed on a firm, smooth foundation with the base securely anchored with concrete or other means to prevent floatation. Chainlink fencing should be provided around each sediment/desilting basin to prevent unauthorized entry to the basin or if safety is a concern.

The results of our investigation indicate that the proposed storm drain alignment and desilting/ water quality basin site are suitable from a geotechnical standpoint, provided the recommendations presented in the attached report are considered and implemented in the design and construction.

It should be noted that subsurface was limited to the soils above nine (9) feet bgs due to boring refusal on oversized rock. Based on our field observations, regional mapping, and observations from nearby sites, we anticipate that the soil conditions between 9 and 25 feet bgs are generally similar to the soil conditions in the upper 9 feet bgs. If substantial changes in soil conditions occur during the construction, project geotechnical engineer should be consulted immediately for further evaluation and recommendations.

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APPENDICES

1.0 INTRODUCTION

This report contains the findings of a geotechnical investigation performed by Converse Consultants (Converse) in the City of Upland, California for the construction of a storm drain and a desilting/water quality basin.

The location of the proposed alignment is shown in Figure No. 1, *Site Location Map*. Map obtained from AECOM Water, 14th Street Water Quality Regional Facility has been reproduced to show the approximate location of borings and test pits as shown in Figure No. 2, *Approximate Boring and Test Pit Location Map*.

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

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

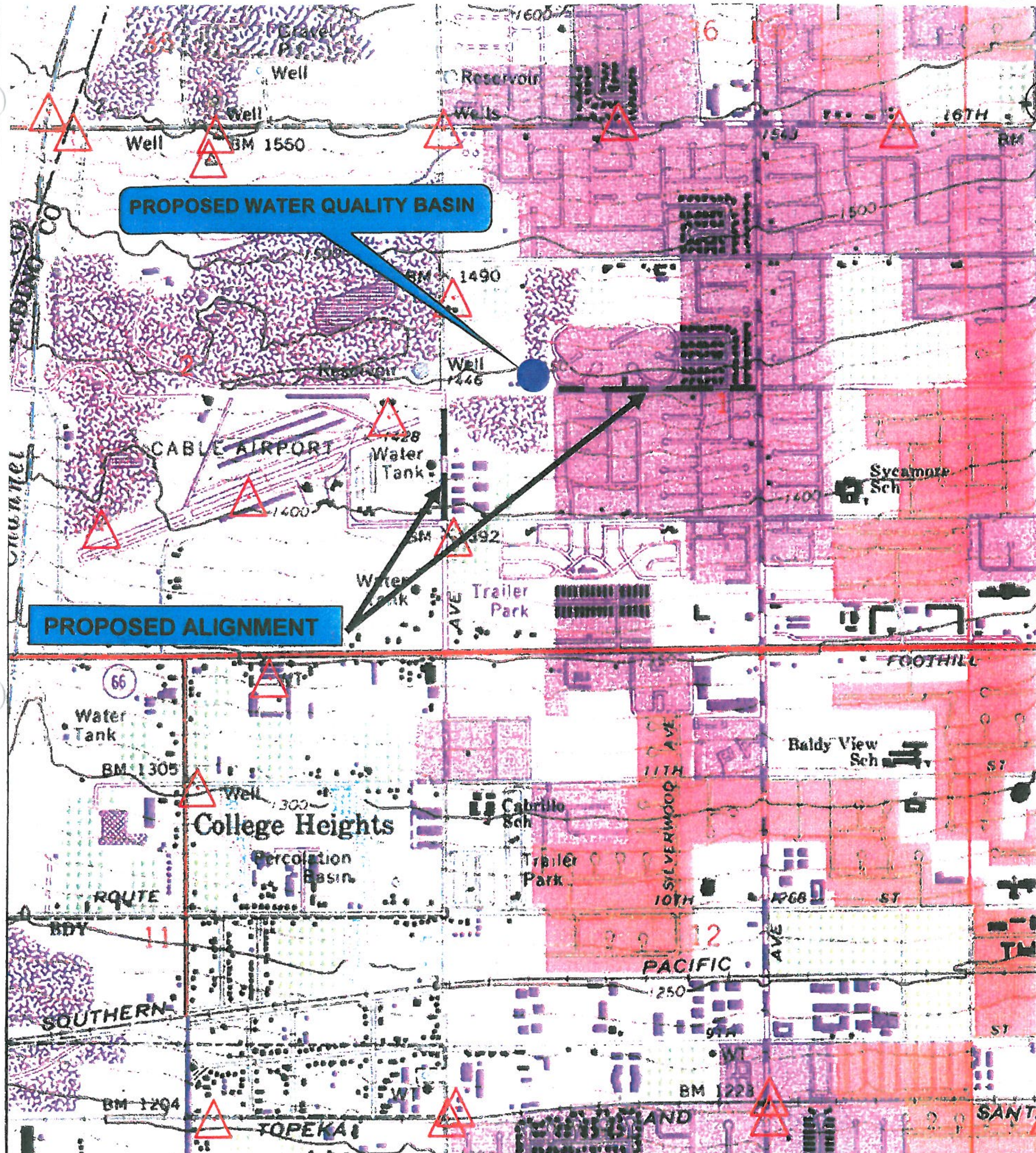
2.0 PROJECT DESCRIPTION

The project consists of construction of a desilting/water quality basin. The basin will store approximately 106-acre feet of water as emergency storm water storage to elevation 1440 feet above mean sea level. It is our understanding that the basin sides vary from 2:1 to 4:1 horizontal:vertical (H:V).

The project will consist of construction of approximately 1,200 linear feet of 84 inches diameter storm drain along 14th Street from Mountain Avenue to the basin. The depth of the invert at the lowest point will be about 25 feet below existing ground surface (bgs). On Mountain Avenue north of 14th Street, the project consists of approximately 1,000 linear feet of 84-inch diameter storm drain with depth to invert at about 15 feet bgs. On Benson Avenue from the basin to the 13th Street, the project consists of approximately 1,100 linear feet of either 42-inch or 66-inch diameter storm drain with depth to invert at about 16 feet bgs.

3.0 SCOPE OF WORK

The scope of this investigation included the following tasks:



SITE LOCATION MAP

14th Street Water Quality Regional Facility

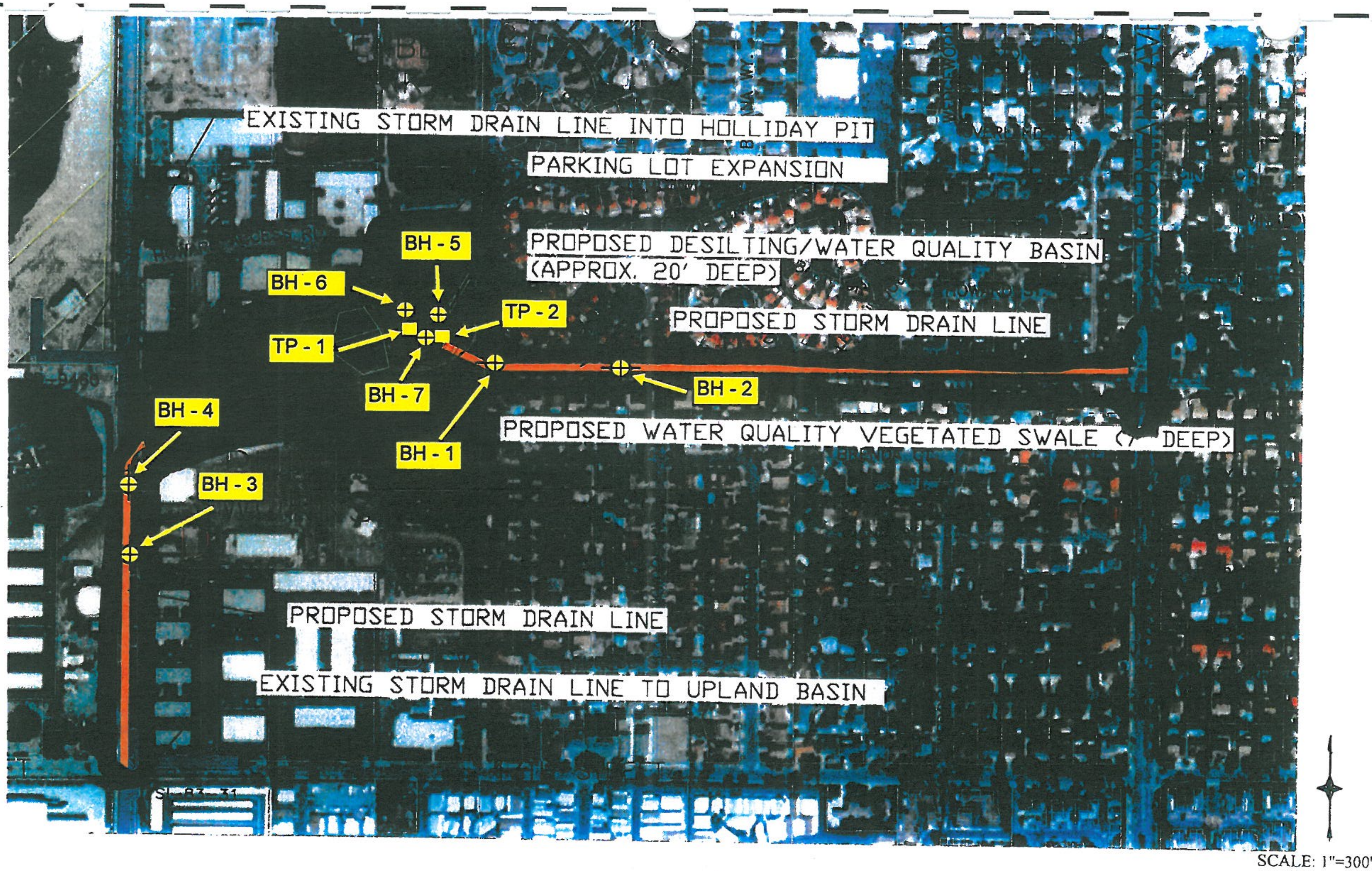
City of Upland, California

For: AECOM Water

Project Number: 10-81-227-01

Converse Consultants

Figure No. 1



APPROXIMATE BORING AND TEST PIT LOCATION MAP

14th Street Water Quality Regional Facility

City of Upland

For: AECOM Water

Project Number: 10-81-227-01

Figure No. 2

Converse Consultants

3.1 Literature Review/Site Reconnaissance

A Converse geologist and engineer reviewed available geologic and soils information and conducted a site reconnaissance. The purpose of the site reconnaissance was to observe surface conditions and to select preliminary exploratory borings locations.

3.2 Exploratory Borings

Seven (7) exploratory borings (BH-1 through BH-7) were drilled on July 7, 2010. Borings BH-1 and BH-2 were drilled along the 14^{th} Street, boring BH-3 on Benson Avenue, and boring BH-4 on 5^{th} Avenue. Borings BH-5 through BH-7 were drilled at the desilting/water quality basin location. Borings were planned to be drilled to a maximum depth of 25 feet bgs in order to observe the soils near the planned bottom elevation of the basin and the planned pipeline elevations. Six of the borings encountered refusal on oversized rock at depths of 2.0 to 4.5 feet bgs. Boring BH-4 encountered an unknown utility at 6.5 feet bgs. Pictures showing cobbles and boulders encountered during the subsurface exploration are presented in Photo Nos. 1 through 3.

To supplement subsurface information from borings on July 13, 2010, two exploratory test pits (TP-1 and TP-2) were excavated at the basin location to a maximum depth of nine (9) feet bgs. To obtain percolation rate at the desalting/water quality basin location, percolation tests were conducted on both test pits at 9 feet bgs. The procedure and results for the percolation test is presented in Appendix C, *Percolation Test Results.*

Our engineer and geologist visually logged the subsurface conditions encountered in the exploratory borings and test pits at the time of drilling and collected soil samples.

For a description of the field exploration and sampling program, see Appendix a, *Field Exploration*.

3.3 Laboratory Testing

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

- *In Situ* Moisture Contents and Dry Densities (ASTM Standard D2216)
- Sand Equivalent (ASTM Standard D2419)
- Soil Corrosivity Tests (Caltrans 643, 422, 417, and 532)
- Grain Size Distribution (ASTM Standard D422)

Photo No. 1. Pictures showing general view of proposed desilting/water quality basin site.

Photo No. 2. Pictures showing cobbles and boulders encountered during subsurface exploration.

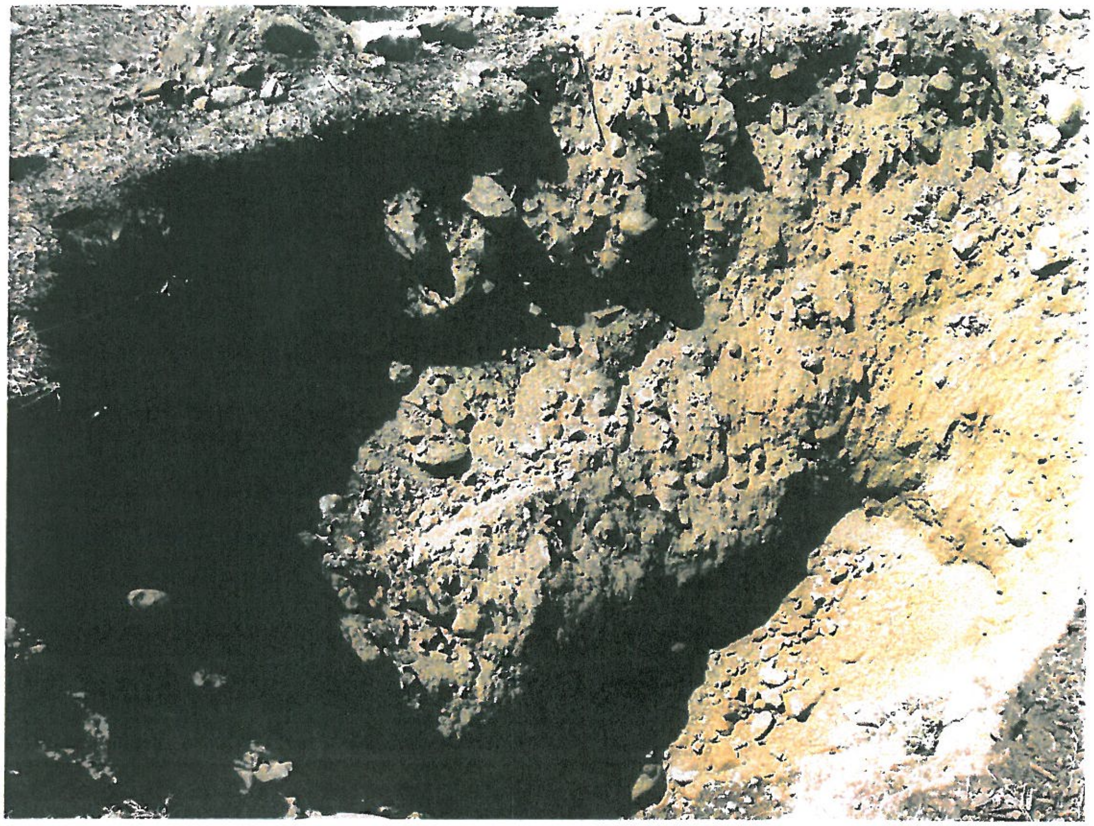


Photo No. 3. Pictures showing cobbles and boulders in the proposed test pit excavation at the desilting/water quality basin site.

- Maximum Dry Density and Optimum-Moisture Content Relationship (ASTM Standard D1557)
- Direct Shear Tests (ASTM Standard D3080)

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

3.4 Analysis and Report Preparation

Data obtained from the field exploration and laboratory testing program were compiled and evaluated. Geotechnical analyses of the compiled data were performed and this report was prepared to present our findings, conclusions and recommendations for the proposed storm drain and desalting/water quality basin.

4.0 SITE CONDITIONS

A general description of the subsurface conditions and various materials encountered during our field exploration along the alignment and the basin location are presented in this section.

4.1 Geologic Setting

The project site is located within the Chino Basin near the northwestern boundary of the Peninsular Ranges Geomorphic Province of Southern California.

The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the south by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto, Cucamonga, and San Andreas Fault Zones, all of which have been known to be active during Quaternary time.

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

The Chino Basin is a broad alluvial valley bounded by the San Gabriel Mountains on the north, the San Bernardino Mountains on the east and northeast, the Santa Ana Mountains on the southwest, and the Puente Hills on the west. The thickness of the alluvium is more than 800 feet in the central area of the basin with a maximum thickness of 1,300 feet near the Ontario area.

4.2 Subsurface Conditions

Based on our field observation and site exploration data, the proposed storm drain will be constructed along asphalt concrete paved streets. Alluvial soil encountered below the asphalt concrete paved streets consisted of sand, gravel and cobbles to the maximum depth explored of approximately 6.5 feet bgs.

The basin location is underlain by fill material comprised of mixtures of silty sand, gravel, cobbles and boulders to a depth of about 6 feet bgs. Below the fill layer, native alluvial deposits consisting of sand, gravel, cobbles, and boulders were encountered to a maximum depth explored of 9 feet bgs. The fill soils were similar in composition to the native soils.

Because the depth of our subsurface investigation was limited by extremely rocky conditions, it is necessary to infer the conditions between the maximum depth explored of 9 feet bgs, and the depth of the planned improvements, approximately 25 feet bgs.

Regional geologic mapping (Morton and Miller, 2006) indicates that the site is underlain by young alluvial fan deposits. The western portion of the project area is mapped as late Holocene deposits (unit Qyf_5) consisting of "unconsolidated to slightly consolidated coarse-grained sand to boulder alluvial-fan deposits having slightly dissected to essentially undissected surfaces." The eastern portion of the project area is mapped as middle Holocene deposits (unit Qyf_3) consisting of "slightly to moderately consolidated silt, sand, and coarse-grained sand to boulder alluvial-fan deposits having slightly to moderately dissected surfaces." The contact between these two very similar alluvial units runs north-south through the proposed basin area.

USDA soil survey data (USDA, 2010) was reviewed. Soil maps are of limited use in evaluating subsurface conditions because they only provide data for the upper 60 inches. The USDA report indicated that the upper 10 inches of the western portion of the site consists of very stony loamy sand and that the upper 36 inches of the eastern portion of the site consists of gravelly to very gravelly loamy sand. Below a depth of 10 to 36 inches to a maximum reported depth of 60 inches, the entire site is consists of very stony sand.

A large surface mine operated by Holliday Rock is located approximately 1,000 feet northwest of the proposed basin location. During an informal telephone conversation, an engineer from Holliday Rock indicated that the subsurface conditions are generally consistent from the surface to at least 150 feet bgs. He also indicated that freshly excavated areas of their pit do not hold water during rain events. Based on our field observations, our review of regional maps, and our communications with operators of nearby sites, we anticipate that the subsurface conditions at approximately 25 feet bgs in the at the proposed basin location will be generally similar to the conditions observed at 6 to 9 feet bgs. Soil density may increase somewhat with depth, but we anticipate that the rock matrix will limit densification by the overburden. It is our opinion that observations and testing conducted in our test pits are generally applicable to the design of the basin and pipelines.

During our exploration and laboratory testing no odors or other evidence of contaminated soils and/or hazardous materials were noticed based on visual observations. It should be noted, our scope of work did not include any environmental sampling and testing.

For additional information on the subsurface conditions, see Appendix A, *Field Exploration.*

4.3 Existing Pavement Sections

Pavement thicknesses as measured at boring locations are presented in the table below.

Table No. 1, Existing Pavement Sections

Street Name	Boring Number	Asphalt Concrete Thickness, inch	Aggregate Base Thickness, inch	Borehole Representing
14th Street	BH-1	4.0	4.0	Along storm drain pipeline alignment
14th Street	BH-2	4.0	4.0	Along storm drain pipeline alignment
Benson Avenue	BH-3	6.0	0.0	Along storm drain pipeline alignment
Benson Avenue	BH-4	6.0	0.0	Along storm drain pipeline alignment

4.4 Groundwater

Groundwater was not encountered in any borings drilled to the maximum depth of approximately 6 feet bgs and on the test pits excavated to a depth of approximately 9 feet bgs. In April 2009, Los Angeles County Department of Public Works well number 4517H, located approximately 0.5 miles north of the project area, contained groundwater at a depth of 393 feet bgs. The well has been monitored since 1933 with the highest historical groundwater level at 95 feet bgs in 1983. Groundwater is not expected to be encountered during the excavations for this project.

4.5 Excavatability

The earth materials should be excavatable by conventional heavy-duty earth moving and trenching equipment. <u>A significant amount of oversized materials consisting of gravel (up to 3 inches in diameter), cobbles (3 to 12 inches in diameter) and boulders (12 to 24 inches in diameter and occasionally larger) will be encountered within excavations. These materials will require special handling and disposal</u>.

4.6 Percolation testing for desilting/water quality basin

Two percolation tests were conducted on the test pits at the desilting/water quality basin site. Due to the rocky conditions, standard test methods such as California Test 750 (CalTrans), USBR 7300-89 (well permeameter), or ASTM D3385 (double-ring infiltrometer) were not applicable. An improvised continuous (falling head) test procedure was utilized for percolation testing, as outlined in the Appendix C, *Percolation Test Results*.

The result of the percolation tests indicated that the site soil at about 9 feet below ground surface has a percolation rate of approximately 8 inches per hour.

Based on laboratory test data and information provided by AECOM Water, the site soils are in the Soil Conservation Service texture class of “Sand”, which typically implies a minimum infiltration rate of approximately 8 inches per hour. A typical infiltration rate of over 8 inches per hour is consistent with Converse’s testing and also with informal communication that the pits at Holliday Rock do not retain water during rain events until they have become silted.

Based on information provided by AECOM Water, we understand that other basins in the vicinity of the project site have recharge rates in the range of 0.5 to 1.5 inches per hour. These historical recharge rates may reflect reduced infiltration due to compaction during grading or siltation during basin usage. While the natural percolation rate is approximately 8 inches per hour or more, the actual future basin performance may be more comparable to nearby basins, depending on construction and maintenance procedures.

4.7 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions along the storm drain alignment and at the basin location should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring and test pit locations.

5.0 LABORATORY TEST RESULTS

5.1 Physical Testing

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

- *In-situ* Moisture and Dry Density – *In-situ* dry density and moisture content of soils measured for BH-4 was 120 pounds per cubic feet (pcf) and 5 percent, respectively. Result of *in-situ* moisture and dry density test is presented on the Logs of Borings in Appendix A, *Field Exploration.*

- Sand Equivalent – One (1) representative sample of the site soil from six (6) to nine (9) feet below ground surface (bgs) was tested to evaluate Sand Equivalent (SE) in accordance with the ASTM Standard D24A test method. The test result is presented in Table No. B-1, *Sand Equivalent Test Results,* in Appendix B, *Laboratory Testing Program.* The value of the measured SE was 64.

- Grain Size Analysis – Three (3) representative samples were tested to determine the relative grain size distribution according to ASTM Standard D422. Test results indicated that the sample tested can be classified as Sandy Gravel and Gravelly Sand. The texture of the sub-gravel fraction of the soil may be classified as Sand under the USDA Soil Conservation Service criteria. Test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results,* in Appendix B, *Laboratory Testing Program.*

- Maximum Dry Density and Optimum Moisture Content – Result of one (1) typical moisture-density relationship of representative soil sample tested according to ASTM Standard D1557 are presented in Drawing No. B-2, *Moisture-Density Relationship Results,* and in Table No. B-2, *Maximum Dry Density Test Results* in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 132.0 pounds per cubic feet (pcf) and optimum moisture content was 8.0.

- Direct Shear – One direct shear test was performed according to ASTM Standard D3080. Results of the direct shear tests are presented in Drawing No. B-3, *Direct Shear Test Results,* and in Table No. B-4, *Direct Shear Test Results,* in Appendix B, *Laboratory Testing Program.*

5.2 Chemical Testing - Corrosivity Evaluation

One (1) representative sample of the site soil was tested for corrosivity according to Caltrans test methods (643, 422, 417 and 532) by Schiff Associates of Claremont, California. Test was conducted to evaluate corrosivity with respect to common construction materials such as concrete and steel. The test results are discussed below and are presented in Appendix B, *Laboratory Testing Program.* The test includes pH, sulfate and chloride content, and saturated minimum electrical resistivity.

The maximum sulfate content of the sample tested was 27 mg/kg, which indicated that site soil is not deleterious to concrete.

The chloride concentration was 2.0 ppm. The pH value of the site soil was 7.9. The measured value of the minimum electrical resistivity when saturated was 12,400 Ohm-cm. These values indicate that the site soil is "Mildly Corrosive" for ferrous metals in contact with the soil.

6.0 FAULTING AND SEISMICITY

6.1 Faulting

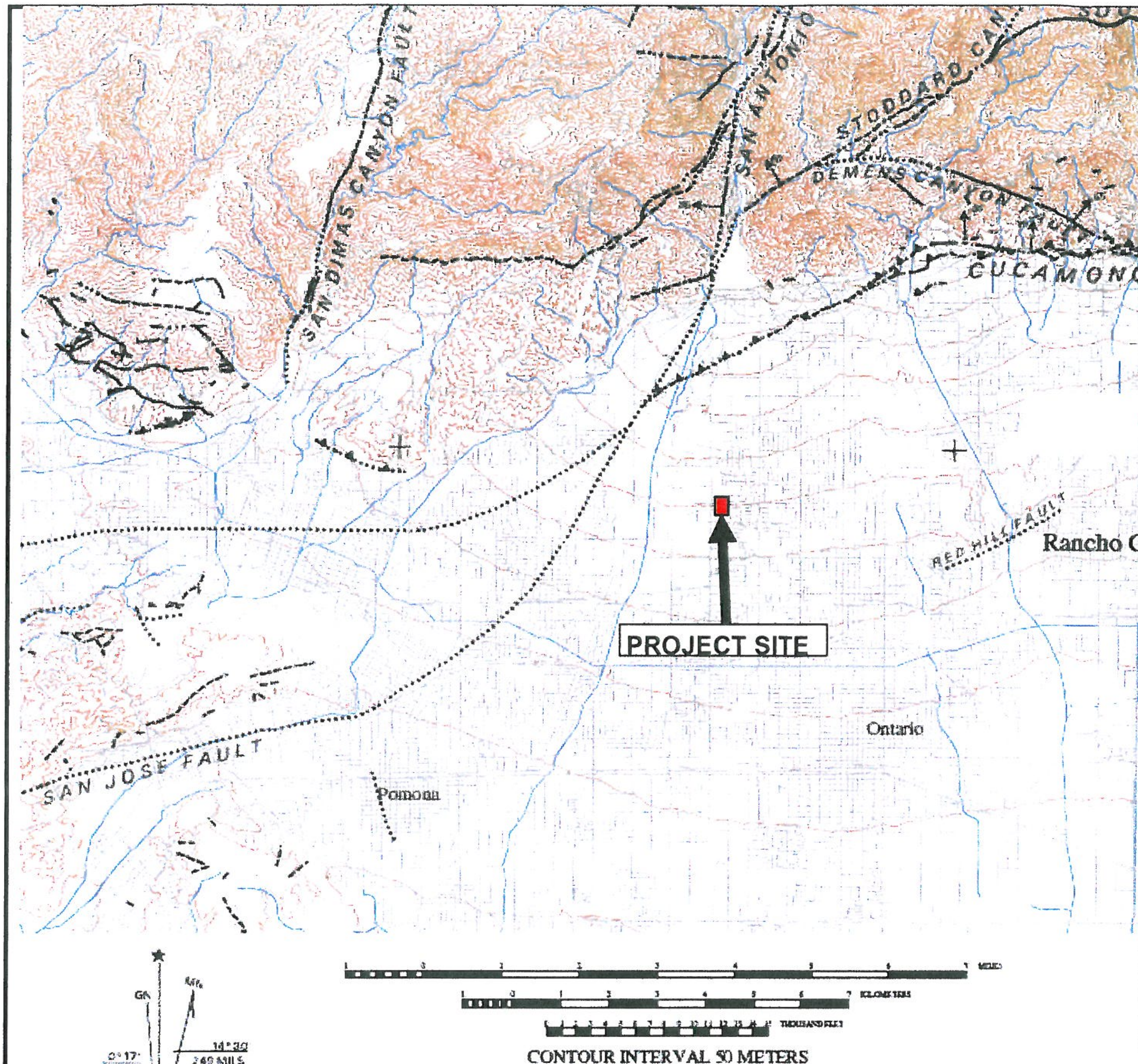
The site is not located within a currently designated State of California Earthquake Fault Zone. Based on a review of existing geologic information no known active surface fault zone crosses or projects toward the site. For fault locations see Figure No. 3, *Fault Location Map.*

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

The following Table No. 2, *Seismic Characteristics of Regional Active Faults*, contains a list of active and potentially active faults within 100 kilometers of the subject sites.

Table No. 2, Seismic Characteristics of Regional Active Faults

Fault Name	Closest Distance To The Site, (km)	Seismic Source Type	Moment Magnitude (Mw)	Slip Rate (mm/year)
SAN JOSE	0.9	B	6.5	0.50
CUCAMONGA	3.1	B	7.0	5.00
SIERRA MADRE (Central)	5.5	B	7.0	3.00



Adapted from Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangle, California Geologic Survey Open File Report 2006-1217.

FAULT LOCATION MAP

14th STREET WATER QUALITY REGIONAL FACILITY	Project Number: 10-81-227-01
City of Upland, San Bernardino County, California	**Figure No.:**
For: AECOM Water	**3**

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Fault Name	Closest Distance To The Site, (km)	Seismic Source Type	Moment Magnitude (Mw)	Slip Rate (mm/year)
CHINO-CENTRAL AVE. (Elsinore)	10.8	B	6.7	1.00
CLAMSHELL-SAWPIT	20.8	B	6.5	0.50
SAN JACINTO-SAN BERNARDINO	21.6	A	6.7	12.00
ELSINORE-WHITTIER	25.1	A	6.8	2.50
SAN ANDREAS - Southern	25.6	A	7.4	24.00
SAN ANDREAS - 1857 Rupture	25.8	A	7.8	34.00
ELSINORE-GLEN IVY	29.1	A	6.8	5.00
CLEGHORN	29.6	B	6.5	3.00
RAYMOND	30.5	B	6.5	0.50
VERDUGO	39.4	B	6.7	0.50
SAN JACINTO-SAN JACINTO VALLEY	42.2	A	6.9	12.00
NORTH FRONTAL FAULT ZONE (West)	42.6	B	7.0	1.00
HOLLYWOOD	50.7	B	6.5	1.00
NEWPORT-INGLEWOOD (L.A. Basin)	55.2	B	6.9	1.00
SIERRA MADRE (San Fernando)	59.4	B	6.7	2.00
SAN GABRIEL	59.8	B	7.0	1.00
ELSINORE-TEMECULA	60.6	A	6.8	5.00
NEWPORT-INGLEWOOD (Offshore)	62.0	B	6.9	1.50
PALOS VERDES	66.7	B	7.1	3.00
HELENDALE - S. LOCKHARDT	73.7	B	7.1	0.60
MALIBU COAST	78.5	B	6.7	0.30
SANTA SUSANA	78.6	B	6.6	5.00
SAN JACINTO-ANZA	81.6	A	7.2	12.00
NORTH FRONTAL FAULT ZONE (East)	81.6	B	6.7	0.50
HOLSER	86.8	B	6.5	0.40
PINTO MOUNTAIN	88.3	B	7.0	2.50
ANACAPA-DUME	94.5	B	7.3	3.00
CORONADO BANK	96.7	B	7.4	3.00
LENWOOD-LOCKHART-OLD WOMAN SPRGS	97.5	B	7.3	0.60

6.2 *CBC (2007) Seismic Design Coefficients*

Seismic parameters based on 2007 California Building Code and site coordinates 34.11 north latitude and 117.68 west longitude are provided in Table No. 3. Site coordinates for a position near by Benson Avenue and 14th Street has been selected to represent the project site.

Table No. 3, CBC Seismic Design Parameters

Seismic Parameters	Values
Site Class	"D"
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	2.22g
Mapped 1-second Spectral Response Acceleration, S_1	0.73g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5
Design Spectral Response Acceleration for short period, S_{ds}	1.48g
Design Spectral Response Acceleration for 1-second period, S_{d1}	0.73g

6.3 Secondary Effects of Seismic Activity

A buried pipeline is subjected to dynamic stresses due to ground acceleration during earthquake events. A seismic event may also affect a buried pipeline through ground surface rupture, soil liquefaction, landslides, lateral spreading, earthquake-induced flooding, tsunamis, and seiches. A discussion on a site-specific evaluation of each of these seismic effects is presented below:

Surface Fault Rupture: The proposed storm drain alignment and desilting/water quality basin site is not located within a currently designated State of California Earthquake Fault Zone. Based on review of existing geologic information, no major surface fault crosses the alignment. Although there is always a risk of ground rupture from a previously unknown fault, the risk of such an event at the project site is considered very low.

Soil Liquefaction: Liquefaction is defined as the phenomenon in which a cohesionless soil mass within about the upper 50 feet of the ground surface suffers a substantial reduction in its shear strength, due the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction occurs in submerged granular soils during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows:

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

Historical data indicates depth to groundwater is more than 90 feet bgs. The probability of wide area of soil beneath the basin site to be fully saturated and seismic event happening at the same time is low. The site contains significant quantity of cobbles and boulders, hence, the probability of developing significant pore water pressure during seismic event is considered to be low for the soils beneath the basin site. Hence, the potential for liquefaction induced damage is considered to be low.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The ground along the storm drain alignment is relatively flat. The project site has a low potential for seismically induced landslides affecting the area. For basin with side slopes 2:1 and 4:1 (H:V), the potential for slides is low if the slopes are well protected and maintained.

Lateral Spreading: Seismically induced lateral spreading involves lateral movement of earth materials due to ground shaking. It differs from a slope failure in that ground failure involving a large movement does not occur due to the flatter slope of the initial ground surface. Lateral spreading is characterized by near-vertical cracks with predominantly horizontal movement of the soil mass involved over the liquefied soils. The potential for lateral spreading along the alignment and basin site is considered low.

Earthquake-Induced Flooding: The potential for seismically induced flooding affecting the storm drain alignment and desilting/water quality basin is considered to be low.

Tsunamis: Tsunamis are tidal waves generated by fault displacement or major ground movement. Based on the location of the site, tsunamis do not pose a hazard.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on the location of the site, seiches do not pose a hazard.

7.0 EARTHWORK RECOMMENDATIONS

7.1 General

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

Deleterious material, including organics, asphalt, and debris generated during excavation should not be placed as backfill. Oversized materials (larger than 3 inches) should not be used as trench backfill.

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

The soils consultant should provide continuous observation and testing services during subgrade preparation, trench backfill, placement, and compaction phases of the project in order to ensure compliance with the design concepts, specifications and earthwork recommendations presented in this report and to allow revision in the event the subsurface conditions differ from those anticipated.

7.2 Storm Drain Subgrade Preparation

Any soft and/or unsuitable materials encountered at the storm drain subgrade should be removed and replaced with compacted fill or an adequate bedding material.

For a majority of the proposed storm drain alignment, the subsurface materials at the proposed invert depths should be suitable as pipe subgrade. The final subgrade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipeline placed on bedding material. Gravel, cobbles and boulders are expected to be encountered along with fine to coarse grained sand. Protruding oversize particles larger than two (2) inches in dimension should be removed from the trench bottom and replaced with compacted on-site materials free of oversize particles.

7.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface.

Gravels and cobbles were encountered below the pavement along major portion of the proposed pipelines alignments. These materials will require special handling and disposal. Excavated on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site.

The project soils consultant should provide continuous observation and testing services during subgrade preparation, backfill/fill placement and compaction in order to ensure compliance with the project specifications.

The following recommendations are provided as a guide for the field quality control of the trench backfill. Additional consultation may be required during construction.

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

- Trench zone backfill shall be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method. At least the upper one (1) foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method, or as required by the City of Upland.

- Particles larger than one (1) inch should not be placed within 12 inches of the top of pipe or pavement subgrade. Rocks larger than three (3) inches in the largest dimension should not be placed as trench backfill. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil.

- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within three (3) percent of optimum moisture content for sandy soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

- The field density of the compacted soil should be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

- Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction. The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.

- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

7.4 Select Imported Fill Materials

Imported soils, if any, used as trench or pit 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
- Have a Plasticity Index of 10 or less

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

7.5 Excavation for Desilting/Water Quality Basin

The desilting/water quality basin depth is expected to be about 25 feet bgs at the southern end and about 45 feet bgs at the northern end. Test pit excavation showed that the basin site is underlain by fill material comprised of mixtures of silty sand, gravel, cobbles and boulders to a depth of about 6 feet bgs. Below the fill layer, native alluvial deposits consisting of sand, gravel and cobbles were encountered to the maximum depth explored of 9 feet bgs.

Because of the presence of dry sandy soil with gravel and cobbles, the final slopes for the basin should have a 2:1 horizontal:vertical (H:V) or flatter slope ratio. It is our understanding that the basin slopes are generally 4:1 (H:V) with small areas at 2:1 (H:V). The slopes should be observed by the geotechnical consultant during grading to determine whether loose sand, excessive oversize rock, or other unfavorable conditions are present at the slope face. If such conditions are observed in slopes steeper than 4:1 (H:V), additional compaction, a replacement fill slope, or other remedial measures may be recommended. Slope protection measures are presented in Section 8.8, *Desilting/Water Quality Basin Site Slope Protection.*

In order to maintain the anticipated infiltration rate, the density of the basin floor should remain approximately the same as prior to grading. Additional compaction of the basin floor should be avoided. The size of the earth working equipment used to excavate the bottom 2 feet of the basin should be limited to the minimum size required to efficiently excavate the encountered soils. If the density of the basin floor after excavation exceeds the in-situ density of nearby undisturbed soils, the basin floor may be scarified to a depth of 24 inches to restore permeability. Alternatively, 24 inches of soil may be excavated from the basin floor and replaced without additional compactive effort.

8.0 DESIGN RECOMMENDATIONS

8.1 General

Subsurface information below nine (9) feet bgs was not obtained for this project due to limitations on deep excavations. We have assumed that the soil conditions below nine (9) feet to be similar to the soil conditions encountered up to nine (9) feet bgs. If substantial changes in soil conditions occur during the construction, project geotechnical engineer should be consulted immediately for further evaluation and recommendations.

The following design recommendations are based on our analysis of the data obtained during field investigation, laboratory testing, and our understanding of the proposed project.

8.2 Lateral Earth Pressures and Resistance to Lateral Loads

The following subsections outline lateral earth pressures and resistance to lateral loads. Lateral earth pressures and resistance to lateral loads are estimated by using on-site soils with a total unit weight of 132 pounds per cubic foot (pcf).

8.2.1 Active Earth Pressures

The active earth pressure behind any buried wall depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures. Presence of expansive soils may result in additional pressures due to soil expansion. The lateral earth pressures are presented in the following table.

Table No. 4, Lateral Earth Pressures

Loading Conditions	Equivalent Fluid Pressure
Active earth conditions (wall is free to deflect at least 0.001 radian) -	40 pcf
At-rest (wall is restrained) -	60 pcf
Seismic	16H (at the top of the reverse triangle, where H is the design height of the wall)

These pressures assume a level ground surface behind the walls for a distance greater than the walls height, no surcharge, no hydrostatic pressure, and soil expansion index less than 30. Adequate drainage could be provided to avoid hydrostatic pressure build-up against the walls. When hydrostatic pressures buildup, the incremental earth pressures below the water level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

8.2.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by combination of friction acting at the base of foundations and passive earth pressure. A coefficient of friction of 0.4 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 300 psf per foot of depth may be used for resistance against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 3,000 psf.

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

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

8.3 Soil Parameters for Design

Structural design of pipeline and basin requires proper evaluation of all possible loads including dead and live or transient loads. The stresses and strains induced on the depend on many factors, including the type of soil density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils.

The recommended values of the various soil parameters for the pipe design and basin design are provided below:

- Average compacted fill unit weight — γ=132 pcf
- Angle of internal friction of soils — $\phi = 33^{o}$
- Soil cohesion — c = 0 psf
- Coefficient of friction between backfill and native soils — fs = 0.4
- Bearing pressure against alluvial Soils — 3,000 psf
- Coefficient of passive earth pressure — Kp = 3.4

8.4 Modulus of Soil Reaction (E')

Deflection control in flexible pipe installation involves assessment of the deflection occurring during installation as well as that occurring due to the service loads (i.e. soils and superimposed loading). The modulus of soil reaction presented below is developed using U. S. Bureau of Reclamation method. The modulus of soil reaction is based on evaluation of both native soils at the proposed pipe invert depth and assuming that sandy or gravely material will be used for pipe bedding with no more than 30 percent of fines. We also assumed the pipe bedding will be densified to about 85 to 90 percent of laboratory maximum dry density. We recommend the Modulus of Soil Reaction, E' be 1500 pounds per square inch.

8.5 Bearing Pressure for Anchor and Thrust Blocks

Allowable net bearing pressure of 3,000 pounds per square foot may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 24 inches wide.

Resistance to lateral forces can be assumed to be provided by friction at the base of thrust blocks and by passive earth pressure. An ultimate value of coefficient of friction of 0.4 may be used between the thrust block and the supporting natural soil or compacted fill. A passive earth pressure of 300 psf per foot of depth may be used for the sides of thrust blocks or anchors poured against undisturbed or recompacted soils. The value of the passive lateral earth pressure should be limited to 3,000 psf. Frictional and passive resistance can be combined for the design of anchors and thrust blocks.

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

8.6 Inlet and Outlet Structures for Desilting/Water Quality Basin

Basin inlet and outlet structures should be constructed of durable materials, such as concrete or masonry block. Corrugated metal pipe (CMP) and plastic (HDPE) risers and drain pipes are popular in engineering design, but are susceptible to crushing and flotation in basins. A concrete inlet or outlet structure is generally preferable to a masonry block structure because it is sturdier and more durable. Provisions should be made for sufficient reinforcement and anchoring.

The specific flow-controlling elements of an inlet or outlet structure may include one or more of the following: a circular or noncircular orifice; a rectangular, trapezoidal, triangular or V-notch weir, culvert entrance control or a riser overflow opening.

The proposed inlet and outlet structures may be supported on continuous (strip) spread footings. Actual footing size should be determined based on structural requirements. Continuous spread footings should be at least 18 inches wide. The depth of embedment below lowest adjacent soil grade of footings should be at least 18 inches. Footings located on the floor of the basin should be founded on the native soils, scarified; moisture conditioned to near optimum moisture content, and compacted to a minimum of 90 percent of the laboratory maximum dry density.

An allowable net bearing capacity of 3,000 pounds per square foot (psf) may be used.

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

The net allowable bearing value indicated above is for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

The static settlement of structures supported on strip and/or spread footings will depend on the actual footing dimensions and the imposed vertical loads. For structures designed for 3,000 pounds per square foot bearing pressure, settlement in the range of ¼-inch to ½-inch should be anticipated. Due to the granular nature of the subsurface soils, most of the settlement should occur during construction. In order to evaluate differential settlement, data on the relative dimension of adjacent footings, magnitude of imposed loads and distance between footings is needed. In the absence of such data, and based on our experience for similarly loaded footings, the differential settlement may be taken as equal to about one half of the total settlement over a horizontal distance of 50 feet.

8.7 Shoring Design

The shoring for the pipeline excavations may be cantilevered consisting of conventional soldier piles and lagging or sheet piles, or may be laterally supported by walers and cross bracing. Drilled excavations for soldier piles may require the use of drilling fluids to prevent caving and to maintain an opened hole for pile installation due to sandy soils. Design of shoring requires proper evaluation of all possible loads acting on the walls, including dead and live or transient loads. The stresses and strains induced on the walls depend on many factors, including the type of soil density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils.

For the design of cantilever shoring supporting a level grade, an equivalent fluid pressure of 40 psf/ft of depth below grade may be used. Braced shoring should be designed to support a uniform rectangular lateral earth pressure of 28 psf, based on Figure No. 4, *Recommended Lateral Earth Pressure for Braced Excavation*.

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

For the design of soldier piles spaced at least two diameters on-center, the passive resistance of the soils adjacent to the piles may be assumed to be 400 psf/ft of embedment depth. Soldier pile members placed in drilled holes should be properly backfilled with a sand/cement slurry or lean concrete in order to develop the required passive resistance. Contractor should have special provisions for soldier pile removal. For sheet piles, a passive resistance of 300 psf/ft of embedment, up to a maximum of 3,000 psf, may be used.

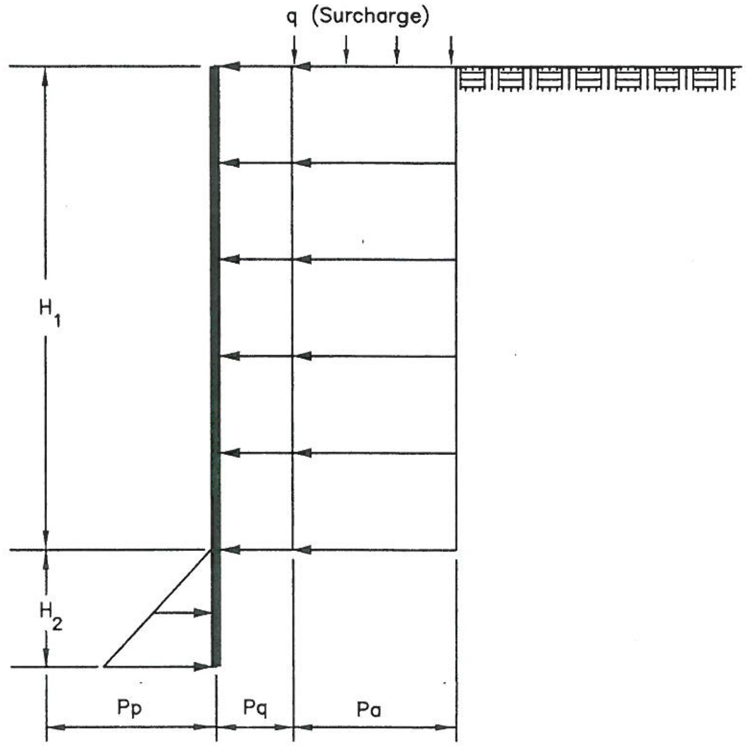
The lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier piles. Accordingly, the lagging between the piles may be designed for a nominal pressure of up to a maximum of 400 psf.

8.8 Desilting/Water Quality Basin Site Slope Protection

Rock or vegetation shall be used to protect the basin inlet and slopes against erosion. The outflow from the basins shall be provided with outlet protection to prevent erosion and scouring of the embankment and channel. Structure shall be placed on a firm, smooth foundation with the base securely anchored with concrete or other means to prevent floatation. Chain link fencing should be provided around each sediment/desilting basin to prevent unauthorized entry to the basin or if safety is a concern.

Following are the maintenance and inspection recommendations for desilting/water quality basin.

TEMPORARY BRACED EXCAVATION
LATERAL EARTH PRESSURE



$$P = Pq + Pa$$
$$= 0.5q + 28H_1 \text{ (350 psf minimum)}$$ – active earth pressure

$$Pp = 300\ H_2 \leq 3000 \text{ psf}$$ – passive earth pressure (on native or compacted soils)

$$\mu = 0.4$$ – allowable friction coefficient

Notes:

1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp and Pa are the passive and active earth pressure respectively; Pq is the incremental surcharge earth pressure; and μ is the allowable friction coefficient, applied to dead normal loads acting on non–pile supported elements.
3. Earth pressures assume no hydrostatic pressures. If hydrostatic pressures are allowed to build up, the incremental earth pressures below the ground–water level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.
4. Pp includes a safety factor of 1.5.
5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement of slab.
6. For traffic surcharge, use a uniform pressure of 100 psf over the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURE FOR BRACED EXCAVATION

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10–81–227–01

Converse Consultants

Figure No. 4

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- Inspect sediment/desilting basins before and after rainfall events and weekly during the rest of the rainy season.
- During extended rainfall events, inspect at least every 24 hours.
- Examine basin banks for seepage and structural soundness.
- Check inlet and outlet structures and spillway for any damage or obstructions.
- Install an emergency drain with a valve into the outlet box so that the basin can be drained in case of emergency.
- Remove standing water from the basin within 72 hours after accumulation. Sub-drains (trench drains filled with large diameter cobbles and gravel) could be installed along the basin floor. These drains would increase the overall recharge rate of the basin serving the basin to drain faster.
- Repair damage and remove obstructions as needed.
- Check inlet and outlet area for erosion and stabilize if required.
- Remove accumulated sediment when its volume reaches one-third the volume of the sediment storage.
- Properly dispose of sediment and debris removed from the basin.

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 General

Since the storm drain alignment will be within existing street, sloped excavations may not be feasible. Where the side of the excavation is vertical cut, it should be adequately supported by temporary shoring to protect workers and adjacent structures.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1989, current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be necessary.

Per California Code of Regulations, Title 8, Section 1540, Excavation, Subchapter 4, *Construction Safety Orders*, sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.

The subsurface exploration for this project indicates presence of mostly non-cohesive mixtures of sand and silt combined with gravel and cobbles. To guard against caving, raveling or running ground conditions, the Contractor must install appropriate excavation support. Any voids created by shoring withdrawal should be backfilled or filled with slurry.

9.2 *Temporary Sloped Excavations*

Temporary open-cut excavation pits may be constructed for storm drain alignment with side slopes as recommended in Table No. 5, *Slope Ratios for Temporary Excavations.* Temporary cuts encountering soft and unsuitable soils, dry loose cohesionless soils or loose trench backfill may have to be constructed at a flatter gradient than presented below.

Table No. 5, Slope Ratios for Temporary Excavations

Depth of Cut (feet)	Recommended Maximum Slope (Horizontal: Vertical)[1]
0-4	1:1
4-20	1.5:1

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

10.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

During excavations and subgrade preparation for the proposed improvements, placement of the storm drain and trench backfill, and construction of desilting/ water quality basin, a geotechnical representative should be present to observe soil conditions and test the density and moisture of the backfill.

It should be noted that subsurface information below nine (9) feet bgs was not obtained for this project due to limitations on deep excavations. We have assumed that the soil conditions below nine (9) feet to be similar to the soil conditions encountered up to nine (9) feet bgs. If substantial changes in soil conditions occur during the construction, project geotechnical engineer should be consulted immediately for further evaluation and recommendations.

11.0 CLOSURE

This report has been prepared for the exclusive use of AECOM Water to assist in the design and construction of the proposed project. Any reliance on this report by third parties shall be at third party's sole risk. Our services have been performed in accordance with applicable state and local ordinances, and generally accepted practices within our profession.

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

Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field investigations and laboratory tests, combined with interpolation and extrapolation of soil conditions between and beyond the boring locations.

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APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting drilling test borings. During the site reconnaissance, the surface conditions were noted and the locations of the test borings and test pits were determined. The test borings were located by pacing or by rough measurements relative to existing streets and boundary features and should be considered accurate only to the degree implied by the method used. Discussion on the field investigation methods are presented below.

Borings

Seven (7) exploratory borings (BH-1 through BH-7) were drilled on July 7, 2010. All of the borings could not be drilled to the planned depth due to auger refusal on sand with gravel and cobbles. The borings were terminated from 2 to 4 feet below ground surface (bgs). Boring BH-4 was terminated at 6.5 feet bgs due to existing unknown underground utility. Borings were backfilled with loose soil cuttings.

Test borings were advanced using truck mounted drill rigs equipped with 8-inch diameter hollow-stem augers for soil sampling. Soil encountered in the borings were logged by a Converse representative and classified in the field by visual examination in accordance with the Unified Soil Classification System (ASTM 2488). The field descriptions have been modified where appropriate to reflect laboratory test results.

Test Pits

Two test pits (TP-1 and TP-2) were excavated on the basin site on July 13, 2010 to a maximum depth of 9 feet bgs. The test pits were excavated using a backhoe equipped with a 24-inch wide bucket and a sampler. Soils were continuously logged and classified in the field by visual examination in accordance with the Unified Soil Classification System. The field descriptions have been modified where appropriate to reflect laboratory test results.

Relatively undisturbed ring and disturbed bulk samples of the subsurface soil were obtained from the borings and test pits where possible. The relatively undisturbed samples were obtained using California Modified Samplers (2.4 inches inside diameter and 3.0 inches outside diameter) lined with thin sample rings. The sampler was driven into the bottom of the borehole with successive drops of a 140-pound hammer falling 30 inches by means of automatic trip hammer. The number of successive drops of the driving weight ("blows") required for every six (6) inches for a total of 18 inches of penetration of the sampler are shown on the Logs of Borings in the "blows" column.

The sum of the blow counts for the second and third 6-inch penetration for California Modified Sampler as well as for Standard Penetration Test and the corresponding apparent density/consistency of the soil are presented in Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*.

The soil was retained in brass rings (2.4 inches in diameter and one-inch in height). The bottom portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to the laboratory. Bulk soil samples were collected in plastic bags and brought to the laboratory.

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

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For Logs of Borings, see Drawing Nos. A-2 through A-8, *Logs of Borings*. For Logs of Test Pits, see Drawing Nos. A-9 and A-10, *Logs of Test Pits*.

Percolation Tests

To obtain the percolation rate of water on the proposed basin location, percolation tests were performed on the test pits. Percolation test procedure and results are presented in Appendix C, *Percolation Test Results*. Test pits were backfilled loose with spoils and wheel rolled on the same day.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

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

LABORATORY TESTING ABBREVIATIONS

TEST TYPE

(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h
Disturb	Dist.

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er
Permeability	perm
Soil Cement	sc

Apparent Density	Very Loose	Loose	Medium	Dense	Very Dense
SPT (N)	< 4	4 - 11	11 - 30	31 - 50	> 50
CA Sampler	< 5	5 - 12	13 - 35	36 - 60	> 60
Relative Density (%)	< 20	20 - 40	40 - 60	60 - 80	> 80

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
SPT (N)	< 2	2-4	5-8	9-15	16-30	> 30
CA Sampler	< 3	3-6	7-12	13-25	26-50	> 50

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10-81-227-01

Drawing No. A - 1

Log of Boring No. BH - 1

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1446 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		4" ASPHALT CONCRETE / 4" AGGREGATE BASE						
		ALLUVIUM (Qal): **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, some silt, brown.						ma
		Boring located on 14th Street. Auger refusal at 4 feet. No groundwater encountered during drilling. Borehole backfilled with soil cuttings and surface patched with asphalt concrete on 7-7-2010.						

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. **10-81-227-01**

Drawing No. **A - 2**

Log of Boring No. BH - 2

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1448 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		4" ASPHALT CONCRETE / 4" AGGREGATE BASE						
		ALLUVIUM (Qal): **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, few cobbles, brown.						
		Boring located on 14th Street. Auger refusal at 2.5 feet. No groundwater encountered during drilling. Borehole backfilled with soil cuttings and surface patched with asphalt concrete on 7-7-2010.						

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10-81-227-01

Drawing No. A - 3

Log of Boring No. BH - 3

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1420 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		6" ASPHALT CONCRETE / NO AGGREGATE BASE						
		ALLUVIUM (Qal): **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, some cobbles, brown.		X				
		Boring located on Benson Avenue. Auger refusal at 2.0 feet. No groundwater encountered during drilling. Borehole backfilled with soil cuttings and surface patched with asphalt concrete on 7-7-2010.						

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10-81-227-01

Drawing No. A - 4

Log of Boring No. BH - 4

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1435 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		6" ASPHALT CONCRETE / NO AGGREGATE BASE						
		ALLUVIUM (Qal): **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, brown.						
5		**GRAVELLY SAND (SP):** fine to coarse-grained, gravel up to 3/4" in largest dimension, some silt, brown.			11/20/21	5	120	ds ma,ca,er
		Boring located on Benson Avenue. End of boring at 6.5 feet due to existing unknown underground utility. No groundwater encountered during drilling. Borehole backfilled with soil cuttings and surface patched with asphalt concrete on 7-7-2010.						

Log of Boring No. BH - 5

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1453 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, brown.						
		Boring located at proposed basin area. Auger refusal at 2 feet. No groundwater encountered during drilling. Borehole backfilled loose with soil cuttings on 7-7-2010.						

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10-81-227-01

Drawing No. A - 6

Log of Boring No. BH - 6

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1449 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, brown.						
		Boring located at proposed basin area. Auger refusal at 2 feet. No groundwater encountered during drilling. Borehole backfilled loose with soil cuttings on 7-7-2010.						

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10-81-227-01

Drawing No. A - 7

Log of Boring No. BH - 7

Dates Drilled: 7/7/2010 Logged by: CGJ Checked By: HS

Equipment: CME 75 / 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): ±1449 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		FILL: **SANDY GRAVEL (GP):** fine to coarse-grained sand, gravel up to 2-1/2" in largest dimension, brown.						
		Boring located at proposed basin area. Auger refusal at 2 feet. No groundwater encountered during drilling. Borehole backfilled loose with soil cuttings on 7-7-2010.						

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. **10-81-227-01**

Drawing No. **A - 8**

Log of Boring No. TP - 1

Dates Drilled: 7/13/2010 Logged by: CG Checked By: HS

Equipment: 24" BACKHOE Driving Weight and Drop: -

Ground Surface Elevation (ft): ±1452 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
5		**FILL:** **GRAVELLY SAND (SP):** fine to coarse-grained, gravel up to 3" in largest dimension, some cobbles and boulders, trace silt, brown.						
		ALLUVIUM (Qal): **GRAVELLY SAND (SP):** fine to coarse-grained, gravel up to 3" in largest dimension, some cobbles, brown.						
		End of test pit excavation at 9.0 feet. No groundwater encountered during excavation. Test pit backfilled loose with soil cuttings on 7-13-2010.						

Log of Boring No. TP - 2

Dates Drilled: 7/13/2010 Logged by: CG Checked By: HS

Equipment: 24" BACKHOE Driving Weight and Drop: -

Ground Surface Elevation (ft): ±1450 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
5		**FILL:** **GRAVELLY SAND (SP):** fine to coarse-grained, gravel up to 3" in largest dimension, some cobbles and boulders, trace silt, brown.						
		ALLUVIUM (Qal): **SANDY GRAVEL (GP):** fine to coarse-grained, gravel up to 3" in largest dimension, some cobbles, trace silt, brown.		X				ma,se max
		End of test pit excavation at 9.0 feet. No groundwater encountered during excavation. Test pit backfilled loose with soil cuttings on 7-13-2010.						

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B

LABORATORY TESTING PROGRAM

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

Moisture Content and Dry Density

Results of these tests performed on relatively undisturbed samples, were used to aid in the classification and correlation of the soils and to provide qualitative information regarding soil strength and compressibility. As drive samples could not be collected at all planned locations, moisture content and dry density at depths where drive samples were possible, are presented in the Log of Borings. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Sand Equivalent Tests

One (1) representative soil sample from upper five (5) feet below ground surface (bgs) was tested in accordance with the ASTM Standard D2419 to determine the Sand Equivalent (SE). The test result is presented in the following table.

Table No. B-1, Sand Equivalent Test Result

Test Pit No.	Depth (feet)	Soil Description	Sand Equivalent
TP-2	6.0-9.0	Sandy Gravel (GP)	64

Soil Corrosivity

One (1) representative soil sample was tested in to determine 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 pipe materials. These tests were performed by Schiff Associates in Claremont, California. Test results are presented on the following table.

Table No. B-2, Corrosivity Test Results

Sample Location (Boring/Depth)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-4/3.0-4.5	7.9	27	2.0	12,400

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analyses were performed on three (3) selected samples. Testing was performed in general accordance with the ASTM Standard D422 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*.

Laboratory Maximum Density Tests

Laboratory maximum dry density and optimum moisture content relationship test was performed on one (1) representative bulk sample. The test was conducted in accordance with ASTM Standard D1557 test method. The test results are presented on Drawing No. B-2, *Moisture-Density Relationship Results,* and in the following table.

Table No. B-3, Laboratory Maximum Density Test Result

Test Pit No.	Depth (feet)	Soil Classification	Maximum Dry Density, pcf	Optimum Moisture, %
TP-2	6.0-9.0	Sandy Gravel (GP)	132.0	8.0

Direct Shear Test

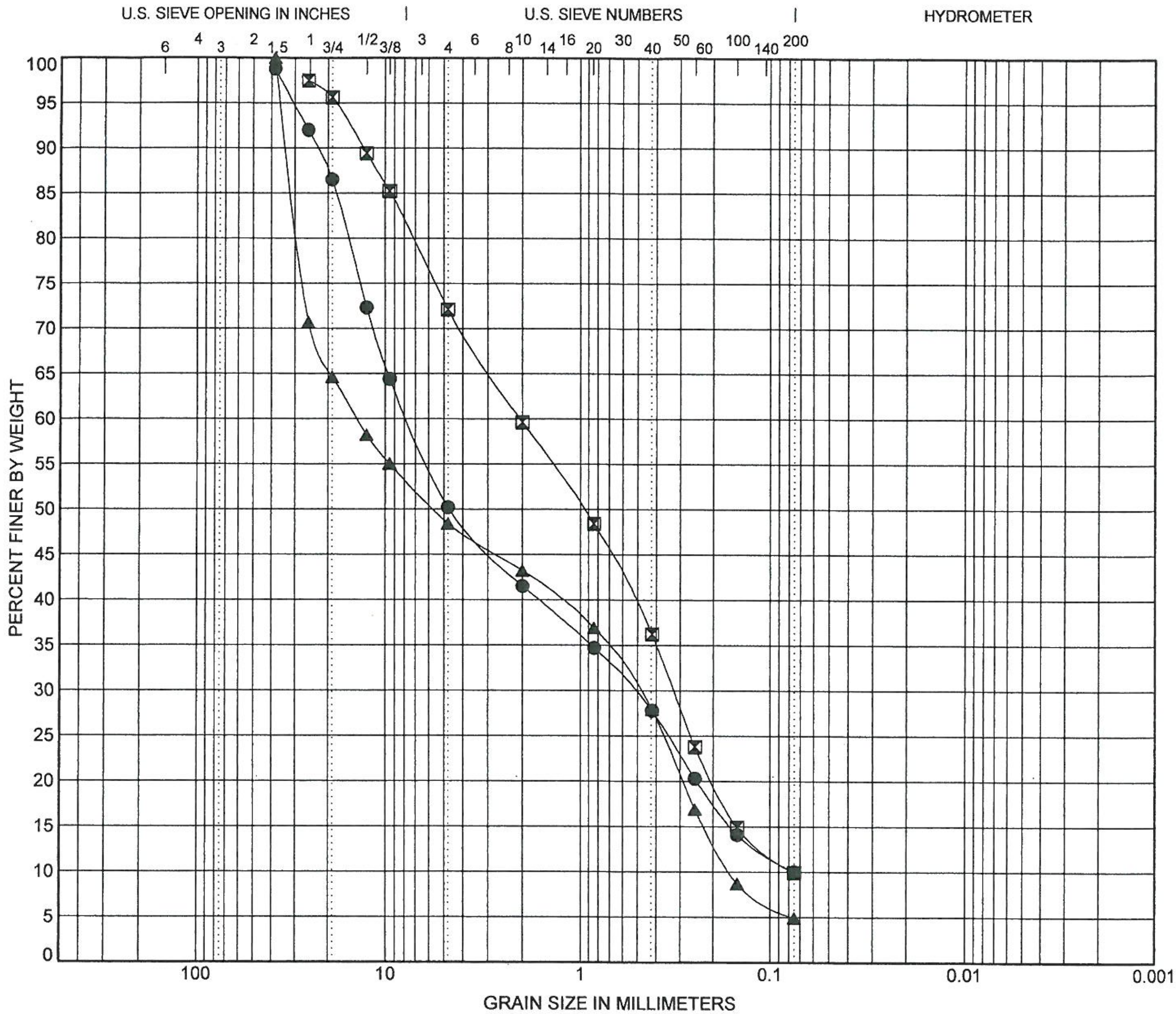
One (1) direct shear test was performed on this project in accordance with ASTM Standard D3080. All samples were tested under soaked moisture conditions. For this test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The 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-3, *Direct Shear Test Results,* and in the following table.

Table No. B-4, Direct Shear Test Result

Sample Location Boring No. (Depth)	Natural/ Remolded	Soil Classification	Average Initial Moisture Content (%)	Average Initial Dry Density (pcf)	Effective Cohesion (psf)	Effective Friction Angle (degree)
BH-4/3.0-4.5	Natural	Gravelly Sand (SP) brown	5.3	119.6	50	33

Sample Storage

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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH - 1**	**1-4**	**SANDY GRAVEL (GP)**				**0.48**	**102.18**
⊠	**BH - 4**	**3.0-4.5**	**GRAVELLY SAND (SP)**				**0.67**	**27.04**
▲	**TP - 2**	**6-9**	**SANDY GRAVEL (GP)**				**0.11**	**86.94**

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH - 1**	**1-4**	**38.1**	**7.664**	**0.524**	**0.075**	**48.6**	**40.2**	**10.0**	
⊠	**BH - 4**	**3.0-4.5**	**25.4**	**2.056**	**0.324**	**0.076**	**25.4**	**62.2**	**9.9**	
▲	**TP - 2**	**6-9**	**38.1**	**14.062**	**0.494**	**0.162**	**51.6**	**43.5**	**4.9**	

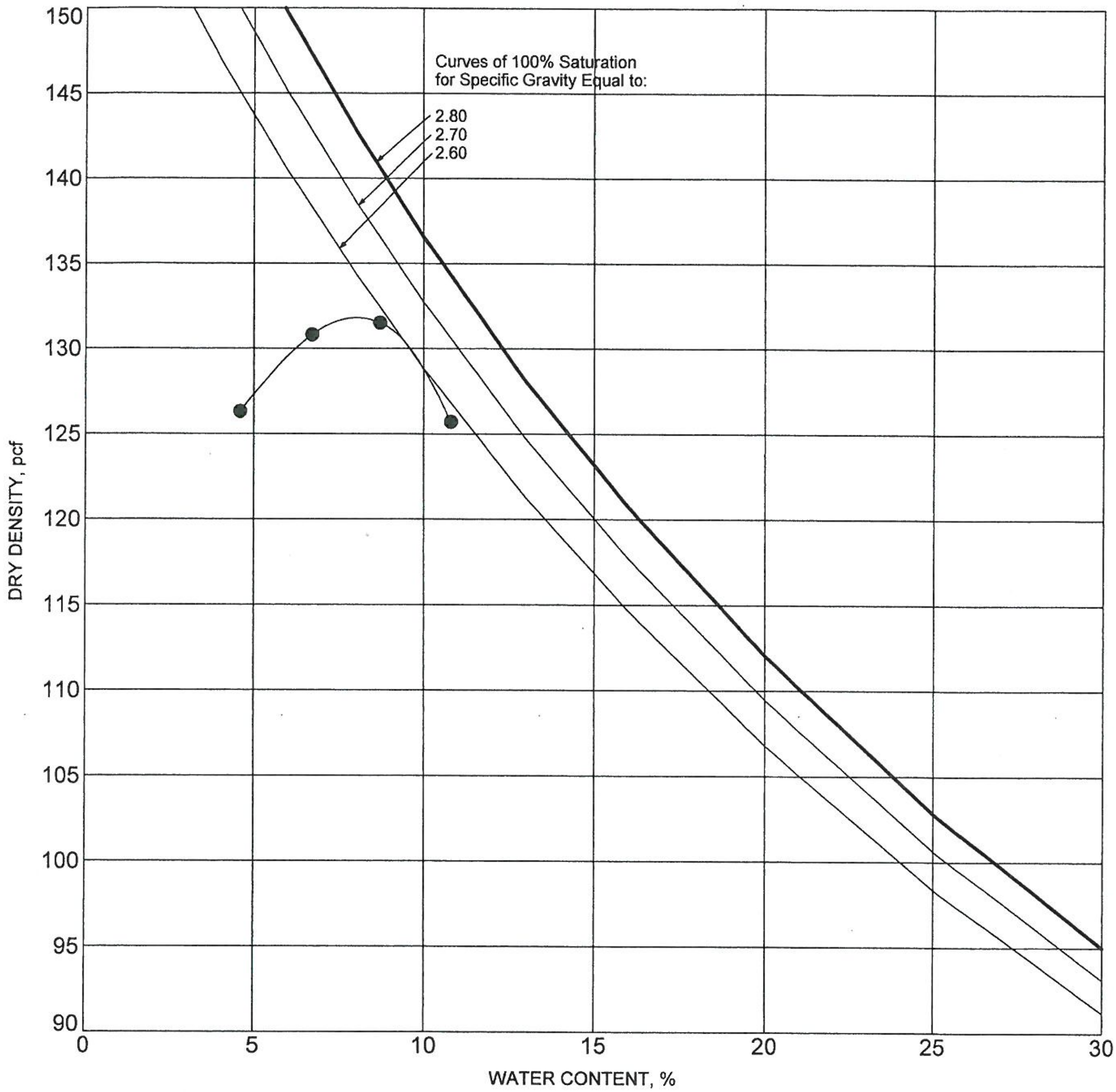
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. **10-81-227-01**

Drawing No. **B - 1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**TP - 2**	**6-9**	**SANDY GRAVEL (GP), brown**	**D1557 - C**	**8.0**	**132.0**

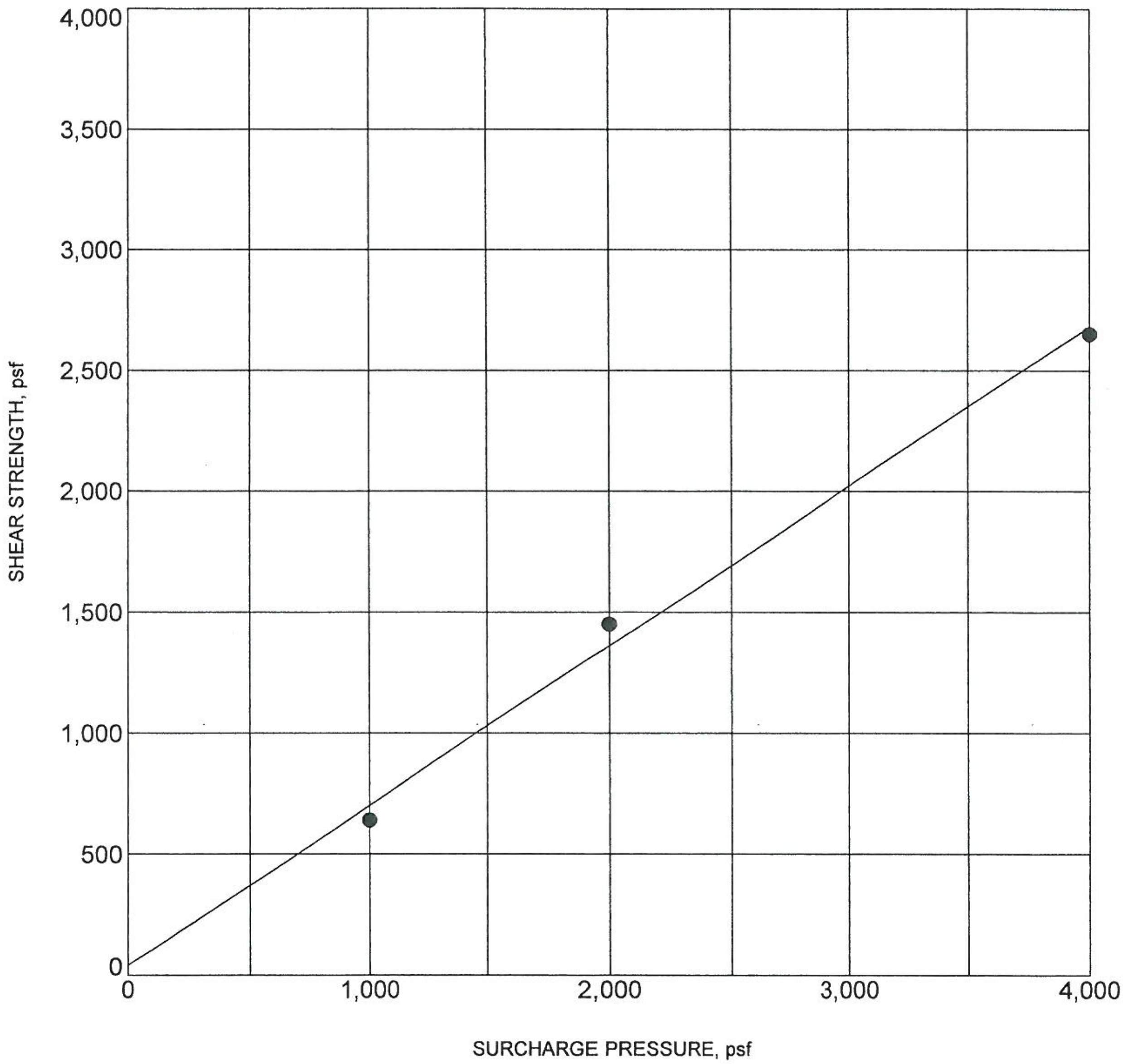
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No.
10-81-227-01

Drawing No.
B - 2



BORING NO. :	BH - 4	DEPTH (ft) :	3.0-4.5
DESCRIPTION :	GRAVELLY SAND (SP)		
COHESION (psf) :	50	FRICTION ANGLE (degrees):	33
MOISTURE CONTENT (%) :	5.3	DRY DENSITY (pcf) :	119.6

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

14th Street Water Quality Regional Facility
City of Upland, California
For: AECOM Water

Project No. 10-81-227-01

Drawing No. B - 3

APPENDIX C

PERCOLATION TEST RESULTS

APPENDIX C

PERCOLATION TEST RESULTS

Two (2) percolation tests were conducted within the test pits located in the proposed desilting/water quality basin on July 13, 2010. The tests were conducted at a depth of approximately nine (9) feet below ground surface. The continuous (falling head) test procedure was utilized for percolation testing, as discussed below.

A test pit approximately 12 feet long x 9 feet deep was excavated using a rubber tire backhoe equipped with a 24-inch wide bucket for soil sampling.

At the bottom of the test pit, a testing surface was prepared in which to conduct the percolation testing.

Using a 6-inch diameter hand auger, a hole was excavated approximately 6-inches in diameter by 6-inches deep. In the bottom of the hole, approximately ½-inch to ¼-inch Pea gravel was placed to stabilize the bottom for testing. A perforated plastic cylinder, approximately 6-inches in diameter by 12-inches in height was placed inside of the hole. The perforations on this cylinder were in the bottom and only 6-inches up from the bottom along the sides of the container. Pea gravel was placed around the sides of the container to a depth of 6 inches, to help stabilized the sidewalls and prevent caving during testing.

Using a blue plastic five (5) gallon water jug, the hole was presoaked by filling the five (5) gallon jug with water and inverting it onto the top of the perforated plastic cylinder. Once the water in the jug was completely empty, testing was allowed to begin that day.

The perforated plastic cylinder was filled with water to approximately 6-inches in height. When the water level inside of the cylinder had dropped by at least 1-inch, the time of drop was recorded. This process was repeated eight (8) times for each test.

All equipment was removed from the bottom of the test pit upon completion of testing, and the test pit was backfilled and wheel rolled using a backhoe.

Test results are presented in the following table. Infiltration rate is calculated as volume of water infiltrated per unit surface area per unit time. Since the cylinder used had perforations on the sides, volume of water infiltrated was divided by the total surface area including sides and bottom of the cylinder to determine the infiltration rate. A safety factor of 2.0 is used to calculate the infiltration rate.

Table No. C-1, Percolation Test Results

Percolation Test / Test Pit Number	Depth Tested (feet)	Average Time for 1 inch drop (min)	Infiltration Rate, q**=Q/A** (Inch/hour)
T-1 / TP-1	9.0	0.76	8
T-2 / TP-2	9.0	0.70	9

Percolation Test Data Sheet

Test No.	T1	Test Pit	TP-1
Job Name:	14th Street Water Quality Regional Facility	Depth of Pit Bottom, feet bgs	9.0
Job No.:	10-81-227-01	Depth of Test Hole, inch	6.0
Location:	City of Upland, CA	Diameter of Test Hole, inch	0.50
Test Date:	7/13/2010	Technician	CG

Test Number	Time of Testing	Time of Testing	Initial Height	Final Height	Drop in Height	time for 1 inch drop, min/inch
	T_I	T_I	d_I	d_f	$F = d_f - d_I = {}_\wedge d$	
	Seconds	Minutes	(inch)	(inch)	(inch)	
1	39	0.65	6.00	5.00	1.00	0.65
2	45	0.75	6.00	5.00	1.00	0.75
3	46	0.77	6.00	5.00	1.00	0.77
4	46	0.77	6.00	5.00	1.00	0.77
5	47	0.78	6.00	5.00	1.00	0.78
6	47	0.78	6.00	5.00	1.00	0.78
7	47	0.78	6.00	5.00	1.00	0.78
8	47	0.78	6.00	5.00	1.00	0.78

Average time for 1 inch drop, min	0.76

Drawing No. C-1

Percolation Test Data Sheet

Test No.	T2	Test Pit	TP-2
Job Name:	14th Street Water Quality Regional Facility	Depth of Pit Bottom, feet bgs	9.0
Job No.:	10-81-227-01	Depth of Test Hole, inch	6.0
Location:	City of Upland, CA	Diameter of Test Hole, inch	0.50
Test Date:	7/13/2010	Technician	CG

Test Number	Time of Testing	Time of Testing	Initial Height	Final Height	Drop in Height	time for 1 inch drop, min/inch
	T_I	T_I	d_I	d_f	$F = d_{f-} d_I = {}_\wedge d$	
	Seconds	Minutes	(inch)	(inch)	(inch)	
1	34	0.57	6.00	5.00	1.00	0.57
2	37	0.62	6.00	5.00	1.00	0.62
3	40	0.67	6.00	5.00	1.00	0.67
4	44	0.73	6.00	5.00	1.00	0.73
5	43	0.72	6.00	5.00	1.00	0.72
6	45	0.75	6.00	5.00	1.00	0.75
7	45	0.75	6.00	5.00	1.00	0.75
8	46	0.77	6.00	5.00	1.00	0.77

Average time for 1 inch drop, min	0.70

Drawing No. C-2