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

GEOTECHNICAL INVESTIGATION REPORT

EASTSIDE WATER TREATMENT FACILITY EXPANSION AND OFFSITE PIPELINE

CITIES OF CHINO AND ONTARIO, SAN BERNARDINO COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 18-81-287-01

Prepared For:

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August 7, 2019

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

August 7, 2019

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Senior Principal Engineer
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Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
Converse Project No. 18-81-287-01

Dear Mr. Rahimian-Pour:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of the Eastside Water Treatment Facility (EWTF) Expansion and Offsite Pipeline Project, located within the Cities of Chino and Ontario, San Bernardino County, California. This report was prepared in accordance with our revised proposal dated December 28, 2018 and your Subcontract Agreement for Professional Services dated January 24, 2019.

Based upon our field investigation, laboratory data, and analyses, the proposed project is considered suitable from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and construction of the project.

We appreciate the opportunity to be of service to Hazen and Sawyer. Should you have any questions, please do not hesitate to contact us at 909-796-0544.

CONVERSE CONSULTANTS

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

Dist.: 4/Addressee
HSQ/JB/ZA/kvg

PROFESSIONAL CERTIFICATION

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

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

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Principal Engineer

EXECUTIVE SUMMARY

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

- The Eastside Water Treatment Facility (EWTF) Expansion and Offsite Pipeline Project is located within the Cities of Chino and Ontario, San Bernardino County, California. The 12.25-acre EWTF is located south of Schaefer Avenue and east of Campus Avenue in the City of Ontario. The property is bounded to the north by Schaefer Avenue, to the south by a vacant land, to the east by a cattle ranch and west by a business. Presently structures within the site includes existing treatment building, disinfection and storage building, water reservoirs, chemical storage tanks and a booster pump station building.

- The approximately 18,650 linear feet of pipeline will originate from the EWTF in the City of Ontario and traverse along Euclid Avenue, Merrill Avenue, Bon View Avenue and Schaefer Avenue into the City of Ontario. Within the project limits, Merrill Avenue, Bon View Avenue and Schaefer Avenue have one lane in each direction with no shoulder. Light to medium vehicular traffic was observed throughout the day. Euclid Avenue has two lanes in each direction with a median, medium to high volume vehicular traffic was observed throughout the day.

- The City of Chino (project owner) desires to expand the existing EWTF (LGAC - to treat 1,2,3 TCP and in-exchange - to treat Nitrate and Perchlorate) from 3,500 gpm to 7,000 gpm. The facility expansion will include three structures (IX treatment building, waste tank and LGAC system). We understand structures will be founded on shallow foundation with slabs-on-grade or mat foundations. The brine disposal line consisting of dual 4-inch diameter HDPE pipeline will route waste to the Inland Empire Brine Line at approximately the intersection of Euclid and Kimball Avenue. The depth to pipe invert will be within 10 feet of the below existing ground surface. The pipeline will be installed using open cut-and-cover technique.

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

- One exploratory boring (BH-01) was drilled on May 29, 2019 to investigate subsurface conditions at the EWTF site. Due to close proximity of existing underground utilities, a 4-inch diameter hand auger was used to drill the upper 10 feet below existing ground surface (bgs). The boring was drilled to the planned maximum depth of 51.5 feet bgs.

- Twelve exploratory borings (BH-02 through BH-13) were drilled between May 28 and 30, 2019 to investigate subsurface conditions along the pipeline alignment. Due to close proximity of existing underground utilities, a 4-inch diameter hand auger was used to drill the upper 5 feet for all borings. The borings were drilled to the planned maximum depth of 16.5 feet bgs except BH-11 which was drilled at a depth of 10.0 feet bgs using hand auger.

- The asphalt concrete thickness encountered at the boring locations varied from 6 to 12 inches and aggregate base thickness varied from 4 to 6 inches.

- Based on the exploratory borings and laboratory test results, the subsurface soil at the treatment facility site consists primarily of a mixture of sand, silt, clay and gravel. Gravel up to 2 inches in largest dimension was encountered in the boring. Moisture content varied from 7 to 35 percent.

- Based on the exploratory borings and laboratory test results, the subsurface soil along the pipeline alignment consists primarily of a mixture of sand, silt, clay and gravel. Gravel up to 2 inches in largest dimension was encountered in most of the borings. Though not encountered in the borings, cobbles or boulders may be present within the pipeline alignment. Moisture content of the upper 10 feet soils varied from 5 to 43 percent. High moisture content (45 percent) was observed at a depth of 5 feet bgs in boring BH-05 (Merrill Avenue). We are not certain; however, this high moisture may be due to the presence of irrigation field on both side of Merrill Avenue.

- Groundwater was not encountered during the investigation to the maximum explored depth of 51.5 feet bgs. Based on the difference between the elevation of the wellhead (Chino Airport well) and the lowest elevation throughout the project limit, historical high ground water is expected to be deeper than 43 feet bgs. Dewatering is not expected to be required during the construction of the project. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

- The project site is not located within a currently mapped State of California or San Bernardino County Earthquake Fault Zone for surface fault rupture.

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

- The expansion index (EI) of the sample tested from the treatment facility site was 0, corresponding to very low expansion potential. The expansion index (EI) of the sample tested from the alignment (Merrill Avenue) was 54, corresponding to medium expansion potential. The measured sand equivalent along the pipeline alignment were between 4 and 12. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material.

- The sulfate contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations. No concrete type restrictions are specified for exposure category S0. A minimum compressive strength of 2,500 psi is recommended. The chloride contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category C1 (concrete is exposed to moisture, but not to external sources of chlorides). For exposure category C1, ACI provides concrete compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

- The measured value of the minimum electrical resistivity of the sample when saturated was 670 ohm-cm at the treatment facility site. <u>This indicates that the soils tested are severely corrosive to ferrous metals in contact with the soil.</u> The measured value of the minimum electrical resistivities of the samples when saturated were between 1,419 and 3,133 ohm-cm along pipeline alignment. This indicates that the soils tested are moderately corrosive to corrosive to ferrous metals in contact with the soil.

- According to the Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive if the pH is 5.5 or less, or chloride content is 500 parts per million (ppm) or greater, or sulfate content is 1,500 ppm or greater, or resistivity less than 2000 ohm-cm. <u>Based on the tested results, soils are considered corrosive. No mitigation is needed for HDPE pipe. However, converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site and site soils.</u>

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

- The surface and subsurface soil materials for the proposed project site are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. <u>Difficult excavation will occur if concentration of gravel is encountered.</u>

- Excavated onsite earth materials cleared of deleterious matter can be moisture conditioned and re-used as compacted fill.

- The footings and slabs-on-grade and pavement should be overexcavated based on Section 9.2, Table No. 6, *Overexcavation Depths*. The overexcavation below the footings, slabs and pavement should be uniform. The overexcavation should extend to at least 2 feet beyond the footprint of the footings and slabs, and at least 1 foot beyond the pavement.

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

- Footings (IX treatment building, waste tank and LGAC system) should be at least 18 inches in width and embedded to at least 18 inches below the lowest adjacent grade. The footing dimensions and reinforcement should be based on structural design. Continuous and isolated footings can be designed based on an allowable net bearing capacity of 2,000 psf.

- Matt foundation recommendation is presented in the Section 10.2 *Mat Foundation Design Parameters*.

- The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 30 feet

- We estimate that the treatment facility site has the potential for up to 0.3 inches of dry seismic settlement with negligible liquefaction induced settlement under groundwater condition deeper than 50 feet bgs during a large earthquake. The dynamic differential settlement of the site may be up to half of the total settlement over 30 horizontal feet.

- Lateral earth pressures and pipe design parameters are presented in the text of this report.

- Pavement design recommendations are presented in the text of this report.

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

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

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APPENDICES

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for the Eastside Water Treatment Facility (EWTF) expansion project, located within the Cities of Chino and Ontario, San Bernardino County, California. The project site (treatment facility and pipeline alignment) is shown in Figure No. 1, *Approximate Project Location Map*.

The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils, and to provide design and construction recommendations for the proposed EWTF Expansion and Offsite Pipeline Project.

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

2.0 PROJECT DESCRIPTION

The 12.25-acre EWTF site is located south of Schaefer Avenue and east of Campus Avenue in the City of Ontario. The City of Chino (project owner) desires to expand the existing EWTF (LGAC - to treat 1,2,3 TCP and in-exchange - to treat Nitrate and Perchlorate) from 3,500 gpm to 7,000 gpm. The facility expansion will include three structures (IX treatment building, waste tank and LGAC system). We understand these structures will be founded on shallow foundation with slabs-on-grade or mat foundations.

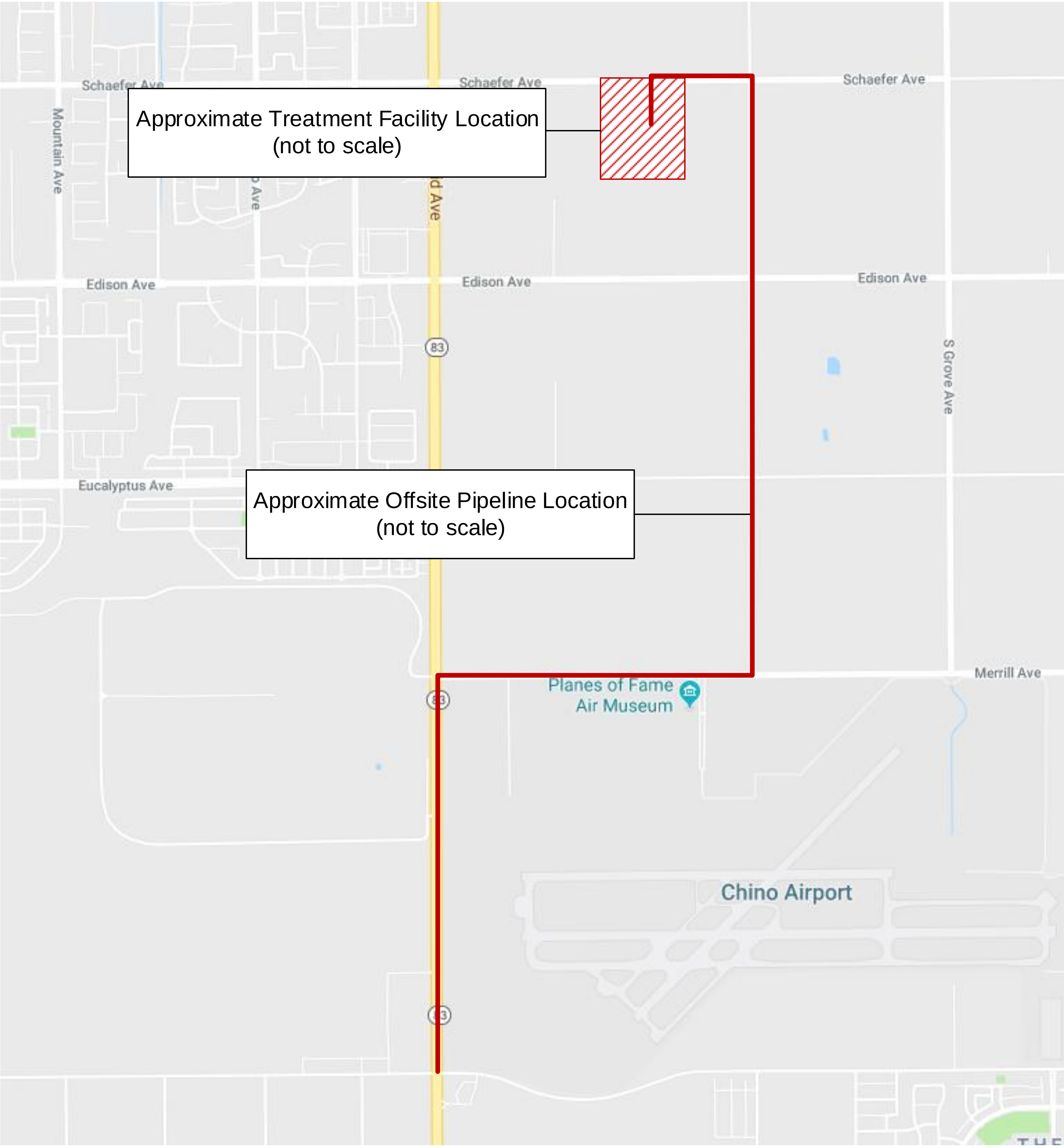
The project will also include approximately 18,650 linear feet of brine disposal pipeline consisting of dual 4-inch diameter HDPE pipe routing waste to the Inland Empire Brine Line at approximately the intersection of Euclid and Kimball Avenue. The depth to pipe invert will be within 10 feet of the below existing ground surface. The pipeline will be installed using open cut-and-cover technique.

3.0 SITE DESCRIPTIONS

Site descriptions for treatment facility site and pipeline are presented below.

Treatment Facility Site

The treatment facility site is located southeast of the intersection of Schaefer Avenue and Campus Avenue, in the City of Ontario, CA. The site is bounded to the north by Schaefer Avenue, to the south by vacant land, to the east by cattle ranch and to the west by a business. The site includes but not limited to existing treatment facility building, disinfection and storage building, water reservoir, chemical tanks, reservoir no. 1, and



Approximate Project Location Map

N

Project: Eastside Water Treatment Facility Expansion and Offsite Pipeline
Location: Cities of Chino and Ontario, San Bernardino County, California

For: Hazen and Sawyer

Project No
18-81-287-01

Converse Consultants

FIGURE NO.
1

booster pump building. Photograph No. 1 depicts the present treatment facility site conditions.

Photograph No. 1, Present Treatment Facility Site, facing southwest.

Pipeline

The pipeline is located along Euclid Avenue, Merrill Avenue, Bon View Avenue and Schaefer Avenue. Within the project limit, Merrill Avenue, Bon View Avenue and Schaefer Avenue have one lane in each direction with no shoulder. Light to medium traffic was observed throughout the day. Euclid Avenue has two lanes in each direction with a median. Medium to high traffic was observed throughout the day. Photographs No. 2 and 3 depict the present alignment conditions.

Photograph No. 2, Present alignment conditions along Merrill Avenue, facing east.

Photograph No. 3, Present alignment conditions along Bon View Avenue, facing north.

4.0 SCOPE OF WORK

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

4.1 Document Review

We reviewed geologic maps, aerial photographs, groundwater data, and other information pertaining to the project site to assist in the evaluation of geologic hazards that may be present. We used pertinent information (the documents cited in Section 14, *Reference*s) to understand the subsurface conditions and plan the investigation for this project.

4.2 Project Set-up

The project set-up consisted of the following tasks.

- Prepared and submitted a geotechnical exploration plan for your review.
- Conducted a field reconnaissance and marked the boring locations such that the drill rig access to all locations was available.
- Obtained permit from the California Department of Transportation (Caltrans) and Cities of Ontario and Chino.
- Prepared traffic control plans based on California MUTCD manual.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring location of any conflict with existing underground utilities.
- Engaged a California-licensed driller and professional traffic control to drill exploratory borings.

4.3 Subsurface Exploration

One exploratory boring (BH-01) was drilled on May 29, 2019 to investigate subsurface conditions at the EWTF site. Due to close proximity of existing underground utilities, a 4-inch diameter hand auger was used to drill the upper 10 feet below existing ground surface (bgs). The boring was drilled to the planned maximum depth of 51.5 feet bgs.

Twelve exploratory borings (BH-02 through BH-13) were drilled between May 28 and 30, 2019 to investigate subsurface conditions along the pipeline alignment. Due to close proximity of existing underground utilities, a 4-inch diameter hand auger was used to drill the upper 5 feet for all borings. The borings were drilled to the planned maximum depth of 16.5 feet bgs except BH-11 which was drilled at a depth of 10.0 feet bgs using hand auger.

Details of borings are presented in the following table.

Table No. 1, Details of Borings

Boring No.	Location	Approx. Station	Depth (ft)
BH-01	Treatment Facility	N/A	51.5
BH-02	Euclid Ave.	21+20	16.5
BH-03	Euclid Ave.	41+80	16.5
BH-04	Euclid Ave.	56+60	16.5
BH-05	Merrill Ave.	71+20	16.5
BH-06	Merrill Ave.	88+50	16.5
BH-07	Merrill Ave.	101+40	16.5
BH-08	Bon View Ave.	122+20	16.5
BH-09	Bon View Ave.	151+50	16.5
BH-10	Bon View Ave.	176+30	16.5
BH-11	Schaefer Ave.	188+40	10.0
BH-12	Schaefer Ave.	195+50	16.5
BH-13	Intersection of Edison Ave. and Campus Ave.	N/A	16.5

(N/A = not applicable)

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

4.4 Laboratory Testing

Representative samples of the project 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 the following.

- *In*-situ moisture contents and dry densities (ASTM D2216 and ASTM D7263)
- Expansion index (ASTM D4829)
- Sand equivalent (ASTM D2419)
- R-value (California Test 301)
- Soil corrosivity (California Tests 643, 422, and 417)
- Grain size distribution (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

EXPLANATION

BH-13 — Number and Approximate Location of Exploratory Boring

1600'

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Approximate Boring Locations Map

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Figure No. 2

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

4.5 Analysis and Report Preparation

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

5.0 SUBSURFACE CONDITIONS

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

5.1 Existing Pavement Sections

The encountered pavement thicknesses at the boring location are included in the following table.

Table No. 2, Existing Pavement Sections

Boring No.	Location	Asphalt Concrete Thickness (in.)	Aggregate Base Thickness (in.)
*BH-01	Treatment Facility	0.0	0.0
BH-02	Euclid Ave.	6.0	5.0
BH-03	Euclid Ave.	6.0	6.0
BH-04	Euclid Ave.	8.0	4.0
*BH-05	Merrill Ave.	0.0	0.0
BH-06	Merrill Ave.	12.0	0.0
*BH-07	Merrill Ave.	0.0	0.0
BH-08	Bon View Ave.	5.0	0.0
BH-09	Bon View Ave.	6.0	0.0
BH-10	Bon View Ave.	5.0	0.0
*BH-11	Bon View Ave.	0.0	0.0
*BH-12	Schaefer Ave.	0.0	0.0
*BH-13	Edison Ave.	0.0	0.0

(* drilled on dirt area; For approximate boring locations, see Figure No. 2a)

5.2 Subsurface Profile

Subsurface conditions at the treatment facility site and along the pipeline alignment are presented below.

<u>Treatment Facility Site</u>
Based on the exploratory borings and laboratory test results, the subsurface soil at the treatment facility site consists primarily of a mixture of sand, silt, clay and gravel. Gravel up to 2 inches in largest dimension was encountered in the boring. Moisture content varied from 7 to 35 percent.

<u>Pipeline</u>
Based on the exploratory borings and laboratory test results, the subsurface soil along the pipeline alignment consists primarily of a mixture of sand, silt, clay and gravel. Gravel up to 2 inches in largest dimension was encountered in most of the borings. Though not encountered in the borings, cobbles or boulders may be present within the pipeline alignment. Moisture content of the upper 10 feet soils varied from 5 to 43 percent. High moisture content (45 percent) was observed at a depth of 5 feet bgs in boring BH-05 (Merrill Avenue). We are not certain; however, this high moisture may be due to the presence of irrigation field on both side of Merrill Avenue.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-14, Logs of Borings, in Appendix A, Field Exploration.

5.3 Groundwater

Groundwater was not encountered during the investigation to the maximum explored depth of 51.5 feet bgs. The GeoTracker database (SWRCB, 2019) was reviewed for groundwater data from sites within close proximity of both the water treatment facility and the pipeline locations. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data.

- CHINO AIRPORT (SL208634049) is located approximately 5,200 feet southwest of the project site. Groundwater was reported at this site at depths ranging from approximately 1 to 143 feet bgs between 2003 and 2018.

The National Water Information System (USGS, 2019) was also reviewed and found to have no sites in proximity to the project site.

The Cooperative Well Measuring Program (WMSS, 2018) was reviewed for available groundwater measurements within the vicinity. The project site is located within California, San Bernardino Meridian T2S, R7W, sec36. Two wells were identified within 1 mile of the overcrossing that contained groundwater measurements. The state well numbers, most recent groundwater elevation and depth below groundwater surface, and position relative to the overcrossing are shown in the following table.

Table No. 3, Groundwater Measurements

State Well No.	Location	GW Depth (ft bgs)	GW Elevation (amsl)	Date Measured
02S/07W-17G	Approx. 2,000 feet SE of overcrossing	126	567	9/2/1999
02S/07W-17R	Approx. 4,900 feet SE of overcrossing	121	562	12/13/1999

Current groundwater is expected to be deeper than 51.5 feet bgs. Based on the difference between the elevation of the wellhead (Chino Airport well) and the lowest elevation of the project site, historical high ground water is expected to be deeper than 43 feet bgs. Dewatering is not expected to be required during the construction of the project. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation or irrigation events.

5.4 Excavatability

The surface and subsurface soil materials for the proposed project site are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. Difficult excavation will occur if concentration of gravel is encountered.

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

5.5 Subsurface Variations

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

6.0 ENGINEERING GEOLOGY

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

6.1 Regional Geology

The project site is situated within the Chino Basin near the northern boundary of the Peninsular Ranges Geomorphic Province adjacent to the Transverse Ranges province.

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 Andreas and San Jacinto fault zones 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.

6.2 Local Geology

Review of geologic mapping indicates that the project site is underlain by young (Holocene-aged) alluvial fan deposits. These alluvial deposits primarily consist of gravel, sand, and silt of valleys and floodplains (Dibblee and Ehrenspeck, 2001). Where encountered in our borings, the alluvium primarily consisted of clay, silt, sand, and gravel.

7.0 FAULTING AND SEISMICITY

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

7.1 Faulting

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 project site is not located within a currently mapped State of California or San Bernardino County Earthquake Fault Zone for surface fault rupture (CGS, 2007; San Bernardino County, 2010b). The closest known major faults to the project site with mappable surface projections is the Newport-Inglewood Fault with a closest distance of 9.4 mile (15.13 km) to the southwest.

Table No. 4, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 50 kilometers of the site. The data presented below was calculated using the National Seismic Hazard Maps Database (USGS, 2008) and other published geologic data.

Table No. 4, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Chino, alt 2	7.35	strike slip	29	1	6.80
Chino, alt 1	7.58	strike slip	24	1	6.70
San Jose	12.24	strike slip	20	0.5	6.70
Cucamonga	15.56	thrust	28	5	6.70
Elsinore	16.07	strike slip	241	n/a	7.85
Sierra Madre	16.11	reverse	76	2	7.30
Puente Hills (Coyote Hills)	24.18	thrust	17	0.7	6.90
San Jacinto	29.09	strike slip	241	n/a	7.88
Clamshell-Sawpit	32.35	reverse	16	0.5	6.70
S. San Andreas	34.11	strike slip	548	n/a	8.18
Puente Hills (Santa Fe Springs)	35.92	thrust	11	0.7	6.70
Raymond	36.84	strike slip	22	1.5	6.80
Cleghorn	37.66	strike slip	25	3	6.80
San Joaquin Hills	38.99	thrust	27	0.5	7.10
Elysian Park (Upper)	43.11	reverse	20	1.3	6.00
Puente Hills (LA)	45.14	thrust	22	0.7	7.00
North Frontal (West)	48.67	reverse	50	1	7.20
Newport Inglewood	48.92	strike slip	208	1.3	7.50
Verdugo	49.51	reverse	29	0.5	6.90

(Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/)

7.2 Seismic Design Parameters

Seismic parameters based on the California Building Code (CBSC, 2016) provided in the following table were determined using the Seismic Design Maps application (OSHPD, 2019). The coordinates selected correspond to the approximate center of the treatment facility site and pipeline alignment.

Table No. 5, CBC 2016 Seismic Parameters

Seismic Parameters	Treatment Facility	Pipeline
Site Coordinates	34.0045N, 117.6402W	33.9931N, 117.6432W
Site Class	D	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.5g	1.5g
Mapped 1-second Spectral Response Acceleration, S_1	0.6g	0.6g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5	1.5
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	1.5g	1.5g
MCE 1-second period Spectral Response Acceleration, S_{M1}	0.9g	0.9g
Design Spectral Response Acceleration for short period S_{ds}	1g	1g
Design Spectral Response Acceleration for 1-second period, S_{d1}	0.6g	0.6g
Maximum Peak Ground Acceleration, PGA_M	0.503g	0.535g

7.3 Secondary Effects of Seismic Activity

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

Surface Fault Rupture: The project site is not located within a currently designated State of California or San Bernardino County Earthquake Fault Zone (CGS, 2007; San Bernardino County, 2010b). There are no known active faults projecting toward or extending across the project site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

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

Soil liquefaction generally occurs in submerged granular soils and non-plastic silts during or after strong ground shaking. There are several general requirements for liquefaction to occur and they are as follows.

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

Based on review of hazard maps, there is no availability of data to determine liquefaction susceptibility. Based on our analysis, the treatment facility site has negligible liquefaction potential.

Seismic Settlement: Seismically-induced settlement occurs in unsaturated, unconsolidated, granular sediments during ground shaking associated with earthquakes. Based on our analysis, the treatment facility site has the potential for up to 0.3 inches of dry seismic settlement under current and historical groundwater condition deeper than 50 feet bgs and 43 feet bgs, respectively.

Landslides: Seismically induced landslides and slope failures are common occurrences during or soon after large earthquakes. Due to the flat nature of the site, the potential for seismically induced landslides affecting the proposed site is considered to be low.

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

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

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. There is no risk for seiching along the alignment of the proposed pipeline; however, there are small bodies of water located south of the treatment facility. Seiching is considered to be a risk during construction at the treatment facility.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is not located within a designated dam inundation zone (San Bernardino County, 2010a). The risk for earthquake-induced flooding to affect the project site is considered low.

8.0 LABORATORY TEST RESULTS

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

8.1 Physical Testing

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

Treatment Facility Site

- In-situ Moisture and Dry Density – *In-situ* dry density and moisture content of the site soils were determined in accordance to ASTM Standard D2216 and D7263. Dry densities below 10 feet soils (no drive samples due to hand augered to 10 feet bgs) to the maximum explored depth ranged from 85 to 120 pounds per cubic foot (pcf) with moisture contents of 7 to 35 percent. Results are presented in the log of boring (BH-01) in Appendix A, *Field Exploration.*
- Expansion Index – One representative sample from the upper 5 feet soils was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result showed an EI of 0, indicating very low expansion potential.
- Grain Size Analysis – Two representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1a, *Grain Size Distribution Results.*
- Direct Shear – One direct shear test was performed on relatively undisturbed samples under soaked moisture condition in accordance with ASTM Standard D3080. The result is presented in Drawing No. B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

Pipeline

- In-situ Moisture and Dry Density – *In-situ* dry density and moisture content of the site soils were determined in accordance to ASTM Standard D2216 and D7263. Dry densities of the upper 10 feet soils ranged from 74 to 114 pcf with moisture contents of 5 to 43 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Expansion Index – One representative sample from the upper 10 feet soils was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result showed an EI of 54, indicating medium expansion potential.

- Sand Equivalent – Five representative bulk soil samples were tested to evaluate sand equivalent (SE) in accordance with the ASTM Standard D2419 test method. The measured sand equivalents ranged from 4 to 12.
- R-Value – Three representative bulk samples were tested to evaluate the R-value in accordance with California Test 301. The test results indicated R-value of 14, 14 and 31.
- Grain Size Analysis – Four representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawings No. B-1a and B-1b, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship test was conducted on three representative samples in accordance with ASTM D1557. The results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program*. The laboratory maximum dry densities were124.5, 125.5 and 127.0 pcf and optimum moisture contents of 10.5, 10 and 10 percent, respectively.
- Direct Shear – Four direct shear tests were performed on relatively undisturbed samples under soaked moisture condition in accordance with ASTM Standard D3080. The results are presented in Drawings No. B-3 through B-7, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

8.2 Chemical Testing - Corrosivity Evaluation

Four representative soil samples (one from treatment facility site and 3 from the pipeline alignment) were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purposes of the tests was to determine the corrosion potential of site soils when placed in contact with common pipe materials. The test was performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Tests 643, 422, and 417. The test results are presented in Appendix B, *Laboratory Testing Program* and summarized below.

Treatment Facility

- The pH measurement of the tested sample was 9.0.
- The sulfate content of the tested sample was 0.0453 percent by weight (453 ppm).
- The chloride concentration of the tested sample was 262 ppm.
- The minimum electrical resistivity when saturated was 670 ohm-cm.

Pipeline

- The pH measurements of the tested samples were 8.6, 8.7 and 8.8.
- The sulfate content of the tested sample were 0.0036, 0.005 and 0.0077 percent by weight (36, 50 and 77 ppm).
- The chloride concentrations of the tested samples were 38, 40 and 44 ppm.
- The minimum electrical resistivities when saturated were 1,419, 2,844 and 3,133 ohm-cm.

9.0 EARTHWORK RECOMMENDATIONS

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

9.1 General

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

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

All debris, surface vegetation, deleterious material, surficial soils containing roots and perishable materials should be stripped and removed from the site.

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

9.2 Overexcavation

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

Table No. 6, Overexcavation Depths

Structure/Pavement	Minimum Excavation Depth
Footings	24 inches below footings bottom or 5 feet below existing ground surface, whichever is deeper
Slabs-on-grade	18 inches below slab
Pavement	12 inches below finish grade

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

If isolated pockets of very soft, loose, eroded, or pumping soil are encountered, the unstable soil should be excavated as needed to expose undisturbed, firm, and unyielding soils.

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

9.3 Engineered Fill

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. Excavated soils should be processed, including removal of roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-site soils used as fill should meet the following criteria.

- No particles larger than 3 inches in largest dimension.
- Rocks larger than one inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 20 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 40 percent fines (passing #200 sieve).

Based on field investigation and laboratory testing results, on-site soils may not be suitable as fill materials.

Imported materials, if required, should meet the above criteria prior to being used as compacted fill. Any imported fills should be tested and approved by geotechnical representative prior to delivery to the site.

9.4 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 12 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

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

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

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

9.5 Site Drainage

Adequate positive drainage should be provided away from the treatment facility site and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. Surface drainage should be directed to suitable non-erosive devices.

9.6 Utility Trench Backfill

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

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

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

9.6.1 Pipeline Subgrade Preparation

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

Any loose, soft and/or unsuitable materials encountered at the pipe subgrade should be removed and replaced with an adequate bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

9.6.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Pipe bedding should follow the Standards of the Cities of Chino and Ontario, and Caltrans (Euclid Ave.) (attached in Appendix D). Additional information for pipe bedding are provided below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate, or crushed rock may be used as pipe bedding material. The sand equivalents of the tested soils ranged from 4 to 12. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. Based on the laboratory test results, on-site soils may not be suitable for pipe bedding. The pipe designer should determine if the soils are suitable as pipe bedding material.

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

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

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

i. $\frac{D15(F)}{D85(B)} \leq 5$

ii. $\frac{D50(F)}{D50(B)} < 25$

iii. Bedding Materials must have less than 5 percent minus 75 µm (No. 200) sieve to avoid internal movement of fines.

Where,
F = Bedding Material
B = Surrounding Native and/or Fill Soils

*D*15(F) = Particle size through which 15% of bedding material will pass
*D*85(B) = Particle size through which 85% of surrounding soil will pass
*D*50(F) = Particle size through which 50% of bedding material will pass
*D*50(B) = Particle size through which 50% of surrounding soil will pass

If the above criteria do not satisfy, commercially available geofabric used for filtration purposes (such as Mirafi 140N or equivalent) may be wrapped around the bedding material encasing the pipe to separate the bedding material from the surrounding native or fill soils.

9.6.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. On-site trench backfill should follow the Standards of the Cities of Chino and Ontario, and Caltrans (Euclid Ave.) (attached in Appendix D). Besides, additional trench backfill recommendations are presented below.

- 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 contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- The field density of the compacted soil should be measured by the ASTM D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) or equivalent.
- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.
- 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.

10.0 DESIGN RECOMMENDATIONS

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

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

10.1 Shallow Foundation Design Parameters

Three structures (IX treatment building, waste tank and LGAC system) may be supported on continuous spread and/or isolated spread footings. The design of the shallow foundations should be based on the recommended parameters presented in the table below.

Table No. 7, Recommended Foundation Parameters

Parameter	Value
Minimum continuous spread footing width	18 inches
Minimum isolated footing width	18 inches
Minimum continuous or isolated footing depth of embedment below lowest adjacent grade	18 inches
Allowable net bearing capacity	2,000 psf

The actual footing dimensions and reinforcement should be based on structural design. The allowable bearing capacity can be increased by 500 pounds per square foot (psf) with each foot of additional embedment and 150 psf with each foot of additional width up to a maximum of 3,500 psf.

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

10.2 Mat Foundation Design Parameters

The proposed concrete pads (IX treatment building, waste tank and LGAC system) may be designed as mat foundation. The modulus of subgrade reaction (k) for design of flexible mat foundation was estimated from the available soil compressibility data and published charts. For design of flexible mat foundation, the following equation may be used.

$k = k_1[(B+1)/2B]^2$

Where:
k= vertical modulus of subgrade reaction for mat foundation, kips per cubic feet
k_1= 200 kcf, normalized modulus of subgrade reaction for 1-square-foot footing
B= foundation width, feet
$E = 33\ W_c^{1.5}\ f_c^{\ 0.5}$ psi
Where, W_c = weight of concrete (pcf)
f_c = compressive strength of concrete at 28 days (psi)

ν = 0.35, Poisson's Ratio

An allowable net bearing capacity of 2,500 psf may be used for mat foundations founded on compacted native soil. The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. The mat foundation dimensions and reinforcement should be based on structural design. For design purposes, the self-weight of the mat foundation can be negligible.

10.3 Lateral Earth Pressures and Resistance to Lateral Loads

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

10.3.1 Active Earth Pressures

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

Table No. 8, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure (psf/ft of depth)
Active earth conditions (wall is free to deflect at least 0.001 radian)	43
At-rest (wall is restrained)	65

These pressures assume a level ground surface behind the wall or foundation with no surcharge and no hydrostatic pressure. If water pressure is allowed to build up behind the walls, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the walls. Specific surcharge load is not known at this time. If a uniform surcharge load is applied on a structure or wall, the lateral earth pressure due to surcharge load should be taken conservatively as 50 psf or uniform surcharge applied to the wall backfill surface within the limits of the active failure wedge multiplying by the lateral earth pressure coefficients.

10.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 240 psf per foot of depth may be used for the sides of the footing poured 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 2,000 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 1 foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

10.4 Slabs-on-Grade

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

Slabs-on-grade should have a minimum thickness of 12, 14 and 18 inches for IX treatment building, waste tank and LGAC system, respectively. Structural design elements of slabs-on-grade, including but not limited to thickness, reinforcement, joint spacing of more heavily-loaded slabs will be dependent upon the anticipated loading conditions and the modulus of subgrade reaction (200 kcf) of the supporting materials and should be designed by a structural engineer.

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

Subgrade for slabs-on-grade should be firm and uniform. All loose or disturbed soils including under-slab utility trench backfill should be recompacted. We recommend to put down at least 6 inches of a free draining crushed aggregate base coarse directly below the slabs.

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

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

10.5 Settlement

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 30 feet.

Our analysis of the potential dynamic settlement is presented in Appendix C, *Liquefaction and Settlement Analysis*. We estimate that the treatment facility site has the potential for up to 0.3 inches of dry seismic settlement with negligible liquefaction induced settlement under groundwater condition deeper than 50 feet bgs during a large earthquake. The dynamic differential settlement of the site may be up to half of the total settlement over 30 horizontal feet.

Generally, the static and dynamic settlement does not occur at the same time. For design purposes, the structural engineer should decide whether static and dynamic settlement will be combined or not.

10.6 Pipe Design Parameters

Structural design of pipelines requires proper evaluation of all possible loads acting on pipes. The stresses and strains induced on buried pipes depend on many factors, including the type of soil, density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils. The recommended values of the various soil parameters for the pipe design are provided in Table No. 9, *Soil Parameters for Pipe Design*.

Table No. 9, Soil Parameters for Pipe Design

Soil Parameters	Euclid Ave.	Merrill Ave.	Bon View Ave.	Schaefer Ave.
Unit weight of compacted backfill (assuming 92% average relative compaction), γ (pcf)	129	128	129	129
Angle of internal friction of soils, ϕ (degree)	27	28	28	30
Soil cohesion, c (pcf)	100	100	50	20
Coefficient of friction between concrete and native soils, fs	0.30	0.30	0.30	0.30

Soil Parameters	Euclid Ave.	Merrill Ave.	Bon View Ave.	Schaefer Ave.
Coefficient of friction between HDPE pipe and native soils, fs	0.25	0.25	0.25	0.25
Bearing pressure against Alluvial Soils (psf)	1,800 psf	1,800 psf	2,000 psf	2,000 psf
Coefficient of passive earth pressure, Kp	2.66	2.77	2.77	3.0
Coefficient of active earth pressure, Ka	0.38	0.36	0.36	0.33
Modulus of Soil Reaction, E' (psi)	1,500 psi	1,500 psi	1,500 psi	1,500 psi

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

10.7 Bearing Pressure for Anchor and Thrust Blocks

An allowable net bearing pressure presented in Table No. 9, *Soil Parameters for Pipe Design* may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 18 inches wide.

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

10.8 Soil Corrosivity

Four representative soil samples (1 from treatment facility site and 3 from pipeline alignment) were tested for corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program* and design recommendations pertaining to soil corrosivity are presented below.

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

We anticipate that concrete structures such as footings, slabs, and flatwork will be exposed to moisture from precipitation and irrigation. Based on the site location and improvements, we anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt or brackish water. ACI specifies exposure category C2 where concrete is exposed to moisture and external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 5,000 psi, maximum water cement ratio of 0.4 and maximum chloride content of 0.15 percent. Concrete cover should be used as an additional provision.

The measured value of the minimum electrical resistivity of the sample when saturated was 670 ohm-cm for the treatment facility site. This indicates that the soils tested are severely corrosive to ferrous metals in contact with the soil (Romanoff, 1957).

The measured value of the minimum electrical resistivities of the samples when saturated were 1,419 to 3,133 ohm-cm along pipeline alignment. This indicates that the soils tested are moderately corrosive to corrosive to ferrous metals in contact with the soil (Romanoff, 1957).

According to the Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive if the pH is 5.5 or less, or chloride content is 500 parts per million (ppm) or greater, or sulfate content is 1,500 ppm or greater, or resistivity less than 2,000 ohm-cm. Based on the tested results, tested soils are considered corrosive.

No mitigation is needed for HDPE pipe. However, converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the facility site and pipeline alignment soils.

10.9 Asphalt Concrete Pavement

Three soil sample were tested to determine the R-value of the subgrade soils. Based on laboratory testing, R-values were between 14 and 31. For pavement design, we have utilized an R-value of 14 and 30 and design Traffic Indices (TIs) of 8 and 12, depending on the street.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2017), Chapter 630 with a safety factor of 0.2 for asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table.

Table No. 10, Recommended Preliminary Pavement Sections

Street	Design R-value	Traffic Index (TI)	Pavement Section		
			Option 1		Option 2
			Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
Euclid Ave. (SR 83)	30	10*	7.0	13.0	15.0
		12*	8.0	16.0	18.0
Merrill, Bon View and Schaefer Ave.	14	8	6.0	12.0	14.0

(*whichever TI is applicable based on Caltrans)

Pavement section should be based on the Standards of the City of Ontario and Caltrans (Euclid Ave.) or Table No. 10, *Recommended Preliminary Pavement Sections*, whichever is applicable. At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design.

Prior to placement of aggregate base, at least the upper 12 inches of subgrade soils should be scarified, moisture-conditioned if necessary, and recompacted to at least 95 percent of the laboratory maximum dry density as defined by ASTM Standard D1557 test method.

Base materials should conform with the Standards of the City of Ontario and Caltrans or Section 200-2.2,"*Crushed Aggregate Base*," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2018) and should be placed in accordance with Section 301-2 of the SSPWC and Caltrans Standards.

Asphaltic concrete materials should conform to the Standards of the City of Ontario and Caltrans or Section 203 of the SSPWC and should be placed in accordance with Section 302-5 of the SSPWC and Caltrans Standards.

11.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation and shoring design recommendations are presented in the following sections.

11.1 General

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

Vertical braced excavations can be considered for the foundations of the proposed structures and pipeline. Sloped excavations may not be feasible in locations adjacent to existing utilities or structures, or other improvements. Recommendations pertaining to temporary excavations are presented in this section.

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

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

11.2 Temporary Sloped Excavations

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

Table No. 11, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
Silty Sand (SM)	C	0-10	1.5:1
Sandy Silt (ML) and Sandy Clay (CL)	B	0-10	1:1

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

For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trench shields should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring are provided in the following section.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction materials, should not be placed within 5 feet of the unsupported slope edge.

Stockpiled soils with a height higher than 6 feet will require greater distance from trench edges.

11.3 Shoring Design

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

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

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

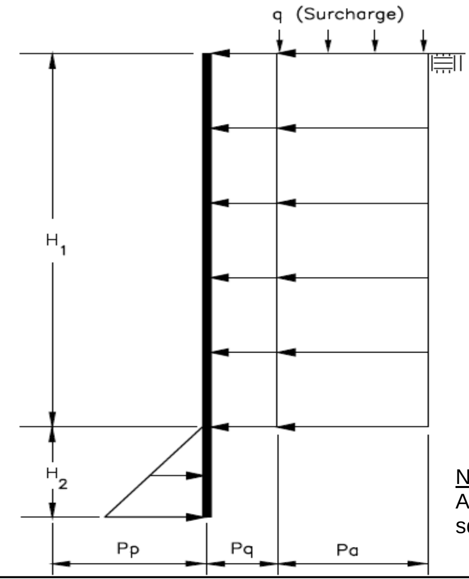
Table No. 12, Lateral Earth Pressures for Temporary Shoring

Lateral Resistance Soil Parameters*	Values
Active Earth Pressure (Braced Shoring) (psf) (A)	24
Active Earth Pressure (Cantilever Shoring) (psf) (B)	40
At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)	60
Passive earth pressure (psf per foot of depth) (D)	230
Maximum allowable bearing pressure against native soils (psf) (E)	2,000
Coefficient of friction between sheet pile and native soils, fs (degree) (F)	0.30

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

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

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



Total Earth Pressure, P

P = Pq + Pa

Pq = 0.5q - incremental surcharge pressure

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

Lateral Pressure Resistance

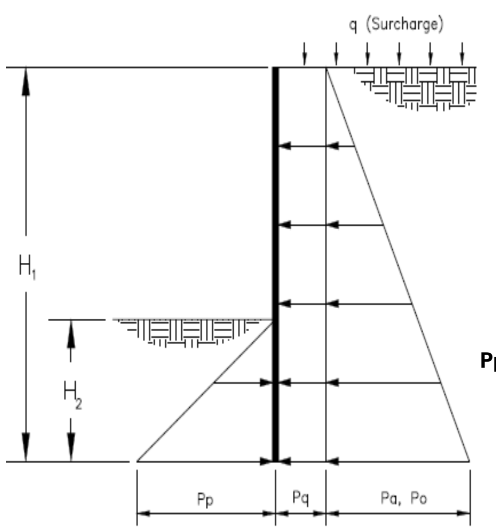
Pp = (D) $H_2 \leq$ (E) psf - passive earth pressure (on native soils)

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

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

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

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



Total Earth Pressure, P

P = Pq + Pa, Po

Pq = 0.5q - incremental surcharge pressure

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

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

Lateral Pressure Resistance

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

μ = (F) - ultimate friction coefficient between steel

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

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

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

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

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

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

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

12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

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

13.0 CLOSURE

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

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

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

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

14.0 REFERENCES

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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 of drilling soil borings. During the site reconnaissance, the surface conditions were noted and the borings were marked at locations approved by Alex Rahimian-Pour with Hazen and Sawyer. The approximate boring locations were established in the field by reference to existing treatment facility site, street centerlines, property boundaries, and other visible features. The locations should be considered accurate only to the degree implied by the method used.

One exploratory boring (BH-01) was drilled on May 29, 2019 to investigate subsurface conditions at the EWTF site. Due to close proximity of existing underground utilities, a 4-inch diameter hand auger was used to drill the upper 10 feet below existing ground surface (bgs). The boring was drilled to the planned maximum depth of 51.5 feet bgs.

Twelve exploratory borings (BH-02 through BH-13) were drilled between May 28 and 30, 2019 to investigate subsurface conditions along the pipeline alignment. Due to close proximity of existing underground utilities, a 4-inch diameter hand auger was used to drill the upper 5 feet for all borings. The borings were drilled to the planned maximum depth of 16.5 feet bgs except BH-11 which was drilled at a depth of 10.0 feet bgs using hand auger.

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

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

Standard Penetration Testing (SPT) was also performed in accordance with the ASTM Standard D1586 test method at 10-foot intervals beginning at 20 feet bgs in the boring BH-01 using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

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

Following the completion of logging and sampling, the borings were backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete, where needed. If construction is delayed, the surface may settle over time. So, we recommend the owner monitor the boring locations and backfill any depressions that might occur, or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement, if possible.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 through A-14, *Logs of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			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

Apparant 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

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **A-1**

Project ID: 18-81-287-01.GPJ; Template: KEY

Log of Boring No. BH-01

Dates Drilled: 5/29/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 703 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
0		**ALLUVIUM** **SILTY SAND (SM):** fine to medium-grained, scattered gravel up to 1.5" in largest dimension, light brown.		X				Hand augered to 10 feet bgs ei, ca, er ma
10			■		6/12/17	7	94	ds
15		**SANDY CLAY (CL):** fine to medium-grained sand, grayish brown.	■		5/2/3	35	85	
20		**SILTY SAND (SM):** fine to medium-grained, scattered gravel up to 1.5" in largest dimension, grayish brown.	X		3/10/13			ma
25		**SANDY SILT (ML):** fine to medium-grained sand, brown.	■		8/11/14	11	120	
30		**SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 2" in largest dimension, brown.	X		10/23/26			

Log of Boring No. BH-01

Dates Drilled: 5/29/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 703 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
		ALLUVIUM **SANDY SILT (ML):** fine to medium-grained sand, grayish brown.	■		5/10/11	22	105	
40		**SILTY SAND (SM):** fine to medium-grained, brown.	☒		9/7/14			
45			■		7/21/23	15	112	
50		**SANDY SILT (ML):** fine to medium-grained sand, brown.	☒		9/12/16			
		End of boring at 51.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped on 5/29/19.						

Log of Boring No. BH-02

Dates Drilled: 5/29/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 589 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/5" AGGREGATE BASE						Hand auger to 5 feet bgs.
		ALLUVIUM **CLAYEY SAND (SC):** fine to medium-grained, brown.						
5		**SANDY SILT (ML):** fine to medium-grained sand, trace clay, grayish brown.			3/5/8	14	114	
								se
					3/4/6	20	110	
10					2/3/4	18	106	
15		**SANDY CLAY (CL):** fine to medium-grained, brown.			5/25/	39	84	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/29/19.

Log of Boring No. BH-03

Dates Drilled: 5/29/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 606 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
0		**6" ASPHALT CONCRETE/6" AGGREGATE BASE**						Hand auger to 5 feet bgs.
		ALLUVIUM **SILTY SAND (SM):** fine to medium-grained, scattered gravel up to 1.5" in largest dimension, brown.						
5		**SANDY CLAY (CL):** fine to medium-grained sand, grayish brown.	■	▩	2/2/3	19	101	
			■	▩	3/4/5	22	102	ca, er, ma, max
10		**CLAYEY SAND (SC):** fine to coarse-grained, brown.	■		2/5/6	13	116	
15		**SANDY CLAY (CL):** fine to medium-grained sand, olive brown.	■		2/8/12	34	92	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/29/19.

Log of Boring No. BH-04

Dates Drilled: 5/29/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 618 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
		8" ASPHALT CONCRETE/4" AGGREGATE BASE						Hand auger to 5 feet bgs. r
		ALLUVIUM **SANDY SILT (ML):** fine to medium-grained sand, scattered gravel up to 1.5' in largest dimension, brown.						
5		**SANDY CLAY (CL):** fine to medium-grained sand, dark brown.			2/3/5	37	78	
					2/5/6	31	88	ds
10					2/4/6	29	92	
15		**SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 1.5' in largest dimension, brown.			5/5/6	20	103	wet

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/29/19.

Log of Boring No. BH-05

Dates Drilled: 5/30/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 625 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
0	ML	**ALLUVIUM** **SANDY SILT (ML):** fine to medium-grained sand, scattered gravel up to 2" in largest dimension, dark brown.						Hand auger to 5 feet bgs.
5	CL	**SANDY CLAY (CL):** fine to medium-grained sand, dark brown.	X	X	5/8/7	43	74	
				X				ei
			X	X	4/9/11	25	99	
10		- light brown	X		4/10/23	22	102	
15	ML	**SANDY SILT (ML):** fine to medium-grained sand, grayish brown.	X		4/14/9	18	110	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 5/30/19.

Log of Boring No. BH-06

Dates Drilled: 5/30/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 631 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
		12" ASPHALT CONCRETE/NO AGGREGATE BASE						Hand auget to 5 feet bgs. se, r
		ALLUVIUM **SANDY CLAY (CL):** fine to medium-grained sand, scattered gravel up to 1.5' in largest dimension, dark brown.						
5					3/7/7	23	93	
					4/5/9	24	99	
10					4/7/8	23	100	
15					8/11/14	21	104	
		End of boring at 16.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/30/19.						

Log of Boring No. BH-07

Dates Drilled: 5/30/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 637 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
0		**ALLUVIUM** **SANDY SILT (ML):** fine to medium-grained sand, scattered gravel up to 1.5" in largest dimension, brown.						Hand auger to 5 feet bgs.
5		**SANDY CLAY WITH GRAVEL (CL):** fine to medium-grained sand, brown.	X	X	18/13/12	17	106	
			X	X	5/8/11	22	100	se, ca, er, ma ds
10		- light brown	X		3/6/9	27	94	
15			X		4/12/11	27	92	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 5/30/19.

Converse Consultants

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For: Hazen and Sawyer

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Drawing No. A-8

Log of Boring No. BH-08

Dates Drilled: 5/28/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 642 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
0		**5" ASPHALT CONCRETE/NO AGGREGATE BASE**						Hand augered to 5 feet bgs.
		ALLUVIUM **SILTY SAND (SM):** fine to medium-grained, grayish brown.						
5			X	X	5/7/10	8	103	
		SANDY SILT (ML): fine to medium-grained sand, trace clay, light brown.	X	X	8/21/39	17	106	max
10		- caliche stringers	X		5/10/12	14	91	
15			X		8/17/23	17	104	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/28/19.

Log of Boring No. BH-09

Dates Drilled: 5/28/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 659 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
0		**6" ASPHALT CONCRETE/NO AGGREGATE BASE**						Hand auger to 5 feet bgs. se
		ALLUVIUM **SANDY SILT (ML):** fine to medium-grained sand, grayish brown.		X				
5		**SILTY SAND (SM):** fine to coarse-grained, brown.	X		7/9/11	11	103	
		SANDY SILT (ML): fine to medium-grained sand, trace clay, grayish brown.	X		3/7/18	16	110	ds
10			X		2/8/15	20	107	
15			X		2/16/25	19	108	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/28/19.

Log of Boring No. BH-10

Dates Drilled: 5/28/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 682 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" ASPHALT CONCRETE/NO AGGREGATE BASE						Hand auger to 5 feet bgs.
		ALLUVIUM **SILTY SAND (SM):** fine to medium-grained, grayish brown.						
5		- trace clay			3/6/12	14	106	
		- trace clay			5/10/13	14	103	ca, er, ma
10		**SANDY CLAY (CL):** fine to medium-grained sand, brown.			5/8/20	22	103	
15		**SANDY SILT (ML):** fine to medium-grained sand, trace clay, brown.			3/7/19	17	111	
		End of boring at 16.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/28/19.						

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Drawing No. A-11

Log of Boring No. BH-11

Dates Drilled: 5/28/2019 Logged by: William Buckley Checked By: James Burnham

Equipment: 4" Hand Auger Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 700 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
0–10		**ALLUVIUM** **SANDY CLAY (CL):** fine to medium-grained sand, grayish brown.		X (0–5)				Hand augered to 10 feet bgs r, max
10		End of boring at 10.0 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped on 5/28/19.						

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Converse Consultants

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Drawing No. A-12

Log of Boring No. BH-12

Dates Drilled: 5/28/2019 Logged by: William Buckley Checked By: James Burnham

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

Ground Surface Elevation (ft): 706 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
0		**ALLUVIUM** **SILTY SAND (SM):** fine to medium-grained, grayish brown.						Hand augered to 5 feet bgs.
5		**SANDY SILT (ML):** fine to medium-grained sand, scattered gravel up to 1.5" in largest dimension, some clay, brown.	■	▨	5/5/7	29	90	
		SILTY SAND (SM): fine to coarse-grained, scattered gravel up to 1.5" in largest dimension, brown.	■	▨	5/7/9	11	97	se, ma ds
10		**SANDY SILT (ML):** fine to medium-grained sand, some clay, light brown.	■		7/12/13	19	91	
15			■		6/20/24	14	99	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 5/28/19.

Log of Boring No. BH-13

Dates Drilled: 5/28/2019 | Logged by: William Buckley | Checked By: James Burnham

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

Ground Surface Elevation (ft): 684 | Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
0		**ALLUVIUM** **SILTY SAND (SM):** fine to medium-grained, scatterd gravel up to 1.5" in largest dimension, grayish brown.						
5			■	⊠	2/7/14	5	102	
			■	⊠	5/9/14	10	106	
10		**SANDY SILT (ML):** fine to medium-grained sand, caliche stringers, brown.	■		8/13/15	14	99	
15			■		7/20/29	14	110	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 5/28/19.

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **A-14**

Appendix B

Laboratory Testing Program

APPENDIX B

LABORATORY TESTING PROGRAM

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

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

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

Expansion Index

Two representative bulk samples were tested in accordance with ASTM Standard D4829 to evaluate the expansion potential. The test results are presented in the following table.

Table No. B-1, Expansion Index Test Results

Boring No./ Location	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-01/Facility	0-5	Silty Sand (SM)	0	Very Low
BH-05/Pipeline	5-10	Sandy Clay (CL)	54	Medium

Sand Equivalent

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

Table No. B-2, Sand Equivalent Test Results

Boring No./Location	Depth (feet)	Soil Description	Sand Equivalent
BH-02/Pipeline	5-10	Sandy Silt (ML)	6
BH-06/Pipeline	1-5	Sandy Clay (CL)	6
BH-07/Pipeline	5-10	Sandy Clay (CL)	4
BH-09/Pipeline	1-5	Sandy Silt (ML)	12
BH-12/Pipeline	5-10	Silty Sand to Sandy Silt (SM-ML)	12

R-value

Three bulk soil samples were tested for resistance value (R-value) in accordance with the Caltrans Test Method 301. The test provides a relative measure of soil strength for use in pavement design. The test results are shown in the following table.

Table No. B-3, R-Value Test Results

Boring No./Location	Depth (feet)	Soil Classification	Measured R-value
BH-04/Pipeline	1-5	Sandy Silt (ML)	31
BH-06/Pipeline	1-5	Sandy Clay (CL)	14
BH-11/Pipeline	0-5	Sandy Clay (CL)	14

Soil Corrosivity

Four representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of the tests were to determine the corrosion potential of site soils when placed in contact with common construction materials. The tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance to California Tests 643, 422 and 417. Test results are presented in the following table.

Table No. B-4, Summary of Soil Corrosivity Test Results

Boring No./ Location	Depth (feet)	pH	Soluble Sulfates (CA 417) (% by weight)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
*BH-01/Facility	0-5	9.0	0.0453	262	670
BH-03/Pipeline	5-10	8.6	0.0077	44	1,419
BH-07/Pipeline	5-10	8.7	0.0036	38	2,844
BH-10/Pipeline	5-10	8.8	0.005	40	3,133

(*based on AP Engineering and Testing, Inc., low resistivity value due to higher sulfate and chloride contents)

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analyses were performed on six select samples in accordance with the ASTM Standard D6913 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*. The results are presented in the following table.

Table No. B-5, Grain Size Distribution Test Results

Boring No./ Location	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-01/Facility	0-5	Silty Sand (SM)	1.0	66.0	33.0	
BH-01/Facility	20.0-21.5	Silty Sand (SM)	0.0	74.0	26.0	
BH-03/Pipeline	5-10	Sandy Clay (CL)	0.0	37.0	63.0	
BH-07/Pipeline	5-10	Sandy Clay with Gravel (CL)	13.0	42.0	45.0	
BH-10/Pipeline	5-10	Silty Sand (SM)	0.0	60.0	40.0	
BH-12/Pipeline	5-10	Silty Sand (SM)	2.0	51.0	47.0	

Maximum Density and Optimum Moisture Content

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

Table No B-6, Summary of Moisture-Density Relationship Results

Boring No./Location	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-03/Pipeline	5-10	Sandy Clay (CL), Grayish Brown	10.0	125.5
BH-08/Pipeline	5-10	Silty Sand to Sandy Silt (SM-ML), Grayish Brown	10.0	127.0
BH-11/Pipeline	0-5	Sandy Clay (CL), Grayish Brown	10.5	124.5

Direct Shear

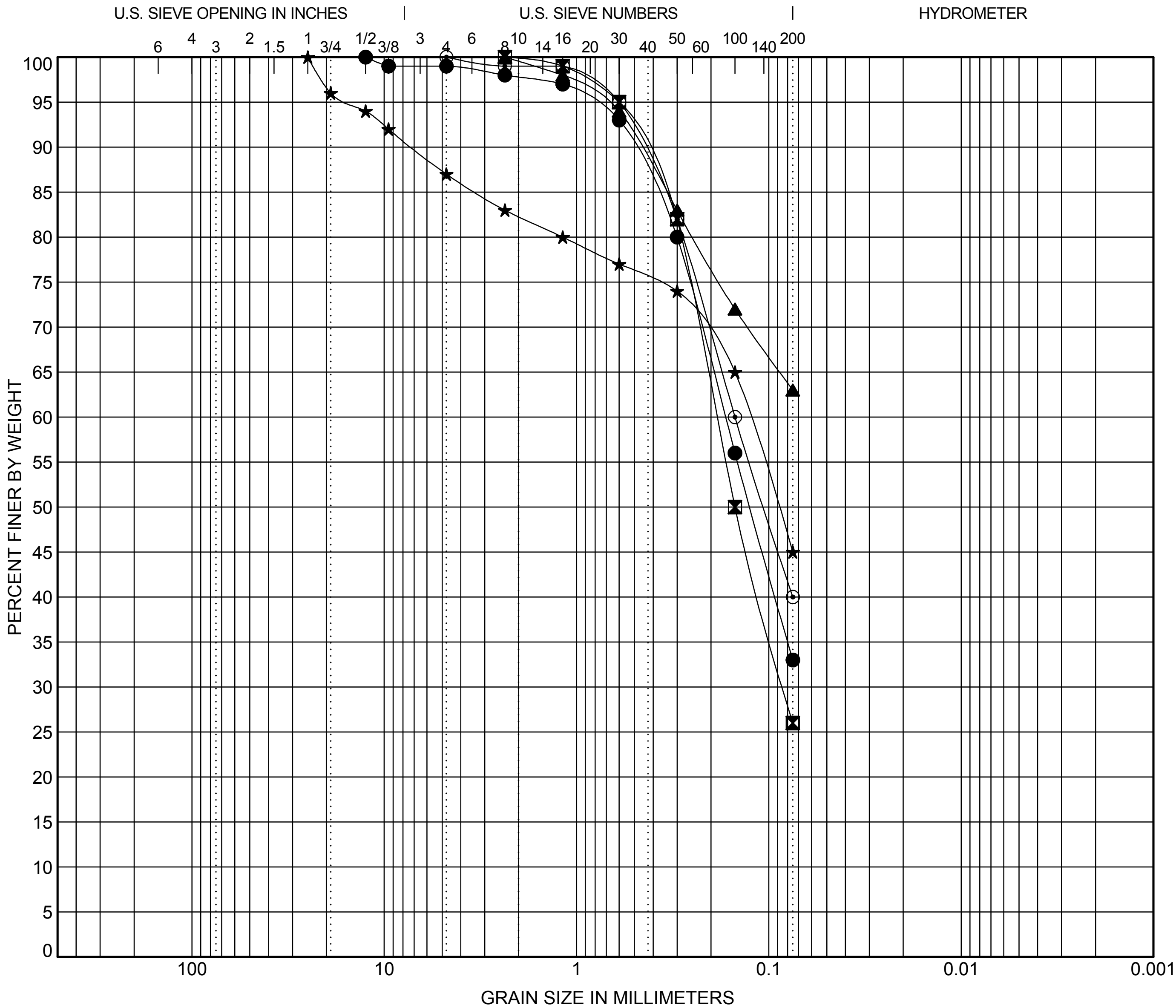
Five direct shear tests were performed on relatively undisturbed samples under soaked moisture conditions in accordance with ASTM Standard D3080. For each test, 3 samples contained in a brass sampler ring were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.01 and 0.02 inch/minute, depending on the sample. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test results, including sample density and moisture content, see Drawings No. B-3 through B-7*, Direct Shear Test Results,* and in the following table.

Table No. B-7, Summary of Direct Shear Test Results

Boring No./ Location	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-01/Facility	10.0-11.5	Silty Sand (SM)	31	90
BH-04/Pipeline	7.5-9.0	Sandy Clay (CL)	27	320
BH-07/Pipeline	7.5-9.0	Sandy Silt (ML)	28	290
BH-09/Pipeline	7.5-9.0	Sandy Silt (ML)	29	250
BH-12/Pipeline	7.5-9.0	Silty Sand (SM)	33	40

Sample Storage

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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-01**	**0-5**	**SILTY SAND (SM)**					
⧗	**BH-01**	**20.0-21.5**	**SILTY SAND (SM)**					
▲	**BH-03**	**5-10**	**SANDY CLAY (CL)**					
★	**BH-07**	**5-10**	**SANDY CLAY WITH GRAVEL (CL)**					
⊙	**BH-10**	**5-10**	**SILTY SAND (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-01**	**0-5**	**12.5**	**0.168**			**1.0**	**66.0**	**33.0**	
⧗	**BH-01**	**20.0-21.5**	**2.36**	**0.186**	**0.084**		**0.0**	**74.0**	**26.0**	
▲	**BH-03**	**5-10**	**2.36**				**0.0**	**37.0**	**63.0**	
★	**BH-07**	**5-10**	**25**	**0.126**			**13.0**	**42.0**	**45.0**	
⊙	**BH-10**	**5-10**	**4.75**	**0.15**			**0.0**	**60.0**	**40.0**	

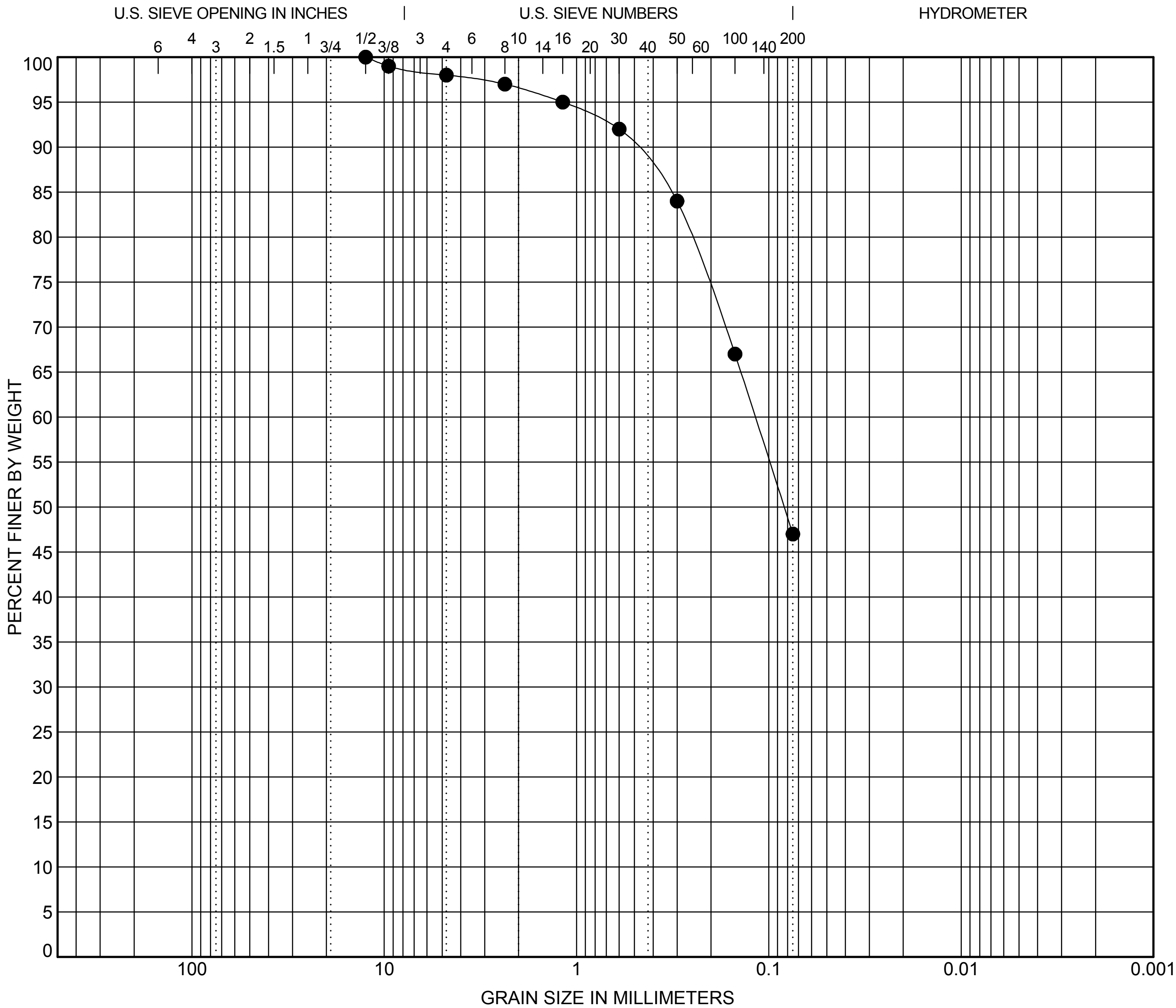
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-1a**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-12**	**5-10**	**SILTY SAND TO SANDY SILT (SM-ML)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-12**	**5-10**	**12.5**	**0.118**			**2.0**	**51.0**	**47.0**	

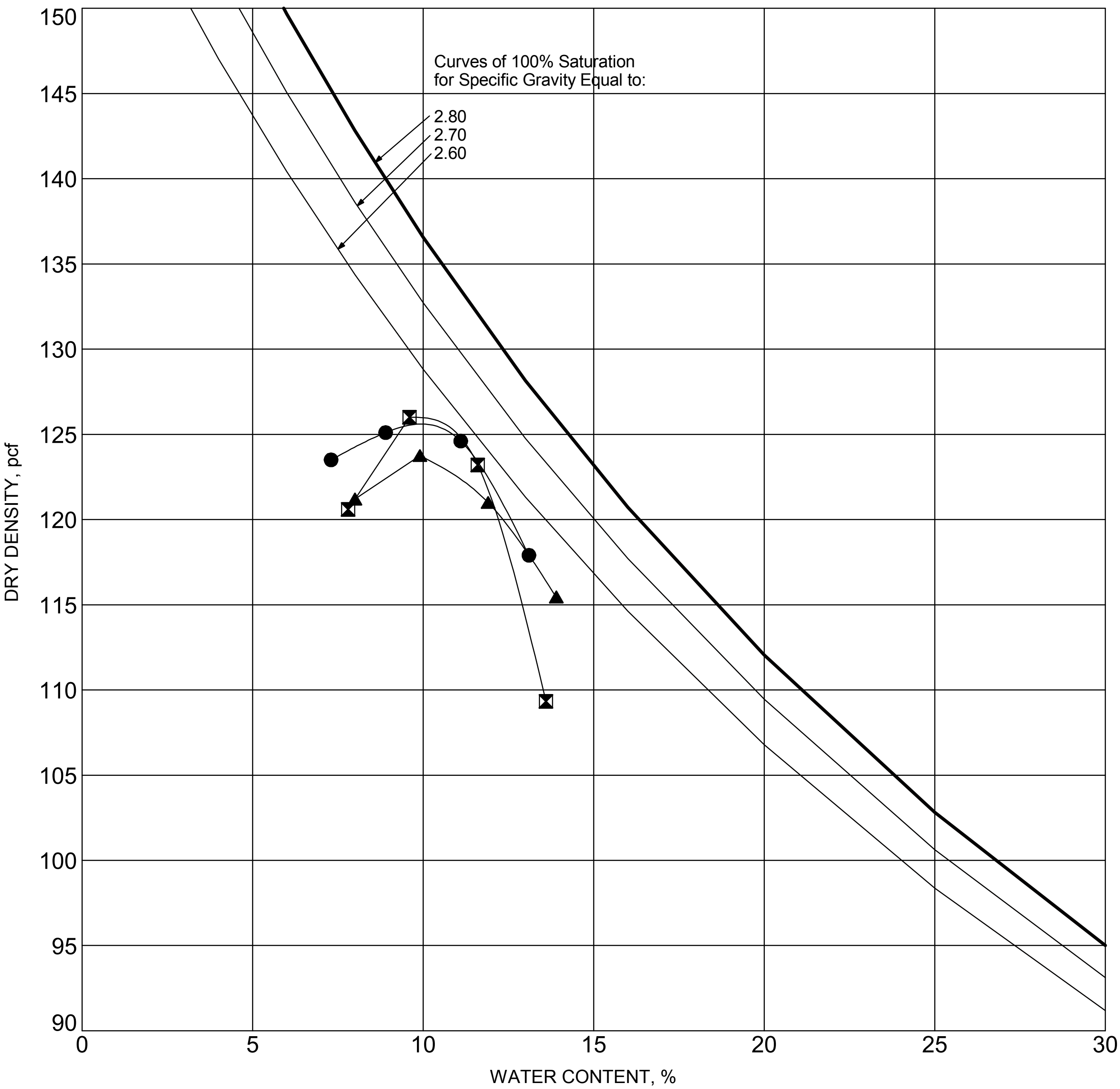
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-1b**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-03	5-10	SANDY CLAY (CL), GRAYISH BROWN	D1557-B	10.0	125.5
⧗	BH-08	5-10	SILTY SAND TO SANDY SILT (SM-ML), GRAYISH BROWN	D1557-A	10.0	127.0
▲	BH-11	0-5	SANDY CLAY (CL), GRAYISH BROWN	D1557-A	10.5	124.5

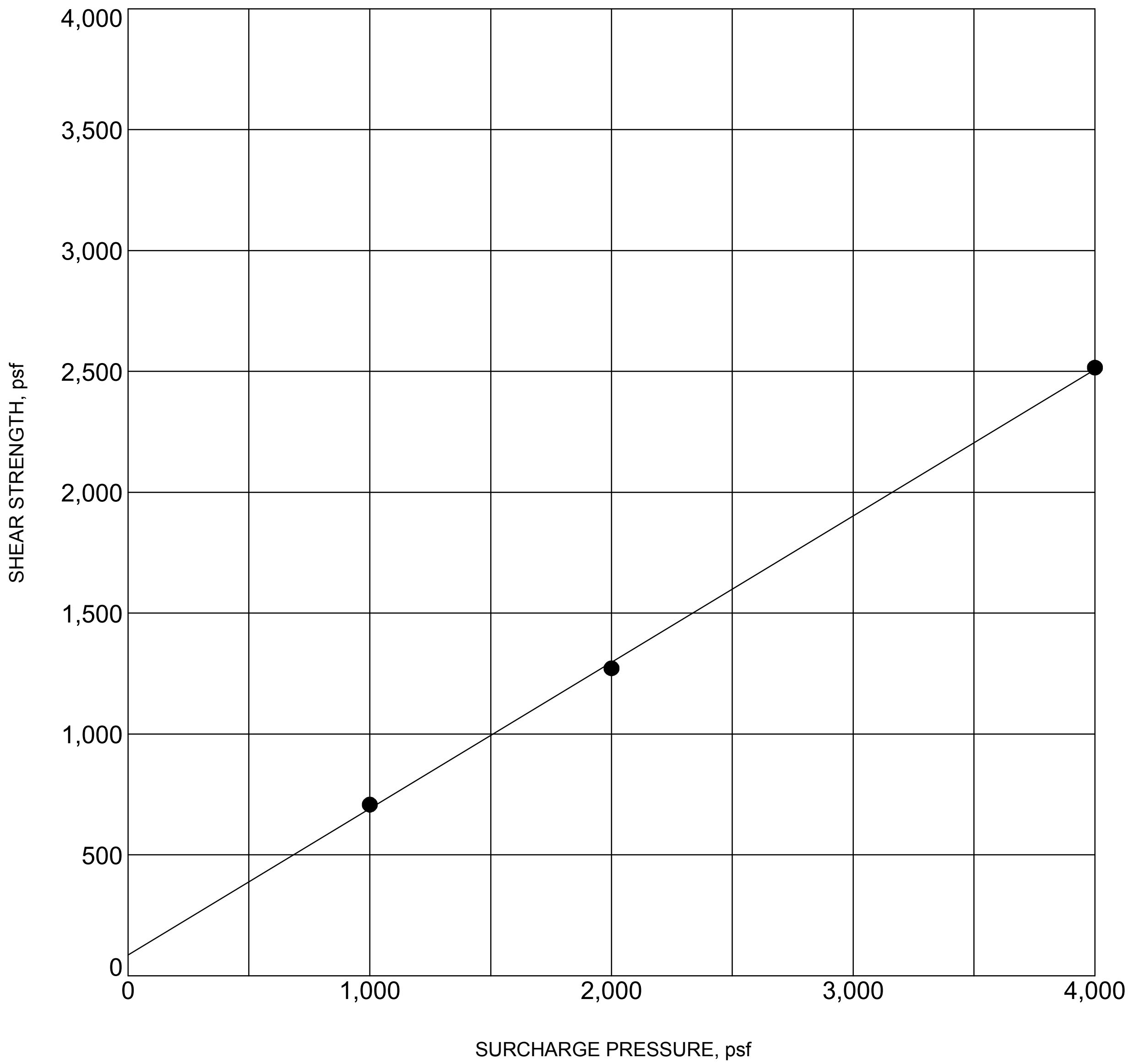
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-2**



BORING NO.	:	**BH-01**	DEPTH (ft)	:	**10.0-11.5**
DESCRIPTION	:	**SILTY SAND (SM)**			
COHESION (psf)	:	**90**	FRICTION ANGLE (degrees):		**31**
MOISTURE CONTENT (%)	:	**7.0**	DRY DENSITY (pcf)	:	**94.0**

NOTE: Ultimate Strength.

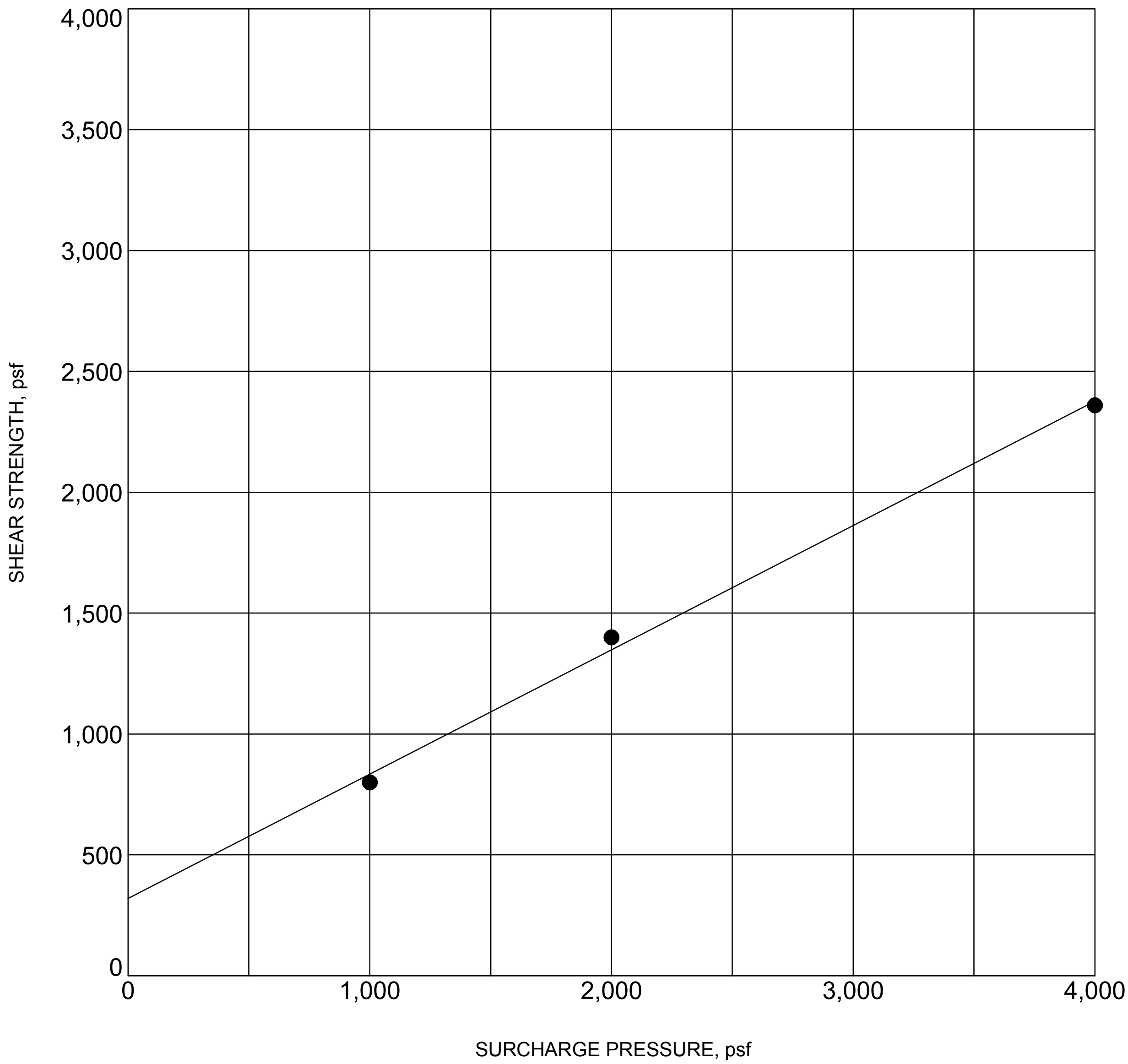
DIRECT SHEAR TEST RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-3**



BORING NO. : **BH-04**	DEPTH (ft) : **7.5-9.0**
DESCRIPTION : **SANDY CLAY (CL)**	
COHESION (psf) : **320**	FRICTION ANGLE (degrees): **27**
MOISTURE CONTENT (%) : **32.1**	DRY DENSITY (pcf) : **87.7**

NOTE: Ultimate Strength.

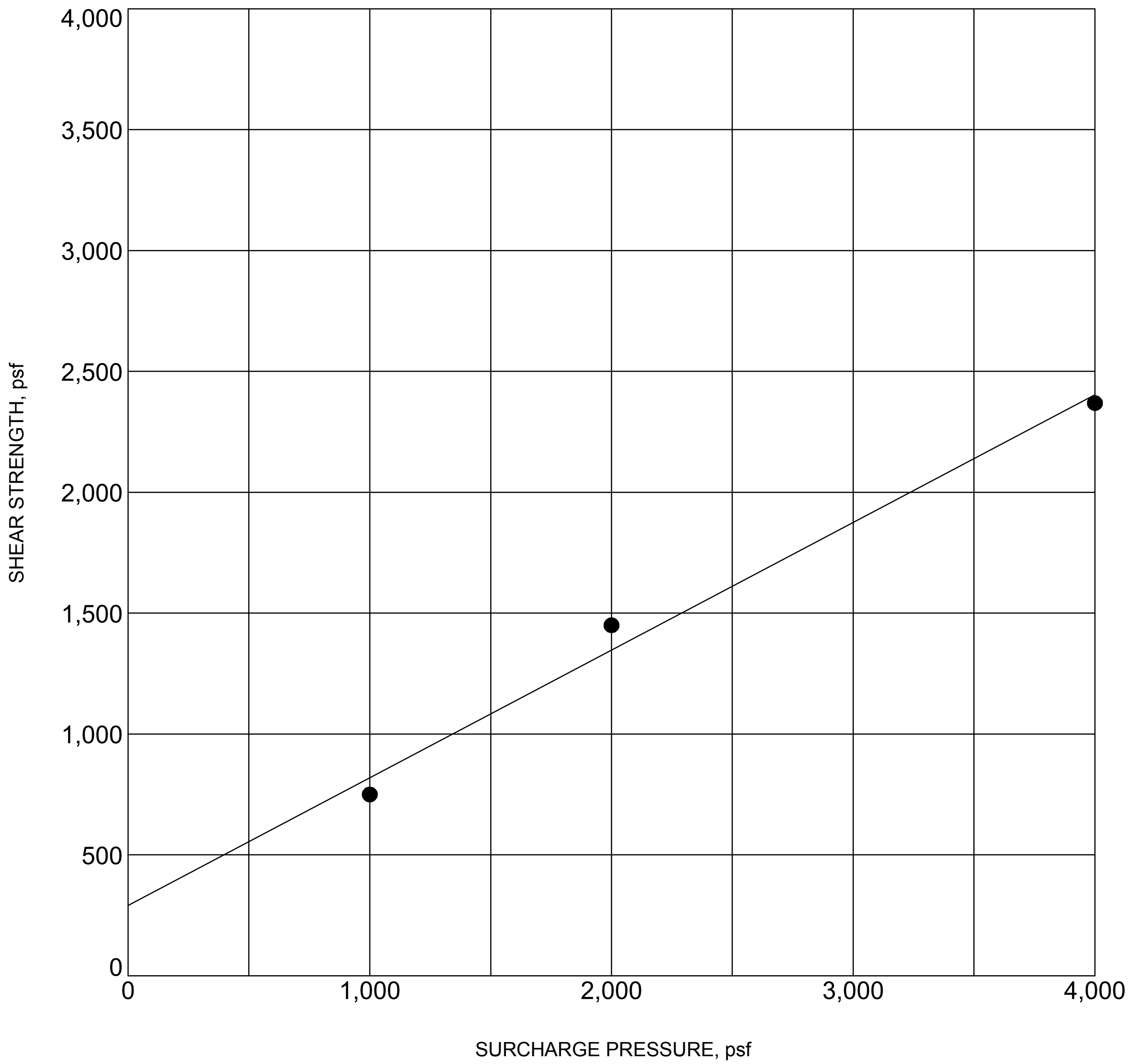
DIRECT SHEAR TEST RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-4**



BORING NO. :	BH-07	DEPTH (ft) :	7.5-9.0
DESCRIPTION :	SANDY CLAY (CL)		
COHESION (psf) :	290	FRICTION ANGLE (degrees):	28
MOISTURE CONTENT (%) :	22.3	DRY DENSITY (pcf) :	100.0

NOTE: Ultimate Strength.

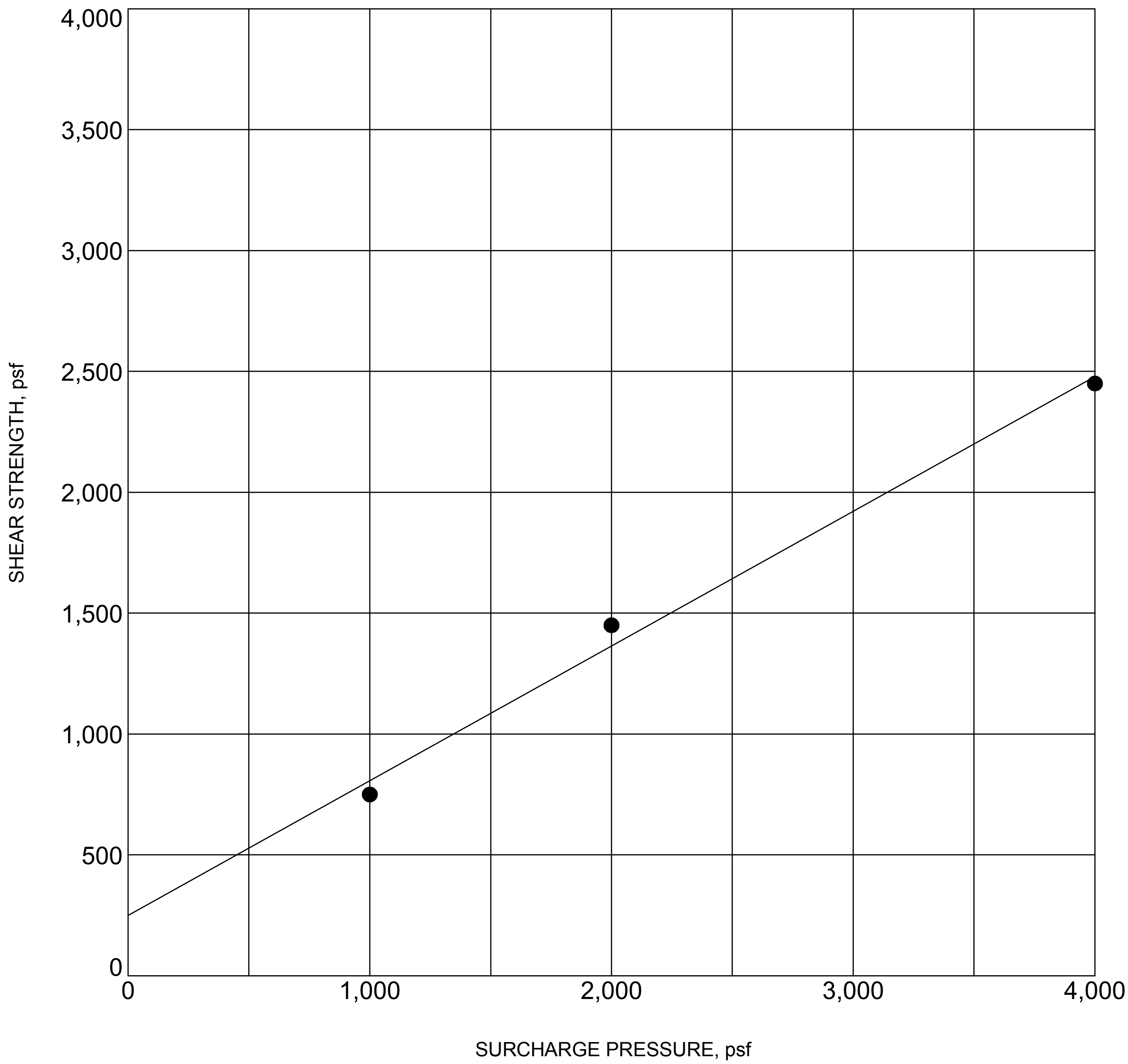
DIRECT SHEAR TEST RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-5**



BORING NO.	:	**BH-09**	DEPTH (ft)	:	**7.5-9.0**
DESCRIPTION	:	**SANDY SILT (ML)**			
COHESION (psf)	:	**250**	FRICTION ANGLE (degrees):		**29**
MOISTURE CONTENT (%)	:	**15.7**	DRY DENSITY (pcf)	:	**109.9**

NOTE: Ultimate Strength.

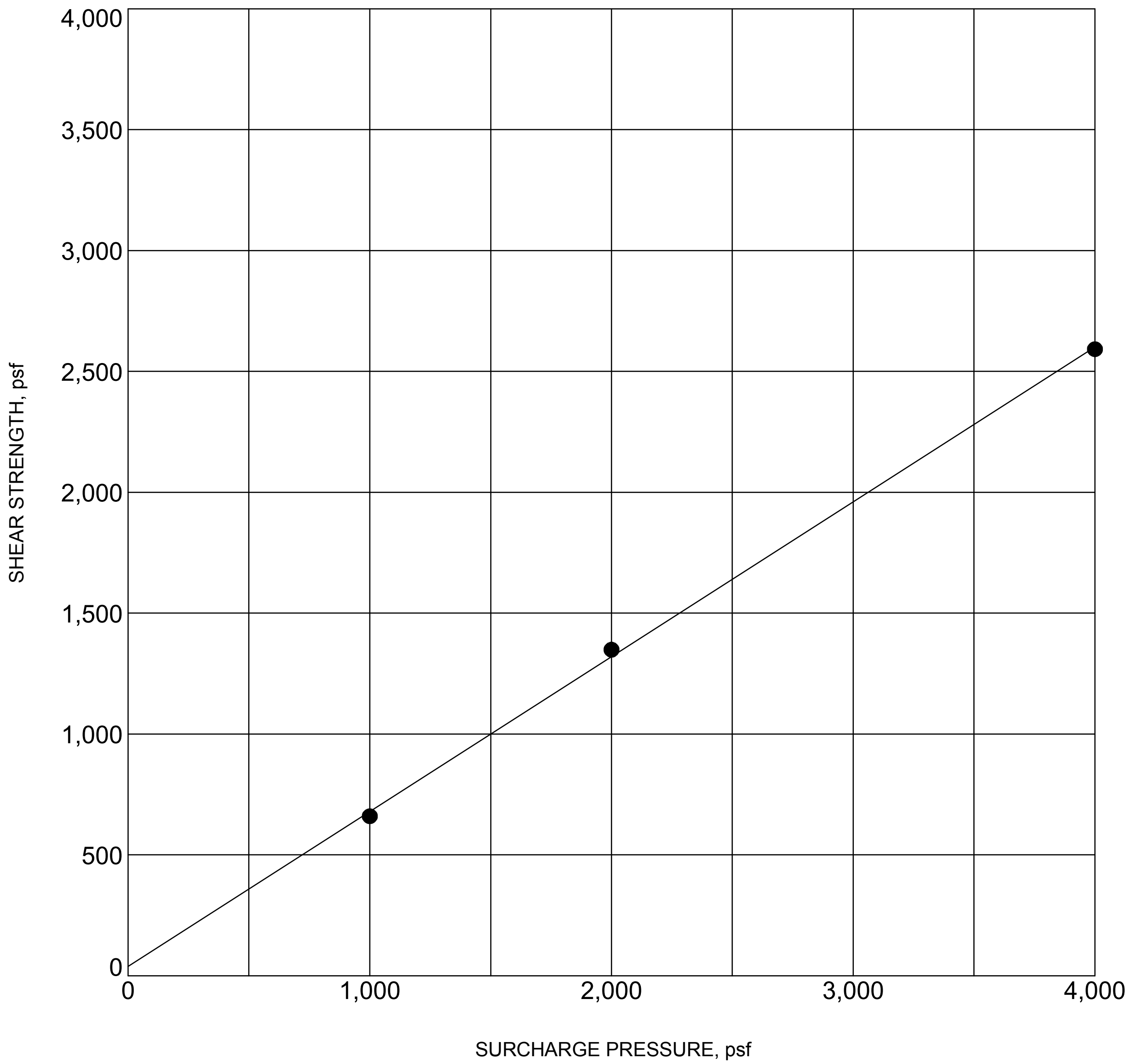
DIRECT SHEAR TEST RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-6**



BORING NO. :	**BH-12**	DEPTH (ft) :	**7.5-9.0**
DESCRIPTION :	**SILTY SAND (SM)**		
COHESION (psf) :	**40**	FRICTION ANGLE (degrees):	**33**
MOISTURE CONTENT (%) :	**11.2**	DRY DENSITY (pcf) :	**97.1**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Eastside Water Treatment Facility Expansion and Offsite Pipeline
Cities of Chino and Ontario, San Bernardino County, California
For: Hazen and Sawyer

Project No. **18-81-287-01**

Drawing No. **B-7**

Appendix C

Liquefaction and Settlement Analysis

APPENDIX C

LIQUEFACTION AND SETTLEMENT ANALYSIS

The subsurface data obtained from the boring BH-01 was used to evaluate the dynamic settlement due to potential densification of relatively loose sediments subjected to ground shaking during earthquakes at the treatment facility site.

A simplified liquefaction hazard analysis was performed using the program SPTLIQ (InfraGEO Software, 2018) using the liquefaction triggering analysis method by Boulanger and Idriss (2014). A modal earthquake magnitude of M 6.76 was selected based on the results of seismic deaggregation analysis using the USGS interactive online tool (https://earthquake.usgs.gov/hazards/interactive/).

A peak ground acceleration (PGA_M) of 0.503g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA was based on the CBC seismic design parameters presented in Section 7.2, *Seismic Design Parameters.*

The result of our analysis is presented on Plate C-1 and summarized in the following table.

Table C-1, Estimated Dynamic Settlement

Location	Groundwater Conditions	Groundwater Depth (feet bgs)	Dry Seismic Settlement (inches)	Liquefaction Induced Settlement (inches)
BH-01	Current	52	0.26	Negligible
	Historical	43		

Based on our analysis, the treatment facility site has the potential for up to 0.3 inches of dry seismic settlement with negligible liquefaction induced settlement under groundwater condition deeper than 50 feet bgs. The dynamic differential settlement of the site may be half of the total settlement over 30 horizontal feet.

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

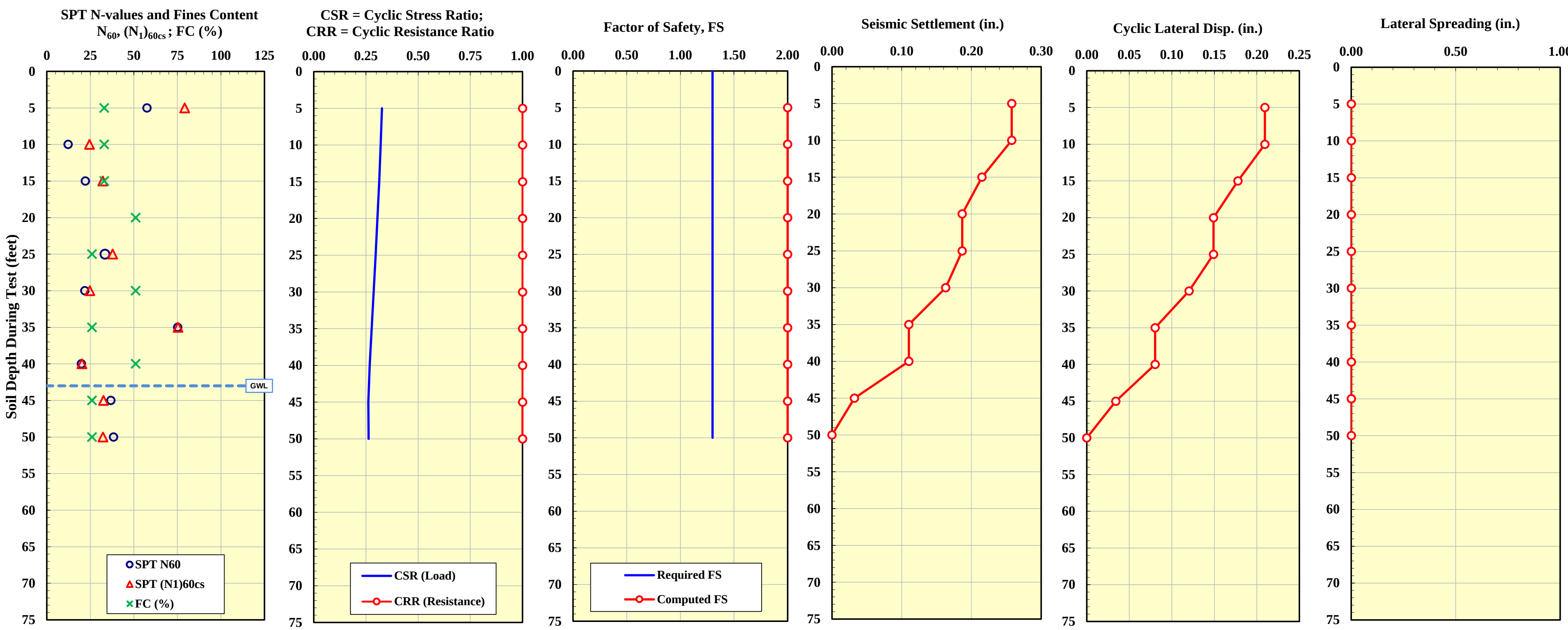
PROJECT INFORMATION	
Project Name	Eastside Water Treatment Facility Expansion and Offsite Pipeline
Project No.	18-81-287-01
Project Location	Cities of Chino and Ontario, San Bernardino County, CA
Analyzed By	Zahangir Alam
Reviewed By	Hashmi Quazi

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

GROUNDWATER LEVEL DATA	
GWL Depth Measured During Test	52.00 feet
GWL Depth Used in Design	43.00 feet

BORING DATA	
Boring No.	BH-01
Ground Surface Elevation	703.00 feet
Proposed Grade Elevation	703.00 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6.76
Peak Ground Acceleration, A_{max}	0.50 g
Required Factor of Safety, FS	1.30



Analysis Methods Used ==>>

Liquefaction Triggering:

Boulanger-Idriss (2014)

Seismic Settlements:

Above GWL: Pradel (1998)

Below GWL: Ishihara and Yoshimine (1992)

Cyclic Lateral Displacements:

Above GWL: Pradel (1998)

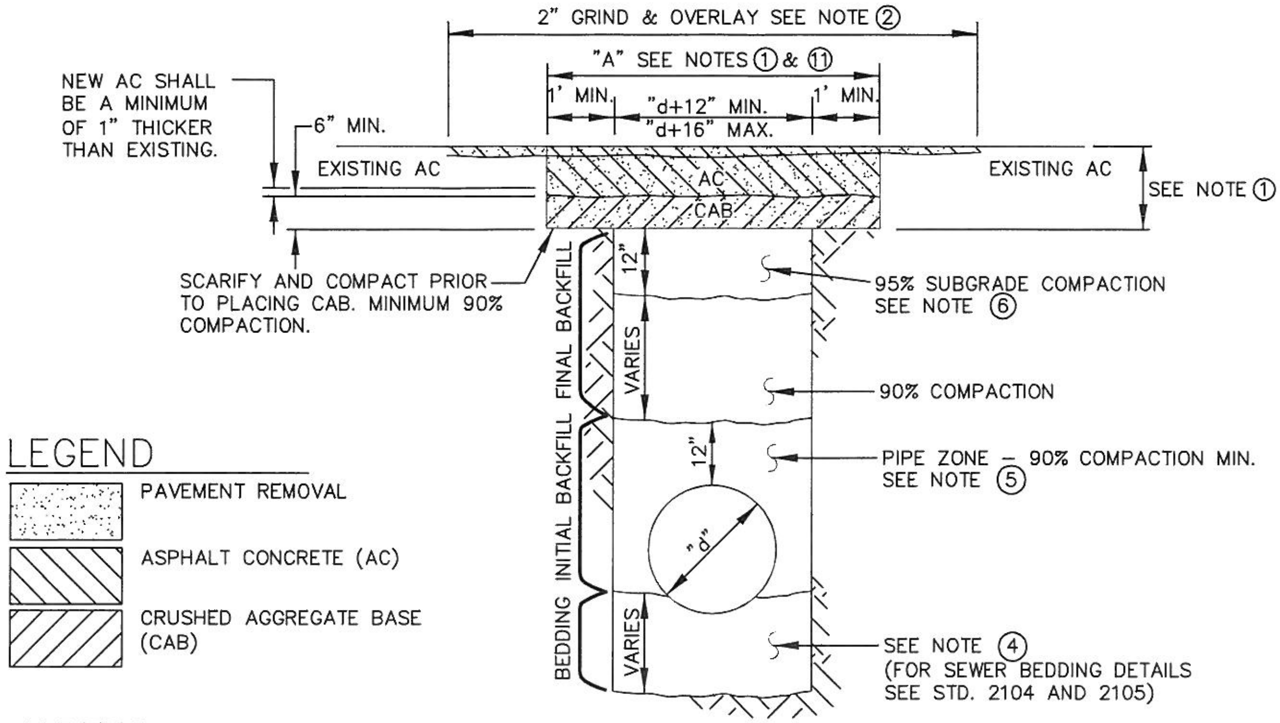
Below GWL: Tokimatsu and Asaka (1998)

Lateral Spreading:

Zhang et al. (2004)

Appendix D

Pipe Bedding and Trench Backfill



NOTES

① EXISTING PAVEMENT SHALL BE WHEEL CUT OR SAW CUT & REMOVED AT THE WIDTH OF DIMENSION "A". THE DEPTH OF THE REMOVAL SHALL BE THE FULL STRUCTURED DEPTH PLUS THE NECESSARY EXCAVATION FOR THE NEW PAVEMENT SECTION AS REQUIRED BY NOTE 9 HEREON.

② FOR LONGITUDINAL TRENCHES OVER 150' IN LENGTH, A MINIMUM 10' WIDE, 2" GRIND AND OVERLAY IS REQUIRED.

③ THE PIPE ZONE WIDTH SHALL BE A MINIMUM OF 12" PLUS THE PIPE DIAMETER AND THE MAXIMUM OF 16" PLUS THE PIPE DIAMETER, IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION (GREEN BOOK).

④ BEDDING MATERIAL SHALL BE USED WHEN THE SAND EQUIVALENT OF THE NATIVE MATERIAL IS LESS THAN 30 AS SPECIFIED IN THE PROJECT PLANS AND SPECIFICATIONS OR AS APPROVED BY THE CITY ENGINEER.

⑤ INITIAL BACKFILL MATERIAL SHALL BE OF SELECT MATERIAL AS SPECIFIED IN THE PROJECT PLANS AND SPECIFICATIONS OR AS APPROVED BY THE CITY ENGINEER. INITIAL BACKFILL SHALL BE COMPACTED TO 90% MIN & TESTED. TEST SHALL BE APPROVED BY THE GEOTECHNICAL ENGINEER AND THE CITY ENGINEER PRIOR TO FINAL BACKFILL.

⑥ FINAL BACKFILL SHALL BE SELECT MATERIALS SPECIFIED IN THE PROJECT PLANS AND SPECIFICATIONS OR NATIVE IF DETERMINED BY THE CITY TO BE ACCEPTABLE AND COMPACTED AS NOTED HEREON. COMPACTION TESTS SHALL BE APPROVED BY THE CITY ENGINEER PRIOR TO PLACEMENT OF C.A.B.

⑦ CRUSHED AGGREGATE BASE (CAB) SHALL BE IN ACCORDANCE WITH SECTION 200–2.2 OF THE STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION ("GREENBOOK") AND SHALL BE COMPACTED & TESTS APPROVED PRIOR TO PLACEMENT OF A.C.

⑧ ASPHALT CONCRETE (AC) SHALL BE REPLACED IN KIND (B–PG 70–10 OR B–AHRM–GG–PG 64–16) AND IN ACCORDANCE WITH SECTIONS 203–06 AND 203–11 OF THE STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION ("GREENBOOK") OR AS APPROVED BY THE CITY ENGINEER.

⑨ COMPACTION TESTS SHALL BE TAKEN EVERY 300', MINIMUM OF 1 PER LOCATION.

⑩ TRENCHES WHICH ARE 30" IN DEPTH OR LESS AND 18" IN WIDTH OR LESS SHALL BE BACKFILLED WITH ONE–SACK CEMENT SLURRY.

⑪ IN THE EVENT OF TRENCH WALL FAILURE, TRENCH LIMITS MAY BE EXTENDED AS DETERMINED BY THE CITY ENGINEER. ADDITIONAL BACKFILL REQUIREMENTS MAY BE REQUIRED. 1' AC/CAB "WING" SHALL BE LOCATED FROM FARTHEST LIMIT OF TRENCH OR TRENCH WALL FAILURE AS DETERMINED BY THE CITY ENGINEER.

⑫ FULL AC REPLACEMENT OF THE ASPHALT BETWEEN THE TRENCH AND THE CURB OR GUTTER SHALL BE REQUIRED FOR ANY TRENCH WHERE THE PAVEMENT REMOVAL IS WITHIN 3' OF CURB OR GUTTER.

⑬ REMOVAL OF 6 OR MORE SEPARATE AREAS OF PAVEMENT WITHIN A 150' LONGITUDINAL LENGTH OF STREET SHALL REQUIRE A TYPE II SLURRY SEAL EXTENDED 5' BEYOND THE LIMITS OF THE OUTERMOST PAVEMENT REMOVAL.

⑭ THERE IS A 3 YEAR MORATORIUM ON NEWLY PAVED STREETS. ANY TRENCHING WITHIN THIS PERIOD REQUIRES APPROVAL FROM THE CITY ENGINEER.

REGISTERED PROFESSIONAL ENGINEER
LOUIS ABI YOUNES
No. 44485
Exp. 03/31/12
CIVIL
STATE OF CALIFORNIA

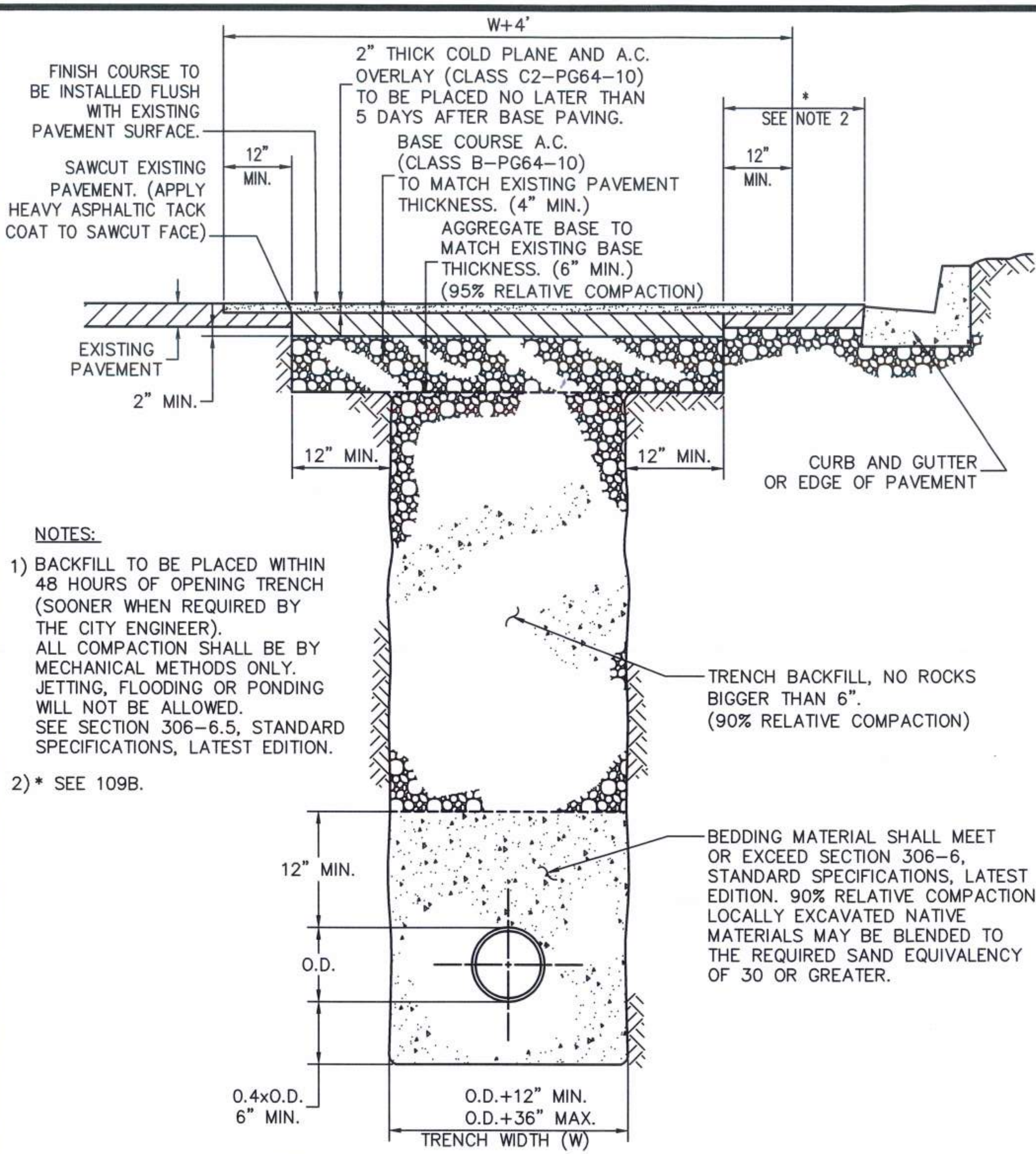
APPROVED BY:
LOUIS ABI–YOUNES CITY ENGINEER — RCE 44485 — DATE 3/22/10
RECOMMENDED:
ASSISTANT CITY ENGINEER — RCE 47462 — DATE 3/22/10

REV	DESCRIPTION	BY	APP'D	DATE

CITY OF ONTARIO

TRENCH EXCAVATION/BACKFILL AND PAVING

STANDARD DRAWING NUMBER **1306**



NOTES:

1) BACKFILL TO BE PLACED WITHIN 48 HOURS OF OPENING TRENCH (SOONER WHEN REQUIRED BY THE CITY ENGINEER).
ALL COMPACTION SHALL BE BY MECHANICAL METHODS ONLY.
JETTING, FLOODING OR PONDING WILL NOT BE ALLOWED.
SEE SECTION 306-6.5, STANDARD SPECIFICATIONS, LATEST EDITION.

2) * SEE 109B.

ALL TRENCHING AND BACKFILL SHALL BE DONE IN ACCORDANCE WITH SECTION 306, STANDARD SPECIFICATIONS, LATEST EDITION.

APPROVED [signature] CITY ENGINEER		10/25/17 DATE	**CITY OF CHINO PUBLIC WORKS DEPARTMENT**	
DATE	REVISION	BY	STANDARD DRAWING	No.
			TRENCH BACKFILL & ROADWAY REPAIR	**109 A**

1) ALL EXISTING STRIPING, RAISED PAVEMENT MARKERS (RPM'S), AND LOOP DETECTORS WITHIN THE LIMITS OF CONSTRUCTION SHALL BE REINSTALLED TO THE SATISFACTION OF THE CITY ENGINEER.

2) IF THE OUTER EDGE OF A UTILITY TRENCH CUT IS LESS THAN 4 FEET FROM THE EDGE OF GUTTER, THE CURB WHERE NO GUTTER EXISTS, OR THE EDGE OF PAVEMENT WHERE NO CURB EXISTS, THE REMAINING PORTION OF THE EXISTING PAVEMENT SECTION SHALL BE REMOVED AND REPLACED IN-KIND PRIOR TO THE GRIND AND OVERLAY PORTION OF THE UTILITY TRENCH CUT STREET RESURFACING TO THE SATISFACTION OF THE CITY ENGINEER.

3) IF TWO (2) OR MORE TRENCH OPENINGS ARE LESS THAN FIVE (5) FEET APART, THE REMAINING PORTION OF THE EXISTING PAVEMENT SECTION SHALL BE REMOVED AND REPLACED IN-KIND PRIOR TO THE GRIND AND OVERLAY PORTION OF THE UTILITY TRENCH CUT STREET RESURFACING TO THE SATISFACTION OF THE CITY ENGINEER.

4) IF THE UTILITY TRENCH CUT IS WITHIN EXISTING CONCRETE PAVEMENT, THE ENTIRE CONCRETE PANEL AFFECTED BY THE UTILITY TRENCH CUT SHALL BE RESTORED IN-KIND.

5) ALL ASPHALT CONCRETE PLACED FOR THE UTILITY TRENCH CUT STREET RESURFACING SHALL BE APPLIED BY A PAVING MACHINE IN ORDER TO ELIMINATE THE UNEVEN "WASHBOARD" EFFECT THAT RESULTS FROM THE MANUAL PLACEMENT OF ASPHALT OVERLAY.

6) PAVEMENT SURFACES MADE OF SPECIAL MATERIALS (I.E. PAVERS, STAMPED CONCRETE, RUBBERIZED ASPHALT) SHALL BE RESTORED IN-KIND.

7) PRIOR TO THE PLACEMENT OF PERMANENT PAVING, THE EXISTING PAVEMENT SHALL BE CUT TO A NEAT STRAIGHT LINE. ANY CRACKED PAVEMENT ADJACENT TO THE TRENCH SHALL BE REMOVED.

8) ALL LIQUIDS GENERATED BY SAWCUTTING SHALL BE COLLECTED AND DISPOSED OF OFF-SITE.

9) ALL USA MARKINGS SHALL BE REMOVED WITHIN 30 DAYS OF COMPLETION OF CONSTRUCTION.

10) THE USE OF CONTROLLED LOW STRENGTH MATERIAL (CLSM) FOR TRENCH BACKFILL SHALL BE SPECIFICALLY APPROVED BY THE CITY ENGINEER.

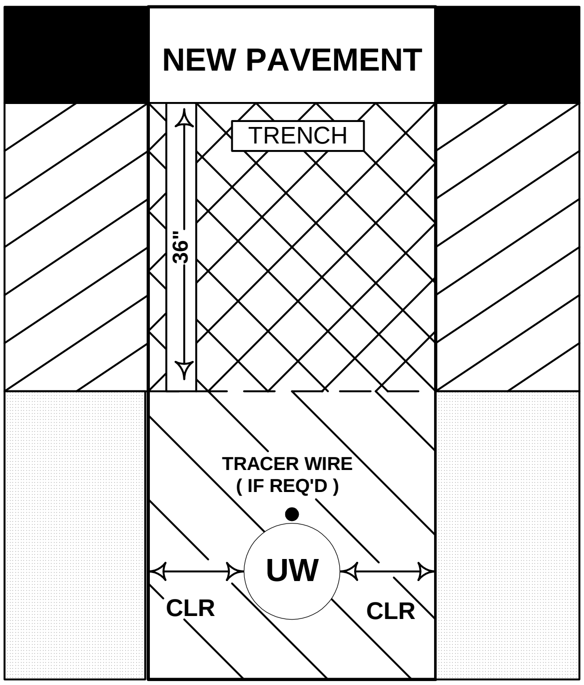
APPROVED	[signature] CITY ENGINEER	10/25/17 DATE	CITY OF CHINO PUBLIC WORKS DEPARTMENT	
DATE	REVISION	BY	STANDARD DRAWING	No.
			TRENCH BACKFILL & ROADWAY REPAIR	109 B

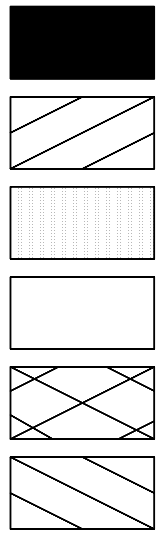
STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

ENCROACHMENT PERMIT TRENCH DETAIL

TR-0153 (REV. 09/2006)

TYPICAL TRENCH DETAIL





Existing Pavement
(Existing HMA Pavement)

Existing Base

Existing Subgrade

New Pavement
(New HMA Pavement)
match existing + 1" - (MAX 7")

New Pavement Base

New Subgrade

UW - Width of Utility or Culvert HMA - Hot Mix Asphalt

CLR - Clearance between product and trench wall TW - Trench Width

CLSM - Controlled Low Strength Material

STRUCTURE BACKFILL SHALL CONFORM TO SECTION 19 - 3.06 OF THE STANDARD SPECIFICATIONS

SLURRY CEMENT BACKFILL SHALL CONFORM TO SECTION 19 - 3.062 OF THE STANDARD SPECIFICATIONS

HMA SHALL CONFORM TO SECTION 39 OF THE STANDARD SPECIFICATIONS

ALL METHODS OF COMPACTION SHALL BE BY MECHANICAL MEANS. PONDING, JETTING OR FLOODING SHALL NOT BE ALLOWED.

AGGREGATE BASE SHALL CONFORM TO SECTION 26 OF THE STANDARD SPECIFICATIONS

WHEN CLSM IS UTILIZED THE MIX DESIGN AND TEST RESULTS SHALL BE SUBMITTED TO THE STATE'S REPRESENTATIVE.

ALL WORK SHALL BE AS AUTHORIZED BY THE APPROVED ENCROACHMENT PERMIT PLANS, AND/OR AS DIRECTED BY THE STATE'S REPRESENTATIVE.

WHEN THE UW IS ≥ 6" THEN THE MINIMUM CLR SHALL BE 6"

COLD PLANING AND RE-SURFACING OVERLAY SHALL BE PARALLEL TO THE ROADWAY AND TO THE NEAREST LANE LINE FOR THE ENTIRE LENGTH OF THE TRENCH/DISTURBED AREAS, AND/OR AS DIRECTED BY THE STATE'S REPRESENTATIVE.

WHEN COLD PLANING IS REQUIRED, THE MINIMUM SHALL BE 0.10' OR AS DIRECTED BY THE STATE'S REPRESENTATIVE TO ACCOMODATE FIELD CONDITIONS.

COLD PLANING MAY BE REQUIRED AT THE DIRECTION OF THE STATE'S REPRESENTATIVE TO ACCOMODATE THE PLACEMENT OF STEEL PLATES.

WHEN TRENCH PLACEMENT IS WITHIN 4' OF CURB & GUTTER, ADDITIONAL COLD PLANING MAY BE REQUIRED AT THE DISCRETION OF THE STATE'S REPRESENTATIVE.

ANY PAVEMENT MARKINGS AND/OR STRIPING REMOVED OR DAMAGED DURING CONSTRUCTION SHALL BE REPLACED AS DIRECTED BY THE STATE'S REPRESENTATIVE.

A TRACER WIRE SHALL BE PLACED ON TOP OF THE FACILITY, WHEN REQUIRED BY THE STATE'S REPRESENTATIVE.

OTHER TRENCH RELATED DETAILS ARE SHOWN IN FIGURE 6.1, CHAPTER 6 OF THE ENCROACHMENT PERMITS MANUAL.

A PAINT BINDER (TACK COAT) OF ASPHALTIC EMULSION CONFORMING TO SECTION 39-4.02, PRIME COAT & PAINT BINDER (TACK COAT) SHALL BE FURNISHED AND APPLIED.

NEW PAVEMENT BASE SHALL CONSIST OF EITHER CL. II AGGREGATE BASE, 2-SACK SLURRY CEMENT, OR CLSM. WHEN TW IS < 24," CL. II AGGREGATE BASE IS NOT RECOMMENDED FOR BACKFILL.

NEW SUBGRADE SHALL CONSIST OF EITHER CL. II AGGREGATE BASE, 2-SACK SLURRY CEMENT, OR CLSM. WHEN TW IS < 24," CL. II AGGREGATE BASE IS NOT RECOMMENDED FOR BACKFILL.