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SCREW PRESS SLUDGE DEWATERING PROJECT

FOR

COUNTY SERVICE AREA (CSA) 70 – GLEND HELEN
DEVORE, CALIFORNIA

WARNING: ALL INDIVIDUALS INTERESTED IN BIDDING ON THIS PROJECT MUST OBTAIN THE FINAL PLANS AND SPECIFICATIONS FROM THE DEPARTMENT MANAGING THE PROJECT OR AS OTHERWISE STATED IN THE ADVERTISEMENT FOR BIDS FOR THE PROJECT. DO NOT USE THE PLANS AND SPECIFICATIONS POSTED ON THE CLERK OF THE BOARD'S WEBSITE FOR BIDDING THE PROJECT.

SAN BERNARDINO COUNTY

SECTION H

APPENDIX “A” GEOTECHNICAL REPORT AND APPENDIX “B” POTHOLE DATA

SCREW PRESS SLUDGE DEWATERING PROJECT

FOR

COUNTY SERVICE AREA (CSA) 70 – GLEN HELEN
DEVORE, CALIFORNIA

PROJECT NO. 30.30.0028

APPENDIX "A"
GEOTECHNICAL REPORT

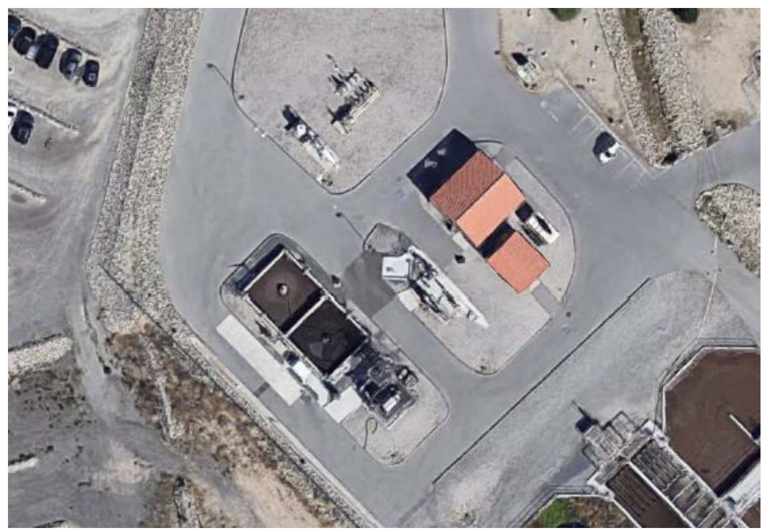
Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

GEOTECHNICAL INVESTIGATION REPORT

SCREW PRESS AT THE LYTLE CREEK NORTH WATER RECYCLING FACILITY

CITY OF SAN BERNARDINO, SAN BERNARDINO COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 18-81-226-01



Prepared For:

SAN BERNARDINO COUNTY
SPECIAL DISTRICTS DEPARTMENT
Mr. Tim Millington
Division Manager
157 West Third St., Second Floor
San Bernardino, CA 92415

Presented By:

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

September 19, 2018

Converse Consultants

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

September 19, 2018

Mr. Tim Millington
Division Manager
San Bernardino County Special Districts Department
157 West Third Street, Second Floor
San Bernardino, CA 92415

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Screw Press at the Lytle Creek North Water Recycling Faciltity
Institution Road
City of San Bernardino, San Bernardino County, California
Converse Project No. 18-81-226-01

Dear Mr. Millington:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of a screw press at the Lytle Creek North Water Recycling Faciltiy (WRF) located south of Institution Road in the City of San Bernardino, San Bernardino County, California. This report was prepared in accordance with our proposal dated July 13, 2018 and your Special Districts Work Oder No. 18407-900 dated July 19, 2018.

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 the San Bernardino County Special Districts Department (SBCSDD). 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.: 3/Addressee
HSQ/JB/ZAkvg

2021 Rancho Drive, Suite 1, Redlands CA 92373
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PROFESSIONAL CERTIFICATION

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

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

REGISTERED PROFESSIONAL ENGINEER
HASHMI S. E. QUAZI
NO. GE 2517
Exp. 6/19
GEOTECHNICAL
STATE OF CALIFORNIA

Zahangir Alam, PhD, EIT
Senior Staff Engineer

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

EXECUTIVE SUMMARY

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

- The Lytle Creek North Water Recycling Facility is located south of the Institution Road in the City of San Bernardino, San Bernardino County, California. The facility is bounded by Institution Road to the north, vacant land and parking lot to the west and east and by existing structure to the south. The existing sludge screening facility is approximately 23’ x 31.5’ and contains a concrete pad of varying in thickness. The current surface of the proposed truck loadout area is covered with gravel. The site is flat with an elevation of approximately 1,643 feet above mean sea level (amsl). The coordinates for the project site are approximately 34.1749° north latitude and 117.3894° west longitude.

- The intent of this project is to demolish the exisitng equipment and surperstructures located on the existing concrete pad to provide a suitable working pad and equipment foundation for a screw press and dewatered slude auger. A 10.5’ x 12.5’ bridging slab (thickness 8 inches) will be placed on top of the existing pad. The project also includes construction of a covered truck loading pad on the south side of the existing sludge screening pad. The covered structure will be placed on separate pier-founded substructure designed for full wind loading with trolley crane/monorail for handling large parts and polymer totes. The proposed covered structure will be approximately 47’ x 28’, covering the existing concrete pad structure and extending approximately 12 feet south of the existing pad. Drainage improvements are also proposed for the truck loadout pad to connect to existing piping.

- 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 August 1, 2018 to investigate subsurface conditions. The boring was drilled to a maximum depth of 20.5 feet below existing ground surface (bgs).

- The subsurface soils at the site primarily of alluvial soils consisting of gravelly silty sand and garvelly sand with trace silt. Gravel up to 2.5 inches in largest dimension were observed in the boring. Though not encountered in the borings, cobbles may be present within the site. Based on hammer blow counts, the upper 10 feet soils are medium dense to dense. Relative compaction of the upper 10 feet soils varies from 83 to 90 percent.

- Groundwater was not encountered in our exploratory boring to the maximum explored depth of 20.5 feet bgs. Based on available data, groundwater is deeper than 48 feet bgs. Groundwater is not expected to be encountered during the construction of this project.

- The project site is not located within a currently designated State of California or San Bernardino County Earthquake Fault Zone. The project site is located adjacent to the San Jacinto Fault zone. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

- 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 at the site was 0, corresponding to very low expansion potential.

- The collapse potential of of the sample tested at the site was 2.1 percent, indicating moderate collapse potential.

- The sulfate and chloride contents of the site soil sample tested correspond to American Concrete Institute (ACI) exposure category S0 and C1, respectively. Design recommendations for these categories are provided in the text of this report.

- The measured values of the minimum electrical resistivity of the sample at the project site when saturated was 9,102 ohm-cm. This indicates that the soils tested are moderately corrosive to ferrous metals in contact with the soils. <u>A corrosion engineer should be consulted for corrosion mitigation measures for ferrous metals in contact with the soil, if necessary.</u>

- Prior to the start of construction all existing underground utilities and appurtenances, if present, should be located at the project site. Prior to place new pad on top of the existing pad, all existing equipment and superstructures should be demolished and removed from the site. All debris, surface vegetation (if any), deleterious material, surficial soils containing gravel, roots and perishable materials (if any) and demolished materials should be stripped and removed from the site.

- Based on our subsurface exploration, we anticipate that the site soil will be excavatable with conventional heavy-duty earthworking and trenching equipment. <u>Excavation will likely be difficult due to the presence of gravel and possible cobbles.</u>

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

- Truck loadout pad area should be overexcavated to a depth of at least 18 inches below pad. The overexcavation should extend at least 2 foot beyond the edge of pad. At least the upper 12 inches of fill beneath pad intended to support vehicle loads should be compacted to at least 95 percent of the laboratory maximum dry density.

- An allowable net bearing capacity of of soils can be taken as 2,200 psf.

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

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

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed by Converse Consultants (Converse) for the proposed design and construction of a screw press at the Lytle Creek North Water Recycling Facility (WRF) located south of Institution Road in the City of San Bernardino, San Bernardino County, California.The project site 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 ptoject.

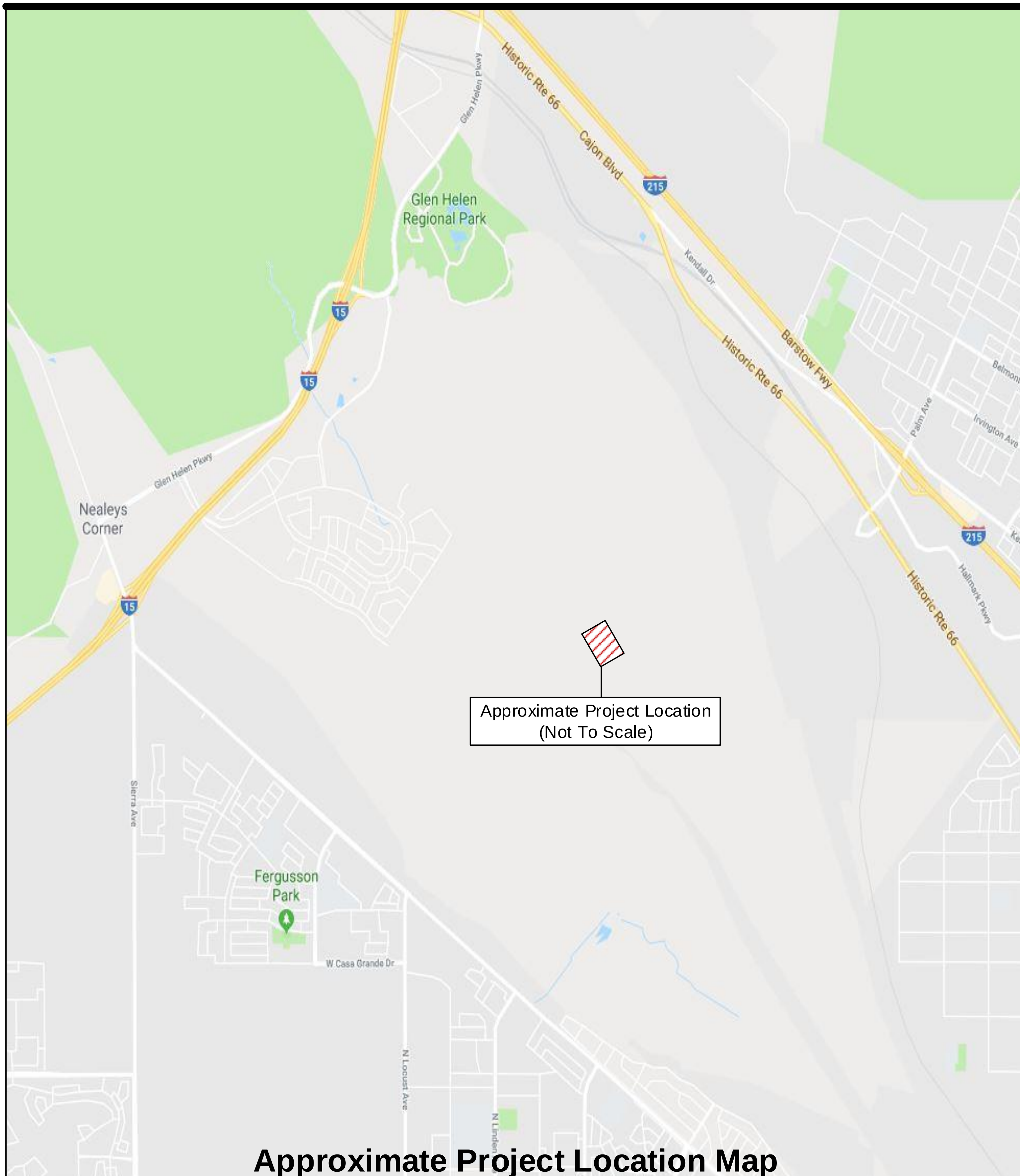
This report is prepared for the project described herein and is intended for use solely by San Bernardino County Special Districts Department. and their authorized agents for design purposes. It should not be used as a bidding document but may be made available to the potential contractors for information 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 intent of this project is to demolish the exisitng equipment and surperstructures located on the existing concrete pad to provide a suitable working pad and equipment foundation for a screw press and dewatered slude auger. A 10.5’ x 12.5’ bridging slab (thickness 8 inches) will be placed on top of the existing pad. The project also includes construction of a covered truck loading pad on the south side of the existing sludge screening pad. The covered structure will be placed on separate pier-founded substructure designed for full wind loading with trolley crane/monorail for handling large parts and polymer totes. The proposed covered structure will be approximately 47’ x 28’, covering the existing concrete pad structure and extending approximately 12 feet south of the existing pad. Drainage improvements are also proposed for the truck loadout pad to connect to existing piping.

3.0 SITE DESCRIPTIONS

The Lytle Creek North Water Recycling Facility is located south of the Institution Road in the City of San Bernardino, San Bernardino County, California. The facility is bounded by Institution Road to the north, vacant land and parking lot to the west and east and by existing structure to the south. The existing sludge screening facility is approximately 23’ x 31.5’ and contains a concrete pad of varying in thickness. The current surface of the proposed truck loadout area is covered with gravel. The site is flat with an elevation of approximately 1,643 feet above mean sea level (amsl). The coordinates for the project



Approximate Project Location Map

Project:	Screw Press at the Lytle Creek North Water Recycling Facility	
Location:	Institution Road	Project No.
	City of San Bernardino, San Bernardino County, California	18-81-226-01
For:	San Bernardino County Special Districts Department	

Converse Consultants

Figure No.
1

site are approximately 34.1749° north latitude and 117.3894° west longitude. Photograph No. 1 depicts the present site conditions.

Photograph No. 1: Present site conditions facing west.

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, groundwater data, and other information pertaining to the project area to assist in the evaluation of geologic hazards that may be present in the site.

4.2 Project Set-up

The project set-up consisted of the following tasks.

- Conducted a site reconnaissance and marked the boring location so drill rig access to the location is available.
- 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 to drill exploratory borings.

4.3 Subsurface Exploration

One exploratory boring (BH-01) was drilled on August 1, 2018 to investigate subsurface conditions. The boring was drilled to a maximum depth of 20.5 feet below existing ground surface (bgs).

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

4.4 Laboratory Testing

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

- *In*-situ moisture contents and dry densities (ASTM D2216/D7263)
- Expansion index (ASTM D4829)
- Soil corrosivity (California Tests 643, 422, and 417)
- Collapse potential (ASTM D4546)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

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

4.5 Analysis and Report Preparation

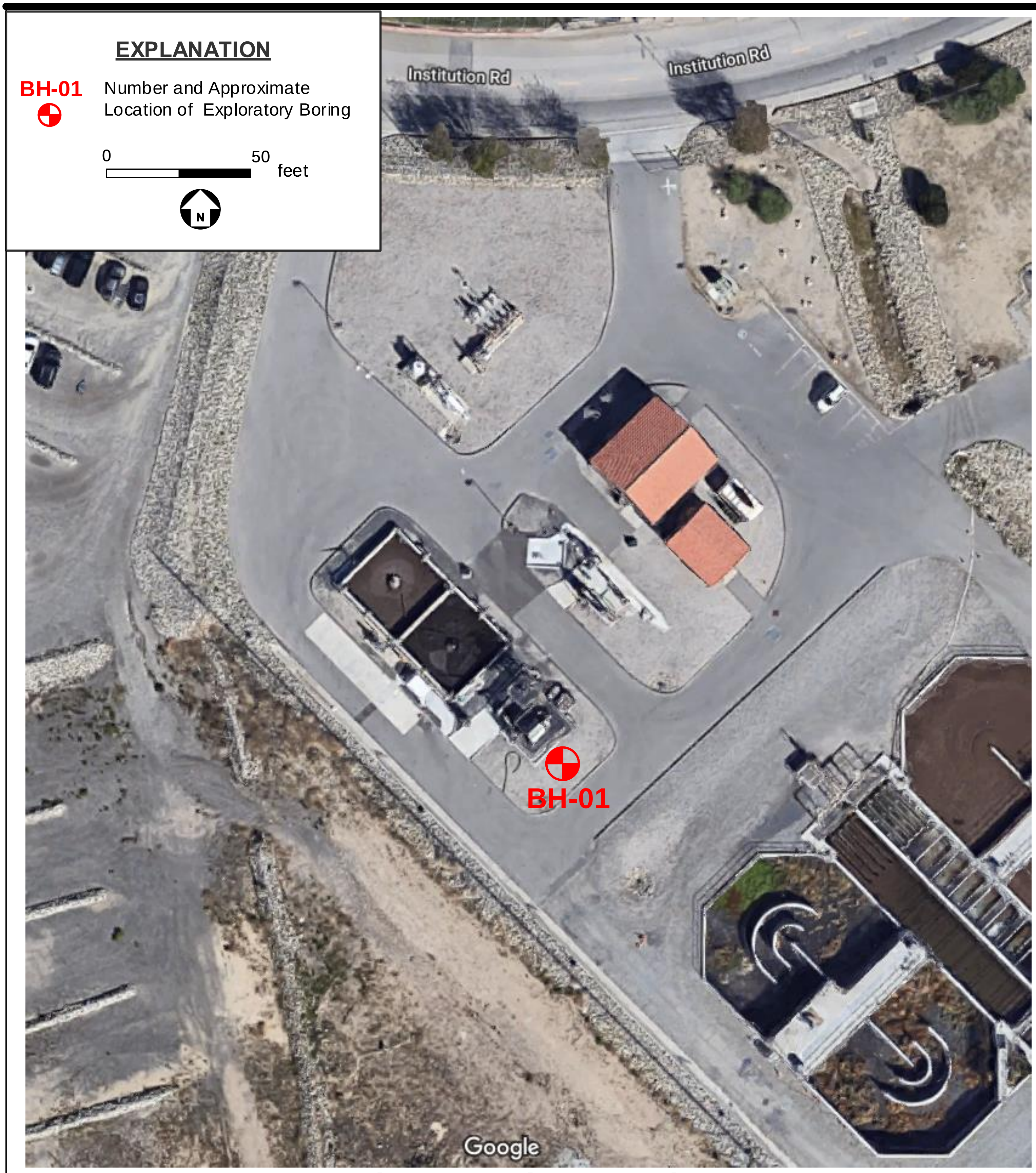
Data obtained from the field exploration and laboratory testing program were compiled and evaluated. Geotechnical analyses of the compiled data were performed and this report was prepared to present our findings, conclusions, and recommendations for the proposed project.

5.0 SUBSURFACE CONDITIONS

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

5.1 Subsurface Profile

Based on the exploratory boring and laboratory test results, the subsurface soils at the site primarily of alluvial soils consisting of gravelly silty sand and gravelly sand with trace



Approximate Boring Location Map

Project:	Screw Press at the Lytle Creek North Water Recycling Facility	
Location:	Institution Road	Project No.
	City of San Bernardino, San Bernardino County, California	18-81-226-01
For:	San Bernardino County Special Districts Department	

Converse Consultants

Figure No.
2

silt. Gravel up to 2.5 inches in largest dimension were observed in the boring. Though not encountered in the boring, cobbles may be present within the site. Based on hammer blow counts, the upper 10 feet soils are medium dense to dense. Relative compaction of the upper 10 feet soils varies from 83 to 90 percent.

For a detailed description of the subsurface materials encountered in the exploratory boring, see Drawing No. A-2, Log of Boring, in Appendix A, Field Exploration.

5.2 Groundwater

Groundwater was not encountered during the investigation to the maximum explored depth of 20.5 feet bgs. Regional conditions were reviewed to estimate expected groundwater depths in the vicinity of the proposed project. Data in the following table was found on the National Water Information System (USGS, 2017a).

Table No. 1, Summary of USGS Groundwater Depth Data

Site No.	Location	Groundwater Depth Range (ft. bgs)	Date Range
341003117232001	Approximately 2,700 feet South of the project site	48.7-424.7	1929-1989
340928117233501	Approximately 1.2 miles SW of the project site	274	1992

The Geotracker website (USGS, 2018) was also reviewed, but did not contain any data in the vicinity of the proposed site.

Based on current and historical data, groundwater is expected to be deeper than 48.7 feet bgs. Dewatering is not expected to be required during the construction. 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.

5.3 Excavatability

The subsurface soil materials at the site are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. <u>Excavation will likely be difficult due to the presence of gravel and possible cobbles.</u>

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 models should be done by an experienced earthwork contractor, and may require test excavations in representative areas.

5.4 Subsurface Variations

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

6.0 ENGINEERING GEOLOGY

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

6.1 Regional Geology

The project site is situated near the northern boundary of the Peninsular Ranges Geomorphic Province adjacent to the Traverse 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.

6.2 Local Geology

The project site is located adjacent to the active wash channels of Lytle Creek Wash and Cajon Wash at the mouth of Lytle Creek. The project site is located approximately 1,100 feet southwest of the San Jacinto Fault Zone (CGS, 1995) and adjacent to a concealed splay of the San Jacinto Fault (Morton and Miller, 2006). According to regional mapping (Dibblee and Minch, 2003; Morton and Miller, 2006) the site is underlain by young (late Holocene-age) alluvial fan deposits consisting of unconsolidated to to slightly consolidated sand, gravel, and boulders.

7.0 FAULTING AND SEISMICITY

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

7.1 Faulting

The project site is not located within a currently designated State of California or San Bernardino County Earthquake Fault Zone (CGS, 1995; San Bernardino County, 2010b). The project site is located adjacent to the San Jacinto Fault zone (Morton and Miller, 2006). The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

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

The following table contains a list of active and potentially active faults within one-hundred (100) kilometers of the subject site. The fault parameters and distances presented in the following table are based on the output from EQFAULT (Blake, 2000), revised in accordance with CGS fault parameters (Cao et. al., 2003).

Table No. 2, Seismic Characteristics of Nearby Active Faults

Fault Name	Approximate Distance (miles (km))	Moment Magnitude (Mw)
San Jacinto-San Bernardino	0.3 (0.5)	6.7
Cucamonga	3.2 (5.1)	6.9
San Andreas-San Bernardino	3.3 (5.3)	7.5
San Andreas-Southern	3.3 (5.3)	7.2
Cleghorn	7.9 (12.7)	6.5
North Frontal Fault Zone (West)	11.2 (18.1)	7.2
San Andreas-Mojave	12.5 (20.1)	7.4
San Jacinto-San Jacinto Valley	14.1 (22.7)	6.9
San Jose	18.0 (29.0)	6.4
Sierra Madre	19.1 (30.7)	7.2
Chino-Central Ave. (Elsinore)	23.2 (37.3)	6.7
Whittier	26.3 (42.4)	6.8
Elsinore-Glen Ivy	26.3 (42.4)	6.8
Clamshell-Sawpit	27.8 (44.7)	6.5
Elysian Park Thrust	31.1 (50.0)	6.7

Fault Name	Approximate Distance (miles (km))	Moment Magnitude (Mw)
Helendale-S. Lockhardt	31.3 (50.4)	7.3
North Frontal Fault Zone (East)	34.2 (55.0)	6.7
Raymond	35.5 (57.1)	6.5
Elsinore-Temecula	36.8 (59.3)	6.8
Pinto Mountain	39.0 (62.8)	7.2
Verdugo	40.2 (64.7)	6.9
San Jacinto-Anza	40.5 (65.1)	7.2
Compton Thrust	42.9 (69.0)	6.8
Lenwood-Lockhart-Old Woman Sprgs	43.9 (70.7)	7.5
Johnson Valley (Northern)	47.3 (76.2)	6.7
Hollywood	48.3 (77.7)	6.4
Newport-Inglewood (L.A.Basin)	49.3 (79.4)	7.1
Newport-Inglewood (Offshore)	50.3 (81.0)	7.1
San Gabriel	51.9 (83.5)	7.2
Landers	52.3 (84.1)	7.3
Sierra Madre (San Fernando)	52.6 (84.6)	6.7
Emerson So.-Copper Mtn.	54.6 (87.9)	7.0
Gravel Hills-Harper Lake	55.1 (88.6)	7.1
San Andreas-Coachella	55.4 (89.2)	7.2
Burnt Mtn.	56.4 (90.8)	6.5
Eureka Peak	57.0 (91.8)	6.4
Palos Verdes	58.3 (93.8)	7.3
Northridge (E. Oak Ridge)	58.7 (94.5)	7.0
Santa Monica	58.8 (94.7)	6.6
Elsinore-Julian	59.2 (95.3)	7.1
Calico-Hidalgo	60.1 (96.7)	7.3

7.2 Seismic Design Parameters

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

Table No. 3, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.1749 N, 117.3894 W
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	2.470g
Mapped 1-second Spectral Response Acceleration, S_1	1.132g

Seismic Parameters	
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	2.470g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.697g
Design Spectral Response Acceleration for short period S_{ds}	1.647g
Design Spectral Response Acceleration for 1-second period, S_{d1}	1.132g
Maximum Peak Ground Acceleration, PGA_M	0.949g

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, 1995; San Bernardino County, 2010b). The project site is located adjacent to the San Jacinto Fault zone (Morton and Miller, 2006). 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 development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

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

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

The project site is not located in an area evaluated as being susceptible to liquefaction by San Bernardino County (San Bernardino County, 2010b).

The current and historical high groundwater levels are deeper than 48.7 feet bgs. Due to the absence of shallow groundwater, the susceptibility of liquefaction at the project site is considered to be low.

Seismic Settlement: Seismically-induced settlement occurs in unsaturated, unconsolidated, granular sediments during ground shaking associated with earthquakes. Due to the presence of medium dense to dense subsurface soils, the risk of seismic settlement is considered to be low.

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

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over underlying materials which are liquefied due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Due to the low risk for liquefaction and 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. Seiching within the reservoirs onsite is possible during a major seismic event. The site is not considered at risk for damage from off-site seiching.

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.

- 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 105.9 to 126.0 pcf with moisture contents of 3 to 5 percent. Results are presented in the log of borings 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.
- Collapse Potential – The collapse potential of one relatively undisturbed sample from the upper 7 feet of soils was tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The test result showed collapse of 2.1 percent, indicating moderate collapse potential.
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship of a representative soil sample was tested in accordance with ASTM D1557 and is presented in Drawing No. B-1, *Moisture-Density Relationship Result*, in Appendix B, *Laboratory Testing Program*. The laboratory maximum dry density with was 139.0 pounds per cubic foot (pcf) and the optimum moisture content of 5.5 percent.
- Direct Shear – One direct shear test was performed on a sample remolded to 90% of the laboratory maximim dry density in accordance with ASTM Standard D3080. The result of the direct shear test is presented in Drawing No. B-2, *Direct Shear Test Result* in Appendix B, *Laboratory Testing Program*.

8.2 Chemical Testing - Corrosivity Evaluation

One representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of the test 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.*

- The pH measurement of the tested sample was 7.9.
- The sulfate content of the tested sample was 0.004 percent by weight.
- The chloride concentration of the tested sample was 34 ppm.
- The minimum electrical resistivity when saturated was 9,102 ohm-cm.

9.0 EARTHWORK RECOMMENDATIONS

Earthwork recommendations for the tank site and pipeline are presented in the following sections.

9.1 General

This section contains our general recommendations regarding earthwork and grading for the proposed screw press, truck loadout pad and piers. 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, if present, should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing structures or utilities.

Prior to placing the new pad on top of the existing pad, all existing equipment and superstructures should be demolished and removed from the site.

All debris, surface vegetation (if any), deleterious material, surficial soils containing gravel, roots and perishable materials (if any) and demolished 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 Remedial Grading

Truck loadout pad will be a flexible pavement and should be uniformly supported on compacted fill. In order to provide uniform support, pad areas should be overexcavated, scarified, and recompacted as follows.

Table No. 4, Overexcavation Depths

Structure	Minimum Excavation Depth
Truck loadout Pad	18 inches below pad

The overexcavation below the truck loadout pad should be uniform. The overexcavation should extend to at least 2 feet beyond the footprint of the truck loadout pad. 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 (if any). Consideration should be given to using slot cuts or other excavation methods which preserve lateral support during excavation operations near the existing facility.

9.3 Engineered Fill

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

- No particles larger than 3 inches in largest dimension.
- Rocks larger than one inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 30 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

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 fill should be scarified to a depth of 12 inches. The scarified soil should be moisture conditioned to within ± 3 percent of optimum moisture for granular soils or 0 to 2 percent above optimum for fine soils. The scarified soil should be recompacted to at least 90 percent of the laboratory maximum dry density prior to the placement of any fill.

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

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

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

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

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

9.5 Shrinkage and Subsidence

The volume of excavated and recompacted soils will decrease as a result of grading. The shrinkage would depend on, among other factors, the depth of cut and/or fill, the grading method and equipment utilized and removal of oversized (greater than 6 inches) materials. For preliminary estimation, shrinkage factors at the site may be taken as presented below.

- The shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) for the upper 5 feet of soils is estimated to be 1.5 percent.

- Subsidence (defined as the settlement of native materials from the equipment load applied during grading) would depend on the construction methods including type of equipment utilized. For estimation purposes, ground subsidence may be taken as 0.1 feet.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

9.6 Site Drainage

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

10.0 DESIGN RECOMMENDATIONS

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 Bearing Capacity of Soils

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

The net allowable bearing capacity of soils in truck loadingout pad zone can be taken as 2,200 psf considering the pad will be placed at a depth of minimum 2 feet below existing ground surface.

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 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.

10.2.1 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 without surcharge for the project site are presented in the following table.

Table No. 5, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure (psf)
Active earth conditions (wall is free to deflect at least 0.001 radian)	40
At-rest (wall is restrained)	60

These pressures assume a level ground surface around the structure for a distance greater than the structure height, no surcharge and no hydrostatic pressure. If water pressure is allowed to build up behind the walls, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the walls.

10.2.2 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 220 psf per foot of depth may be used for the sides of 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 2,200 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.

10.3 Pier Foundation Recommendations

Truck loadout covered structure will be supported on drilled pier foundations deriving their support primarily through skin friction. The piers may be designed for compression using an allowable skin friction value of 180 psf per foot. Allowable skin friction has been determined based on 6-inch diameter drilled pier. This value will be increased with the increment of drilled pier diameter.

This value may be increased by 33 percent for transient wind and seismic forces. For pier design in tension, 50 percent of the recommended allowable skin friction values in compression may be used. For design purpose, the upper 2 feet of the soils should be neglected in determining the skin friction. The equivalent lateral earth pressure equal to 220 psf per foot of depth may be used for the design.

10.4 Asphalt Concrete Pavement

Based on the soil type and experience on similar type of projects, an R-value of 30 was assumed for the design of truck loadout pad. For pavement design, we have utilized an R-value of 30 and design Traffic Index of 10.

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

Table No. 6, Recommended Preliminary Pavement Sections

R-value 30	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
	10	6.0	12.0	14.0

At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design.

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

Base materials should conform with Section 200-2.2,"*Crushed Aggregate Base*," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works

Standards, 2015) and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302.5 of the SSPWC.

10.5 Soil Corrosivity

One representative soil sample was evaluated for corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program* and design recommendations pertaining to soil corrosivity are presented below.

The sulfate content of the sampled soil 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 locations and the results of chloride testing of the site soils, we do not anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

The measured value of the minimum electrical resistivity of the sample when saturated was 9,102 Ohm-cm. This indicates that the soil tested is moderately corrosive to ferrous metals in contact with the soil (Romanoff, 1957). <u>Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures , if necessary, for any ferrous metals in contact with the site and site soils.</u>

11.0 DRILLED PIER FOUNDATION CONSTRUCTION AND INSTALLATION RECOMMENDATIONS

Drilled piers should be constructed in accordance with the Standard Specifications for Public Works Construction section *305-Pile Driving and Timber Construction* (Green Book, 2015). Boring piers should be reasonably clean and free of loose soil prior to installing the concrete piers. The annular space around the piers base should be filled with grout.

It should be the responsibility of the contractor to select proper construction equipment and method to correctly install the piers based on his own interpretation of the information presented in this report.

Groundwater was not encountered in the exploratory boring to a depth of 20.5 feet below existing ground surface. Caving will occur in loose sandy soils. Casing, or other methods approved by the project geotechnical consultant, should be used to support the sides of the pier excavation.

Casing should be used at the discretion of the contractor. Casing should be advanced as drilling proceeds by drilling with a flight or bucket auger smaller in diameter than the inside of the casing. Occasional hammering may be required to advance the casing within the excavation. The casing, when used, should not be left in place as the piers designs are based on skin friction only. Casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. The contractor should have equipment on-site with sufficient pulling capacity to pull the casing at the proper time. The casing should have outside diameter not less than the specified diameter of the pier.

The bottoms of the excavations should be cleaned of any loose cuttings before placing concrete. All applicable state and federal OSHA safety regulations must be satisfied during construction.

Drilled piers installation shall be performed under continuous observation by the project geotechnical consultant to confirm that the subsurface soils are similar to the soils encountered during our field investigation, which have formed the basis of our piers design recommendations. The contractor shall provide access and necessary facilities, including droplights, at his expense, to accommodate piers observations.

Drilled piers installation shall be performed such that compliance with all safety rules and requirements is achieved.

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 SBCSDD. 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 of one soil boring. During the site reconnaissance, the surface conditions were noted, and the approximate location of the test boring was established using existing site and boundary features as reference. The location should be considered accurate only to the degree implied by the method used.

One exploratory boring (BH-01) was drilled on August 1, 2018 to investigate subsurface conditions. The boring was drilled to a maximum depth of 20.5 feet below existing ground surface (bgs).

The boring was 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 a depth of 20.5 feet bgs using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

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

Following the completion of logging and sampling, the boring was backfilled with soil cuttings and tamped. The surface may settle over time, if construction is delayed. Therefore, we recommend the owner monitor the boring location and backfill any depressions that might occur, or provide protection around the boring location to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring log, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For log of boring, see Drawing No. A-2, *Log of Boring*.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST
Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method

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

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

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

STRENGTH

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

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

Screw Press at Lytle Creek North Water Recycling Facility
Institution Road
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No. **18-81-226-01**

Drawing No. **A-1**

Log of Boring No. BH-01

Dates Drilled: 8/1/2018 Logged by: Michael Maldonado Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 1643 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		**POORLY GRADED GRAVEL**						
		ALLUVIUM **GRAVELLY SILTY SAND (SM):** fine to coarse-grained, some gravel up to 2.5" in largest dimension, brown.		X				ei, ca, er max
			X	X	19/19/13	5	126	ds
5		**GRAVELLY SAND (SP):** fine to coarse-grained, gravel up to 2.5" in largest dimension, trace silt, brown.	X		7/8/13	3	116	col
			X		30/30/40	3	122	
10			X	X	18/24/43	3	106	
15			X		38/50-5"			NR
20			X		50-5"	1	110	
		End of boring at 20.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped on 8/1/2018.						

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 Log of Boring, 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 D7263 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Log of Boring in Appendix A, Field Exploration.

Expansion Index

One representative bulk sample was tested to evaluate the expansion potential. The test was conducted in accordance with ASTM Standard D4829. The test result is presented in the following table.

Table No. B-1, Expansion Index Test Result

Boring No./ Location	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-01	1-5	Gravelly Silty Sand (SM)	0	Very Low

Soil Corrosivity Tests

One representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these 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-2, 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	1-5	7.9	0.004	34	9,102

Collapse Tests

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, one collapse test was performed in accordance with the ASTM Standard D4546 laboratory procedure. The sample was loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test result is presented in the following table.

Table No. B-3, Collapse Test Result

Boring No./ Location	Depth (feet)	Soil Classification	Percent Swell + Percent Collapse -	Collapse Potential
BH-01	5.0-6.5	Gravelly Sand (SP)	-2.1	Moderate

Maximum Density and Optimum Moisture Content Tests

Laboratory maximum dry density-optimum moisture content relationship test was performed on a representative bulk sample. The test was conducted in accordance with the ASTM Standard D1557 test method. The test result is presented in Drawing No. B-1, *Moisture-Density Relationship Result,* and are summarized in the following table.

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

Boring No./ Location	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-01	1-5	Gravelly Silty Sand (SM), Brown	5.5	139.0

Direct Shear Tests

One direct shear test was performed on a sample remolded to 90 percent of the laboratory maximum dry density in accordance with ASTM D3080. The test was conducted under soaked moisture condition. For the test, 3 samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing No. B-2*, Direct Shear Test Result,* and the following table.

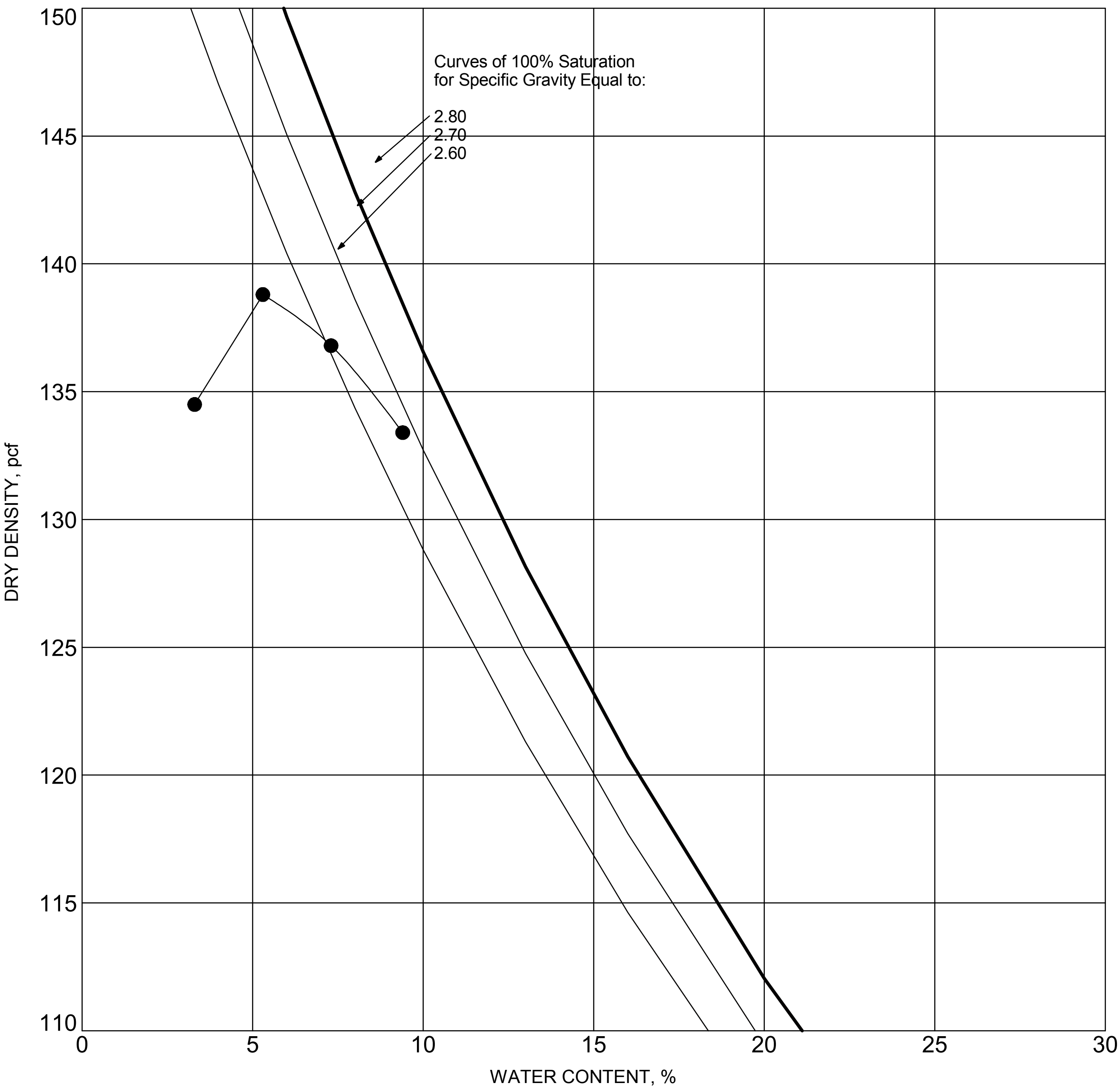
Table No. B-5, Summary of Direct Shear Test Result

Boring No./ Location	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
*BH-01	2.5-4.0	Gravelly Silty Sand (SM)	29	120

(* Remolded to 90% of the laboratory maximum dry density)

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.



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**1-5**	**GRAVELLY SILTY SAND (SM), BROWN**	**D1557- C**	**5.5**	**139.0**

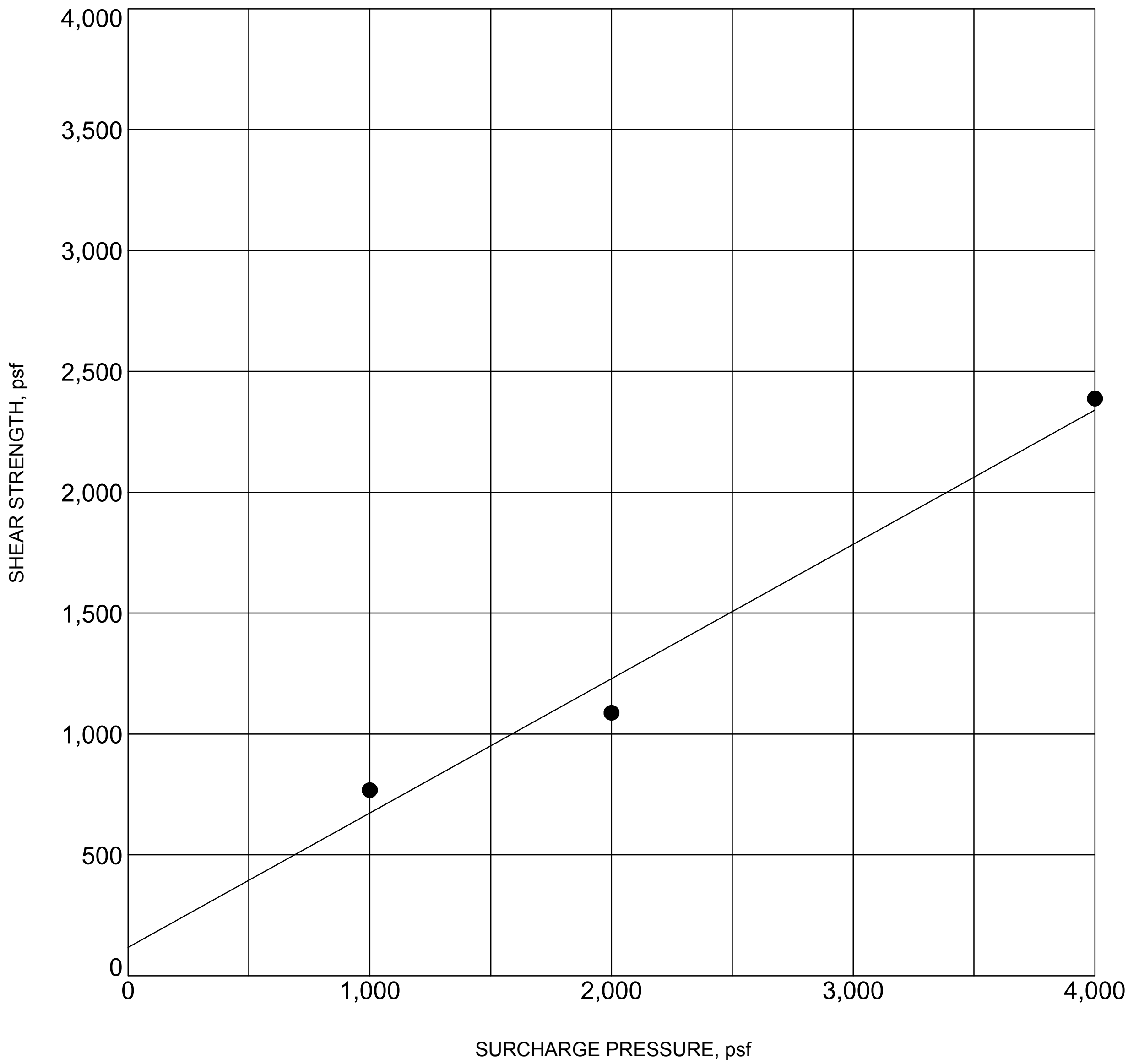
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Screw Press at Lytle Creek North Water Recycling Facility
Institution Road
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No. **18-81-226-01**

Drawing No. **B-1**



BORING NO.	:	**BH-01**	DEPTH (ft)	:	**2.5-4.0**
DESCRIPTION	:	**GRAVELLY SILTY SAND (SM)**			
COHESION (psf)	:	**120**	FRICTION ANGLE (degrees):		**29**
MOISTURE CONTENT (%)	:	**3.0**	DRY DENSITY (pcf)	:	**116.0**

NOTE: Ultimate Strength. Remolded to 90% of the laboratory maximum dry density

DIRECT SHEAR TEST RESULTS

Converse Consultants

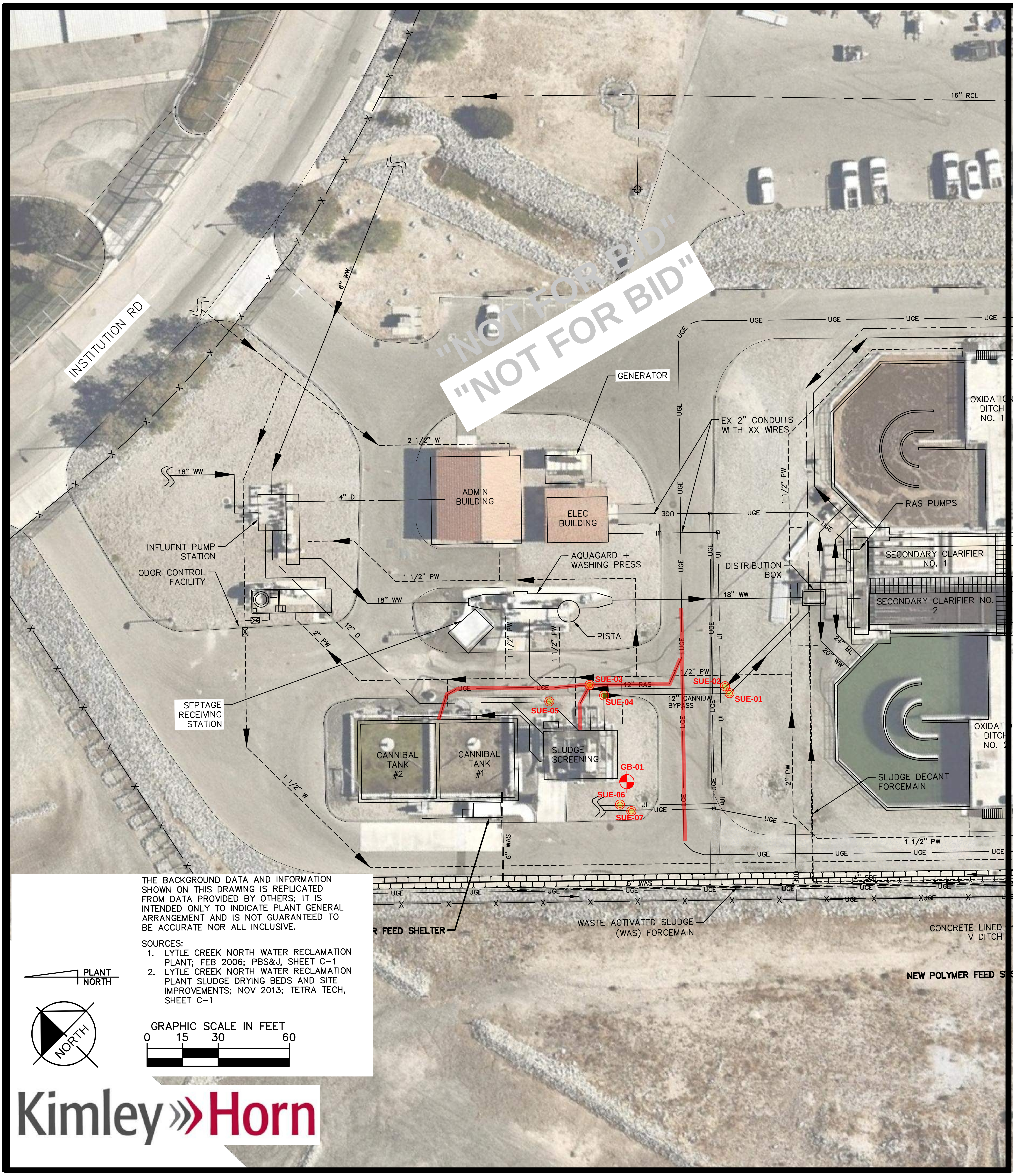
Screw Press at Lytle Creek North Water Recycling Facility
Institution Road
City of San Bernardino, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No. **18-81-226-01**

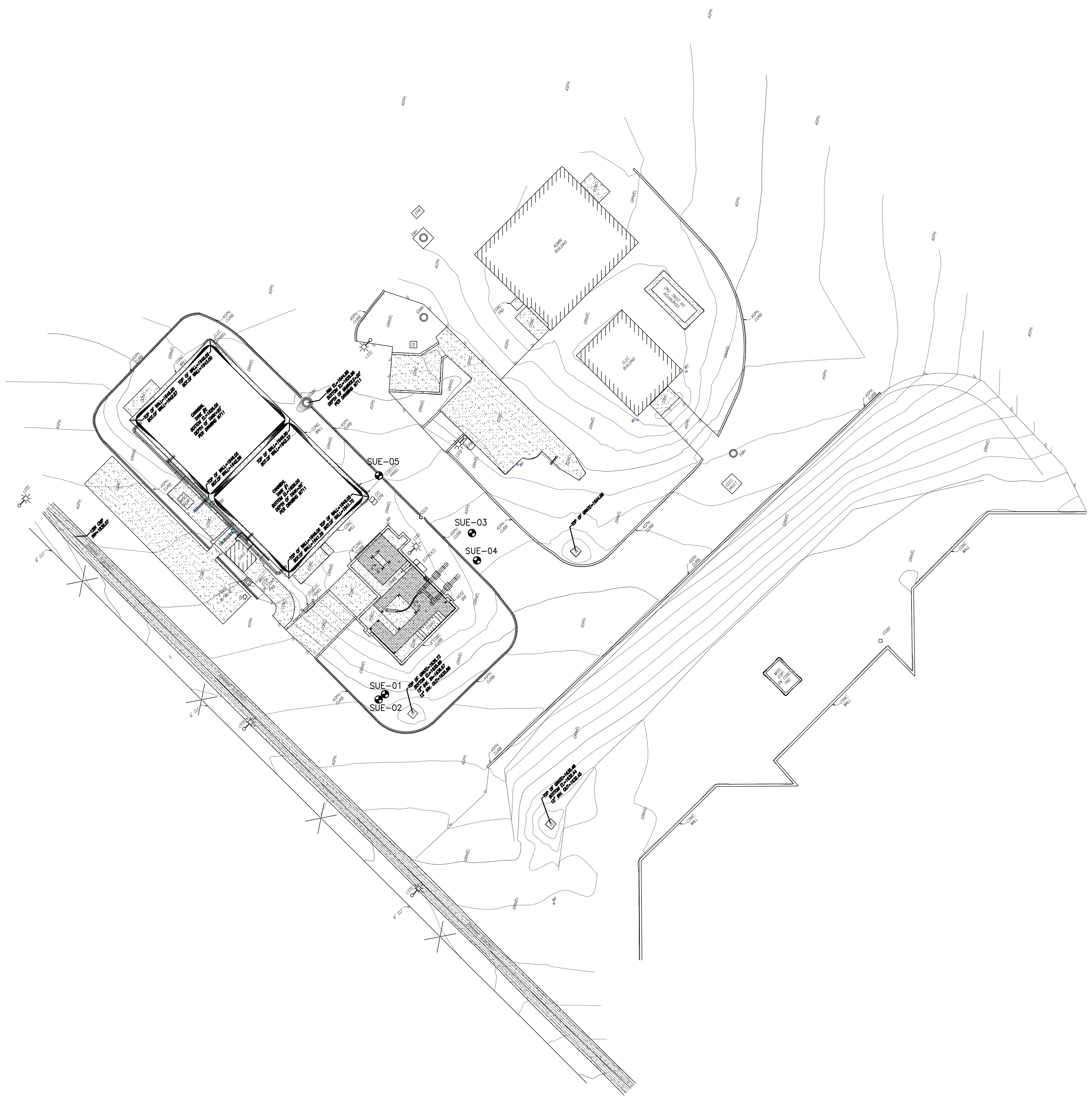
Drawing No. **B-2**

APPENDIX "B"

POTHOLE DATA



"NOT FOR BID"
"NOT FOR BID"
INSTITUTION RD
16" RCL
6" WW
18" WW
4" D
2 1/2" W
GENERATOR
ADMIN BUILDING
ELEC BUILDING
EX 2" CONDUITS WITH XX WIRES
UGE
UI
1 1/2" PW
OXIDATION DITCH NO. 1
RAS PUMPS
SECONDARY CLARIFIER NO. 1
SECONDARY CLARIFIER NO. 2
DISTRIBUTION BOX
INFLUENT PUMP STATION
ODOR CONTROL FACILITY
AQUAGUARD + WASHING PRESS
18" WW
PISTA
12" D
2" PW
24" ML
20" WW
SEPTAGE RECEIVING STATION
12" RAS
12" CANNIBAL BYPASS
SUE-01
SUE-02
SUE-03
SUE-04
SUE-05
SUE-06
SUE-07
GB-01
CANNIBAL TANK #2
CANNIBAL TANK #1
SLUDGE SCREENING
OXIDATION DITCH NO. 2
SLUDGE DECANT FORCEMAIN
1 1/2" W
6" WAS
2" PW
6" WAS
FEED SHELTER
WASTE ACTIVATED SLUDGE (WAS) FORCEMAIN
CONCRETE LINED V DITCH
NEW POLYMER FEED
THE BACKGROUND DATA AND INFORMATION SHOWN ON THIS DRAWING IS REPLICATED FROM DATA PROVIDED BY OTHERS; IT IS INTENDED ONLY TO INDICATE PLANT GENERAL ARRANGEMENT AND IS NOT GUARANTEED TO BE ACCURATE NOR ALL INCLUSIVE.
SOURCES:
1. LYTLE CREEK NORTH WATER RECLAMATION PLANT; FEB 2006; PBS&J, SHEET C-1
2. LYTLE CREEK NORTH WATER RECLAMATION PLANT SLUDGE DRYING BEDS AND SITE IMPROVEMENTS; NOV 2013; TETRA TECH, SHEET C-1
PLANT NORTH
NORTH
GRAPHIC SCALE IN FEET
0 15 30 60
Kimley»Horn



POTHOLE TABLE

POTHOLE NUMBER	SIZE AND TYPE	DEPTH TO TOP OF PIPE (FT)	FINISHED SURFACE ELEVATION	TOP OF PIPE ELEVATION	POTHOLE DEPTH (FT)	POTHOLE WIDTH (FT)
SUE-01	NO UTILITY FOUND	N/A	1639.8'	N/A	6.0'	2.0'
SUE-02	NO UTILITY FOUND	N/A	1639.7'	N/A	6.0'	2.0'
SUE-03	2" PVC RECLAIMED WATER	3.0'	1640.8'	1637.8'	3.0'	2.0'
SUE-04	12" STEEL SEWER SYSTEM	2.75'	1640.6'	1637.9'	2.75'	2.0'
SUE-05	6" PVC GAS	2.92'	1642.1'	1639.2'	2.92'	2.0'

PH1
= POT HOLE LOCATION

NOTES:

1. SUE-05 WAS DUG IN DIRT AND WAS MEASURED FROM THE TOP OF CURB TO THE TOP OF PIPE