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

UPDATED GEOTECHNICAL INVESTIGATION & WATER INFILTRATION TEST REPORT

APPROXIMATELY 20.60-ACRE RESIDENTIAL DEVELOPMENT
SOUTHEAST CORNER OF HOPLAND STREET AND CAHUENGA ROAD
CITY OF VICTORVILLE, SAN BERNARDINO COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 19-81-173-01

Prepared For:
LANSING COMPANIES
12671 High Bluff Drive, Suite 150
San Diego, CA 92130

Presented By:
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2021 Rancho Drive, Suite 1
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909-796-0544

July 16, 2019

Converse Consultants

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

July 16, 2019

Mr. Casey Malone
Project Manager
Lansing Companies
12671 High Bluff Drive, Suite 150
San Diego, CA 92130

Subject: **UPDATED GEOTECHNICAL INVESTIGATION AND WATER INFILTRATION TEST REPORT**
Approximately 20.60-Acre Residential Development
Southeast Corner of Hopland Street and Cahuenga Road
City of Victorville, San Bernardino County, California
Converse Project No. 19-81-173-01

Dear Mr. Malone:

Converse Consultants (Converse) has prepared this updated geotechnical investigation and water infiltration test report to present the findings, conclusions and recommendations for the approximately 20.60-Acre Residential Development project located on the southeast corner of Hopland Street and Cahuenga Road in the city of Victorville, San Bernardino County, California. This report is prepared in accordance with our proposal dated May 14, 2019 and your General Consultant Agreement dated May 16, 2019.

Converse Consultants prepared a geotechnical investigation report (05-81-351-01) for the subject site dated January 27, 2006 for Victory Ridge Estate Homes, LLC (Converse, 2006). A portion of the site was developed. This report includes design and construction recommendations for development of the remaining site.

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 continued service to Lansing Companies. If you should have any questions, please contact the undersigned at 909-796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, PhD, PE, GE
Regional Manager/Principal Engineer

Dist.:   3/Addressee

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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.

Zahangir Alam, PhD, EIT
Senior Staff Engineer

James Burnham, PG
Project Geologist

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

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EXECUTIVE SUMMARY

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

▪ The proposed 20.60-acre residential development site is located on the southeast corner of Hopland Street and Cahuenga Road in the City of Victorville, San Bernardino County, California. The site is irregularly shaped and is roughly bounded on the east by residential developments, Carmelia Drive, and vacant land; on the west by Cahuenga Road; on the north by residential developments and Hopland Street; and on the south by Tawney Ridge Lane. The site is presently vacant. The topography of the site is irregular, but generally trends downwards from approximately 2,910 feet above mean sea level (AMSL) along the eastern-most boundary to approximately 2,875 feet AMSL along the western-most boundary. The landscape is relatively flat and clear of major vegetation.

▪ It was planned to build 129 single-family, one- and two-story homes supported by conventional continuous and/or isolated footing foundations with slab-on-grade. It is our understanding that the development included driveways, in-tract streets with curbs and gutters, sidewalks, landscaped areas, and under- and above-ground utilities. We understand approximately 10-acre of the original 30-acre has been developed with 59 single-family homes, above and below ground utilities and interior streets. We are not aware when the site was graded and who provided observation and testing during grading and post-grading. The remaining 20.60-acre site will now be developed for 70 single-family homes supported by conventional continuous and/or isolated footing foundations with slab-on-grade. The project also includes streets, driveways, curb and gutter, sidewalks, landscape areas and above and underground utilities. A detention basin approximately between 6.5 to 8 feet deep is planned at the northeast corner of the site.

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

▪ For the previous investigation performed by Converse, a total of seven exploratory borings (BH-1 to BH-7) were drilled on December 7, 2005 across the project site, to depths of 16.5 to 51.5 feet below ground surface (bgs).

▪ Additionally, two exploratory borings (BH-8 and BH-9) were drilled on June 3, 2019 to investigate subsurface conditions at the project site. The borings were drilled to depths of 15.8 and 16.4 feet below existing ground surface (bgs). Two exploratory percolation test holes (PT-01 and PT-02) were drilled on June 3, 2019 to perform

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percolation testing. Both percolation test borings were drilled to approximately 8.0 feet below the existing ground surface (bgs). The percolation test holes were re-drilled to 10 feet bgs on July 12, 2019. Logs of borings from the previous and present investigation are included in Appendix A, *Field Exploration*.

- The subsurface soil at the site consists primarily mixture of silt, sand, and gravel. Gravel up to 2 inches in largest dimension was encountered in most of the borings.

- Groundwater was not encountered during our current (2019) or previous (2006) field investigation to the maximum explored depths of 16.4 and 51.5 feet bgs, respectively. Current groundwater is expected to be deeper than 16.4 feet bgs. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the vicinity.

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

- Due to the absence of shallow groundwater, the project site is not considered susceptible to liquefaction.

- The risk to the site from lateral spreading, landsliding, seiches, tsunamis, and earthquake-induced flooding are considered to be low.

- The expansion index (EI) of soil samples from the upper 10 feet varied from 0 to 43, corresponding to very low to low expansion potential. The collapse potentials of the upper 10 feet soils were between 0.25 to 3.03 (including consolidation test) percent, indicating slight to moderate collapse potential.

- 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 were 876 and 4,046 ohm-cm for the site. This indicates that the soils tested are mildly corrosive to severely corrosive to ferrous metals in contact with the soil. <u>Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.</u>

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- 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 utilities and structure (if any).

- Based on our subsurface exploration, we anticipate that the site soils will be excavatable with conventional heavy-duty earthmoving equipment. Difficult excavation may be encountered in areas of high concentration of granular materials.

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

- About five feet of alluvial soils should be removed and replaced with compacted fill, prior to placing additional compacted fill.

- For building pads, deeper excavation may be required below finish grade in cut areas. If less than five feet is removed from original ground (og), excavation should continue to provide a minimum of two feet of compacted fill below bottom of footings. If more than five feet is removed, the bottom surface should be evaluated for suitability by the geotechnical consultant. All over-excavations should extend at least five feet or equal to the depth of over-excavation, whichever is greater, outside the building footprint.

- The cut portion of transition lots (and if necessary, the fill portion) should be excavated to a depth to provide a minimum of two feet of compacted fill beneath the entire pads.

- As a minimum, the upper three feet of surficial soils from all areas receiving asphalt concrete or Portland concrete paving, including driveways, sidewalks, street areas, curbs and gutters and other flatwork should be excavated, removed if necessary, and/or replaced as compacted fill. Such over-excavation should extend at least two feet beyond the pavement area edges.

- As a minimum, the upper three feet of surficial soils within two feet of either side of retaining/perimeter walls less than six feet in height, should be excavated, removed if necessary, and/or processed and replaced as compacted fill. The depth of the structural fill under retaining/perimeter wall footings should be at least two feet or equal to footing width, whichever is greater.

- Fill soils should be placed on scarified and recompacted excavation bottoms, moisture conditioned, and compacted to at least 90 percent of the laboratory maximum dry density. At least the upper 12 inches of fill beneath pavement intended to support

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vehicle loads should be compacted to at least 95 percent of the laboratory maximum dry density.

- Residential one- or two-story wood-frame, lightly loaded structures may be supported on conventional continuous (strip) and/or isolated (spread) footings. Interior and exterior footings should be placed at least 12 inches and 18 inches, respectively, below lowest adjacent soil grade. Width of the continuous and isolated footings for one-story buildings should be at least 12 inches and 18 inches, respectively. Width of the continuous and isolated footings for two-story buildings should be at least 18 inches and 24 inches, respectively. Footings placed at a depth of 12 inches and 18 inches below lowest adjacent grade may be designed based on an allowable net bearing capacity of 2,000 pounds per square foot (psf).

- The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be one inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 40 feet.

- Based on the observed high blow counts below 5 feet bgs in all borings and over-excavation recommendations, we anticipate the site will likely have negligible seismic settlement. For the design purpose, seismic settlement may be taken as 1 inch or less and the differential settlement may be taken as half of the total seismic settlement.

- The recommended infiltration rate is 0.17 inches/hour at 8 feet bgs or 1.01 inches per hour at 10 feet bgs at the location of the infiltration basin.

- 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 are provided in the text of this report.

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

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APPENDICES

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1.0   INTRODUCTION

This updated report contains the findings of the geotechnical investigation and percolation tests performed by Converse for the proposed residential development within a 20.60-acre site located on the southeast corner of Hopland Street and Cahuenga Road in the city of Victorville, San Bernardino County, California. The project location is shown in Figure No. 1, *Approximate Project Location Map.*

Converse Consultants investigated the site on December 7, 2005 by drilling seven exploratory borings ranging in depths from 16.5 to 51.5 feet below existing ground surface (bgs). A geotechnical investigation report was prepared for Victory Ridge Estate Homes, LLC (Converse, 2006).

The purpose of this investigation was to evaluate the current nature and engineering properties of the subsurface soils and groundwater conditions, and to provide updated geotechnical recommendations for the proposed residential development.

This report is written for the project described herein and is intended for use solely by Lansing Companies and their design team.   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 BACKGROUND AND DESCRIPTION

The original approximately 30-acre site is located at the southeast corner of Hopland Street and Cahuenga Road in the City of Victorville, San Bernardino County, California.

It was planned to build 129 single-family, one- and two-story homes supported by conventional continuous and/or isolated footing foundations with slab-on-grade. It is our understanding that the development included driveways, in-tract streets with curbs and gutters, sidewalks, landscaped areas, and under- and above-ground utilities.

We understand approximately 10-acre of the original 30-acre has been developed with 59 single-family homes, above and below ground utilities and interior streets. We are not aware when the site was graded and who provided observation and testing during grading and post-grading.

The remaining 20.60-acre site will now be developed for 70 single-family homes supported by conventional continuous and/or isolated footing foundations with slab-on-grade. The project also includes streets, driveways, curb and gutter, sidewalks, landscape areas and above and underground utilities. A detention basin approximately between 6.5 to 8 feet deep is planned at the northeast corner of the site.

Approximate Project Location
(not to scale)

Approximate Project Location Map

Project:	Approximately 20.60-Acre Residential Development
Location:	City of Victorville, San Bernardino County, California
For:	Lansing Companies

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19-81-173-01

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FIGURE NO.

1

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Rough grading plans have not been prepared or reviewed at the time of this report. Based on our experience with similar projects, site development may include slopes and earth retaining walls (perimeter walls) less than six feet in height. These walls will be founded on conventional continuous footings.

3.0   SITE DESCRIPTION

The proposed 20.60-acre residential development site is irregularly shaped and is roughly bounded on the east by residential developments, Carmelia Drive, and vacant land; on the west by Cahuenga Road; on the north by residential developments and Hopland Street; and on the south by Tawney Ridge Lane. The site is presently vacant.

The topography of the site is irregular, but generally trends downwards from approximately 2,910 feet above mean sea level (AMSL) along the eastern-most boundary to approximately 2,875 feet AMSL along the western-most boundary. The landscape is relatively flat and clear of major vegetation. Few large boulders are randomly dispersed throughout the site and a large depression and gently sloping mound of soil is located roughly in the center of the site in the vicinity of boring BH-9. Short piles of undocumented fill soil are also present throughout the western portion of the site. The present site conditions are shown in Photograph 1 below.

Photograph No. 1, Present site conditions near center-west boundary, facing northwest.

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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 15, *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.

- Conducted a field reconnaissance and marked the boring locations such that the drill rig access to all locations was 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

For the previous investigation performed by Converse, a total of seven exploratory borings (BH-1 to BH-7) were drilled on December 7, 2005 across the project site, to depths of 16.5 to 51.5 feet below ground surface (bgs).

Additionally, two exploratory borings (BH-8 and BH-9) were drilled on June 3, 2019 to investigate subsurface conditions at the project site. The borings were drilled to depths of 15.8 and 16.4 feet below existing ground surface (bgs).

Two exploratory percolation test holes (PT-01 and PT-02) were drilled on June 3, 2019 to perform percolation testing. Both percolation test borings were drilled to approximately 8.0 feet below the existing ground surface (bgs).

Approximate boring and percolation testing locations are indicated in Figure No. 2a, *Approximate Boring and Percolation Test Locations Map*. Previous (2006) approximate boring locations are also attached after Figure No. 2a. For a description of the field exploration and sampling program, see Appendix A, *Field Exploration*.

EXPLANATION

⊕ BH-9   Number and Approximate Location of Exploratory Boring

⊕ PT-2   Number and Approximate Location of Percolation Test Hole

Approximate Boring and Percolation Test Locations Map

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Figure No.
2a

APPROXIMATE BORING LOCATION MAP

EXPLANATION

BH - 1 Number and Approximate Boring Location

APRROXIMATELY 30 — ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Converse Consultants

Scale	NTS	Project No.
Prepared By		05-81-351-01
	KQ	Figure No.
Checked By	BAS	2
Approved By	RJR	

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4.4   Laboratory Testing

Representative soil samples of the project site 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)
- R-value (California Test Method 301)
- Soil corrosivity (California Tests 643, 422, and 417)
- Collapse Potential (ASTM Standard D4546)
- Grain size distribution (ASTM D6913)
- 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 was 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   SITE CONDITIONS

A general description of the subsurface conditions and various materials encountered at the site during our field exploration is contained in this section.

5.1   Subsurface Profile

Based on the exploratory borings, test pits, and laboratory test results, the subsurface soil at the site consists primarily mixture of silt, sand, and gravel. Gravel up to 2 inches in largest dimension was encountered in most of the borings.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-8 (2006) and A-9 through A-12 (2019), Logs of Borings, in Appendix A, Field Exploration.

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5.2 Groundwater

Groundwater was not encountered during our current (2019) or previous (2006) field investigation to the maximum explored depths of 16.4 and 51.5 feet bgs, respectively. The GeoTracker database (SWRCB, 2019) was reviewed for groundwater data from sites within an approximately 1.0-mile radius of both the proposed development. Data in the following table was found on the National Water Information System (USGS, 2019a).

Table No. 1, Summary of USGS Groundwater Depth Data

Alignment No.	Location	Groundwater Depth Range (ft. bgs)	Date Range
343239117194801	West side of Torrance Ln. cross of Village Dr.	137.1-161.9	1992-2014
343149117205301	Approximately 600ft. West of El Evado Rd. between Mojave Dr. and Fontaine Way	143.1	1917
343145117204701	Approximately 15ft. East of El Evado Rd. between Mojave Dr. and Dumosa Drive	211-214	2006-2010
343146117194401	Approximately 15ft. East of El Evado Rd. between Mojave Dr. and Dumosa Drive	198.1-221	2004-2014

Based on available data, the historical high groundwater level reported at wells within approximately one mile of the site was approximately 137.1 feet bgs. Current groundwater is expected to be deeper than 16.4 feet bgs. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the vicinity.

5.3 Excavatability

The subsurface materials at the site are expected to be excavatable by conventional heavy-duty earth moving equipment. Difficult excavation may be encountered in areas of high concentration of granular materials.

The phrase "conventional heavy-duty excavation equipment" is intended to include commonly used equipment such as excavators, scrapers, 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.

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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 between or beyond the boring locations.

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 located in the Mojave Desert Geomorphic Province of Southern California. The Mojave Desert is a broad interior region of isolated mountain ranges separated by wide desert plains. The area is roughly triangular shaped and bounded by the Garlock Fault on the north, the San Andreas Fault on the southwest, and the Colorado River on the east. The drainages are primarily closed and terminate in playas within the valley floors.

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

Extension of the region has resulted in exposure of basement rocks dating to the Precambrian age, deposition of young Holocene-aged sedimentary basins, and eruptions of volcanic units.

6.2   Site Geology

Loose to well-consolidated sand, silt, and pebble-cobble gravel. (Hernandez et al., 2008).

6.3   Flooding

Review of National Flood Insurance Rate Maps indicates that the project site is within a Flood Hazard Zone "X". The Zone "X" is designated as "Areas determined to be outside the 500-year floodplain (FEMA, 2008).

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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. Review of recent seismological and geophysical publications indicates that the seismic hazard for the project is high.

The project site is not located within a currently mapped State of California Earthquake Fault Zone for surface fault rupture. Table No. 2, *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. 2, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
North Frontal (West)	19.35	reverse	50	1	7.20
Helendale-So Lockhart	20.01	strike slip	114	0.6	7.40
Cleghorn	27.36	strike slip	25	3	6.80
S. San Andreas	31.19	strike slip	548	n/a	8.18
San Jacinto	34.44	strike slip	241	n/a	7.88
Cucamonga	41.09	thrust	28	5	6.70
Lenwood-Lockhart-Old Woman Springs	42.71	strike slip	145	0.9	7.50

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

7.2 CBC Seismic Design Parameters

Seismic parameters based on the 2016 California Building Code (CBSC, 2016) are provided in the following table were determined using the Seismic Design Maps application (OSHPD, 2019) and are presented in the following table.

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Table No. 3, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.5409 N, 117.3393 W
Site Class	D
Risk Category	III
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.424g
Mapped 1-second Spectral Response Acceleration, S_1	0.563g
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}	1.424g
MCE 1-second period Spectral Response Acceleration, SM_1	0.845g
Design Spectral Response Acceleration for short period S_{DS}	0.950g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.563g
Maximum Peak Ground Acceleration, PGA_M	0.500g

7.3   Secondary Effects of Seismic Activity

In addition to ground shaking, effects of seismic activity on a project site may include surface fault rupture, soil liquefaction, landslides, lateral spreading, seismic settlement, tsunamis, seiches and earthquake-induced flooding.   Results of a site-specific evaluation of each of the above secondary effects are explained below:

Surface Fault Rupture: The project site is not located within a currently designated State of California Earthquake Fault Zone. Based on review of existing geologic information, no major surface fault crosses through or extends toward the site.   The potential for surface rupture resulting from the movement of a presently unrecognized fault beneath the site is not known with certainty but is considered very low.

Liquefaction: Liquefaction is defined as the phenomenon in a soil mass, because of the development of excess pore pressures, soil mass suffers a substantial reduction in its shear strength. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction. Soil liquefaction occurs in submerged granular soils during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows:

- Soils must be submerged
- Soils must be primarily granular
- Soils must be contractive, that is, loose to medium-dense

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- Ground motion must be intense
- Duration of shaking must be sufficient for the soils to lose shear resistance

Groundwater was not encountered during our current (2019) or previous (2006) field investigation to a maximum depth of 16.4 and 51.5 feet bgs, respectively. Due to the absence of shallow groundwater, the project site is not considered susceptible to liquefaction (USGS, 2010a).

Seismic Settlement: Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. Based on the observed high blow counts below 5 feet bgs in all borings and over-excavation recommendations, we anticipate the site will have negligible seismic settlement.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or after earthquakes in areas of significant relief. The project site is not adjacent to any steep slopes. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be low.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials 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 absence of shallow groundwater and lack of liquefaction potential, the risk for lateral spreading to affect the site is considered low.

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

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Review of the area adjacent to the site indicates the site is not located in any potential inundation path of any reservoir. The potential for flooding of the site due to dam failure is considered very low (USGS, 2010b).

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8.0   LABORATORY TESTING

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the subsurface soils. Tests results are included in Appendix A, *Field Exploration* and Appendix B, *Laboratory Testing Program*. Discussions of the various test results performed for the current investigation (2019) are presented below. The test results from previous investigation (Converse, 2006) are included in Appendix B, *Laboratory Testing Program.*

8.1   Physical Testing

Physical test results are presented as follows.

- *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 109 to 128 pounds per cubic foot (pcf) with moisture contents of 3 to 15 percent. Results are presented in the logs of borings in Appendix A, *Field Exploration.*
- Expansion Index – Two representative samples from the upper ten feet of the site soils was tested to evaluate Expansion Potential in accordance with ASTM Standard D4829. The values of the measured EI are 2 and 3, indicating very low expansion potential.
- R-value – One R-value test was performed on a representative bulk soil sample in accordance with California Test 301. The R-value of the sample tested was 66.
- Collapse – To evaluate the moisture sensitivity (collapse potential) of the encountered soils, three representative ring samples were loaded up to approximately 2 kips per square foot (ksf) in accordance with ASTM Standard D4546, allowed to stabilize under load, and then submerged. The collapse ranged from 0.40 to 2.1 percent, which corresponds to slight to moderate collapse 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-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship test was performed on a representative soil sample in accordance with ASTM Standard D1557. The result is presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density and optimum moisture content of the sample tested was 133.0 pcf and 6.5 percent, respectively.
- Direct Shear – Two direct shear tests were performed on representative samples under soaked moisture condition in accordance with ASTM Standard D3080. The

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results are presented in Drawings No. B-3 and B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

8.2   Chemical Testing - Corrosivity Evaluation

One soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Test Methods 643, 422, and 417. The test results are summarized in the following table and are presented in Appendix B, *Laboratory Testing Program.*

- The pH measurement of the tested sample was 9.3.
- The sulfate contents of the tested sample were 0.0051 percent by weight.
- The chloride concentrations of the tested sample were 42 ppm.
- The minimum electrical resistivity when saturated was 4,046 ohm-cm.

9.0   PERCOLATION TESTING

Two percolation tests (PT-01 and PT-02) were conducted on June 10, 2019 to evaluate water infiltration rate of the site. The infiltration rate at the depth tested in PT-02 was deemed insufficient for the project. The borings were re-drilled to a more coarse-grained soil layer two feet deeper. Two additional percolation tests were conducted on July 12, 2019. The measured percolation test data and calculations for conversion to infiltration rate, porosity correction, and factor of safety are shown on Plates No. 1 through 4, *Estimated Infiltration Rate from Percolation Test Data* and graphically represented on Plates No. 5 and 8, *Infiltration Rate Versus Time* in Appendix C, *Water Infiltration Testing*. The estimated infiltration rate at the test hole is presented in the following table.

Table No. 4, Estimated Infiltration Rates

Percolation Test	Depth (feet)	Soil Type	Infiltration Rate (inches/hour)
PT-01	8	Silty Sand (SM)	1.30
PT-02	8	Sandy Silt (ML)	0.17
PT-01 (2)	10	Silty Sand (SM)	1.27
PT-02 (2)	10	Silty Sand (SM)	1.01

Based on the calculated infiltration rate during the final respective intervals in each test, we recommend an infiltration rate of 0.17 inches per hour at a depth of 8 feet bgs and 1.01 inches per hour at a depth of 10 feet bgs in the area of the infiltration basin.

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10.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

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

10.1 General

This section contains our general recommendations regarding earthwork and site grading for the proposed development. These recommendations are based on our experience with similar projects in the area and the results of our field exploration, laboratory testing, and data evaluation as presented in the preceding sections. These recommendations may need to be modified based on observation of the actual field conditions during grading. While a grading plan is not yet available, it is our present understanding that the import of soil will be required to achieve proposed design grades. All borrow soils should be tested and evaluated by the geotechnical consultant prior to importing to the site.

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 utilities and structure (if any).

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

The final bottom surfaces of all excavations should be observed to locate zones of overly saturated and/or loose unsuitable material of any origin and should be approved by the project geotechnical consultant prior to placing any fill and/or structures. Based on observations, removal of localized areas deeper than those documented may be required during grading. Some variations in the depth and lateral extent of over-excavation recommended in this report should be anticipated.

10.2 Subgrade Preparation-Fill Areas

About five feet of alluvial soils should be removed and replaced with compacted fill, prior to placing additional compacted fill. The actual depth of removal should be based on observations made during grading. The specific over-excavation recommendations are provided in later sections of this report.

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10.3   Over-excavation/Removal within Building Pads

In cut areas, deeper excavation may be required below finish grade. If less than five feet is removed from original ground (og), excavation should continue to provide a minimum of two feet of compacted fill below bottom of footings. If more than five feet is removed, the bottom surface should be evaluated for suitability by the geotechnical consultant. All over-excavations should extend at least five feet or equal to the depth of over-excavation, whichever is greater, outside the building footprint. If future construction is permitted beyond the lateral over-excavation, over-excavation should extend 5 feet beyond the new limits.

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).

10.4   Transition Lots

The cut portion of transition lots (and if necessary, the fill portion) should be excavated to a depth to provide a minimum of two feet of compacted fill beneath the entire pad.

10.5   Over-excavation/Removal for Pavement Areas

As a minimum, the upper three feet of surficial soils from all areas receiving asphalt concrete or Portland concrete paving, including driveways, sidewalks, street areas, curbs and gutters and other flatwork should be excavated, removed if necessary, and/or replaced as compacted fill. Such over-excavation should extend at least two feet beyond the pavement area edges.

10.6   Over-excavation/Removal for Retaining/Perimeter Walls

As a minimum, the upper three feet of surficial soils within two feet of either side of retaining/perimeter walls less than six feet in height, should be excavated, removed if necessary, and/or processed and replaced as compacted fill. The depth of the structural fill under retaining/perimeter wall footings should be at least two feet or equal to footing width, whichever is greater.

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10.7   Engineered Fill

No fill or aggregate 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 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 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.

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

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

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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.

At the time of our field investigation, *in-situ* moisture content of the upper six and one-half feet of native soils ranged from 1 to 13 percent. The optimum moisture contents were between 6.5 and 8.0 percent. Therefore, moisture conditioning may be necessary prior to the material being placed as compacted fill. The amount of processing required for proper moisture conditioning at the site will depend on the variations in the *in-situ* moisture conditions, the equipment, and the processing method.

10.9   Backfill Recommendations Behind Subterranean Wall

Compaction of backfill adjacent to structural walls can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the walls. The compaction of wall backfill should be conducted procedure described in section 10.8 *Compaction fill placement*

10.10 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, and the grading method and equipment utilized. For preliminary estimation, shrinkage factors for various units of earth material 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 range from 6 to 12 percent. An average value of 9 percent may be used for preliminary earthwork planning.
- 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. Ground subsidence may be negligible as the site is previously graded.

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.

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10.11 Site Drainage

Adequate positive drainage should be provided away from the 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.

10.12 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 (if any).

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.

10.12.1 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.

10.12.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Recommendations 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. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

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

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

    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
        $D15(F)$ = Particle size through which 15% of bedding material will pass
        $D85(B)$ = Particle size through which 85% of surrounding soil will pass
        $D50(F)$ = Particle size through which 50% of bedding material will pass
        $D50(B)$ = 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.

10.12.3   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. Trench backfill recommendations are presented below.

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- 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.

11.0 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.

11.1 Shallow Foundation Design Parameters

Residential one- or two-story wood-frame, lightly loaded structures may be supported on conventional continuous (strip) and/or isolated (spread) footings.

Interior and exterior footings should be placed at least 12 inches and 18 inches, respectively, below lowest adjacent soil grade.

Width of the continuous and isolated footings for one-story buildings should be at least 12 inches and 18 inches, respectively. Width of the continuous and isolated footings for two-story buildings should be at least 18 inches and 24 inches, respectively.

Footings placed at a depth of 12 inches and 18 inches below lowest adjacent grade may be designed based on an allowable net bearing capacity of 2,000 pounds per square foot (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 100 psf with each foot of additional width up to a maximum of 3,000 psf.

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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.

11.2   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.

11.2.1 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 are presented in the following table.

Table No. 5, 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)	40
At-rest (wall is restrained)	60

These pressures assume a level ground surface behind the walls for a distance greater than the walls height and 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.

11.2.2 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 250 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.

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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.

11.3   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 10.8 *Compacted Fill Placement*.

Slabs-on-grade should have a minimum thickness of 4 inches for support of nominal live loads. 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.

If moisture-sensitive flooring or environments are planned, slabs-on-grade should be protected by 10-mil-thick polyethylene vapor barriers. The sub-grade surface should be free of all exposed rocks or other sharp objects prior to placement of the barrier. The barrier should be overlain by 2 inches of sand, to minimize punctures and to aid in the concrete curing. At discretion of the structure engineer, the sand layer may be eliminated.

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.

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 (maximum 0.45).

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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.

11.4 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 40 feet.

Based on the observed high blow counts below 5 feet bgs in all borings and over-excavation recommendations, we anticipate the site may have negligible seismic settlement. For the design purpose, seismic settlement may be taken as 1 inch or less and the differential settlement may be taken as half of the total seismic settlement.

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.

11.5 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. 6, *Soil Parameters for Pipe Design*.

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 shutoffs should be installed to limit the potential leakage in the event of damage in a seismic event.

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Table No. 6, Soil Parameters for Pipe Design

Soil Parameters	Parameters
Unit weight of compacted backfill (assuming 92% average relative compaction), γ	130 pcf
Angle of internal friction of soils, ϕ	30
Soil cohesion, c	50 pcf
Coefficient of friction between concrete and native soils, fs	0.30
Coefficient of friction between pipe and native soils, fs	0.25 for RCP/PVC/HDPE pipe
Bearing pressure against Alluvial Soils	2,000 psf
Coefficient of passive earth pressure, Kp	3.0
Coefficient of active earth pressure, Ka	0.33
Modulus of Soil Reaction, E'	1,500 psi

11.6   Bearing Pressure for Anchor and Thrust Blocks

An allowable net bearing pressure presented in Table No. 5, *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.

11.7   Soil Corrosivity

Two representative soil samples (one is 2006 and another in 2019) were evaluated for corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program* and design recommendations pertaining to soil corrosivity are presented below.

The sulfate contents of the sampled soils correspond to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-14, 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 the results of chloride testing of the site soils, we do not anticipate that concrete structures

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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 were 876 and 4,046 ohm-cm for the site. This indicates that the soils tested are mildly corrosive to severely corrosive to ferrous metals in contact with the soil (Romanoff, 1957).

Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.

11.8 Pavement Recommendations

Two soil samples (one in 2006 and another in 2019) were tested to determine the R-value of the subgrade soils. Based on laboratory testing, R-values were 16 and 46. For pavement design, we have utilized an R-value of 16 and design Traffic Indices (TIs) ranging from 5 to 10.

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 below.

Table No. 7, Recommended Preliminary Pavement Sections

	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
Design R-value 16	5	4.0	5.5	7.0
	6	4.0	9.5	9.0
	7	5.0	11.0	11.0
	8	6.0	13.0	13.5
	9	7.0	14.0	15.5
	10	8.0	15.0	17.5

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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, 2018) 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.

11.9   Concrete Flatwork

Except as modified herein, concrete walks, driveways, access ramps, curb and gutters should be constructed in accordance with Section 303-5, *Concrete Curbs, Walks, Gutters, Cross-Gutters, Alley Intersections, Access Ramps, and Driveways*, of the Standard Specifications for Public Works Construction (Public Works Standards, 2018).

The subgrade soils under the above structures should consist of compacted fill placed as described in this report. Prior to placement of concrete, the upper 12 inches of subgrade soils should be moisture conditioned to between within 3 percent of optimum moisture content for coarse-grained soils and 0 and 2 percent above optimum for fine-grained soils.

The thickness of driveways for passenger vehicles should be at least 4 inches, or as required by the civil or structural engineer. Transverse control joints for driveways should be spaced not more than 10 feet apart. Driveways wider than 12 feet should be provided with longitudinal control joints.

Concrete walks subjected to pedestrian and bicycle loading should be at least 4 inches thick, or as required by the civil or structural engineer. Transverse joints should be spaced 15 feet or less and should be cut to a depth of one-fourth the slab thickness.

Positive drainage should be provided away from all driveways and sidewalks to prevent seepage of surface and/or subsurface water into the concrete base and/or subgrade.

12.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

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12.1   General

Prior to the start of construction, all existing underground utilities (if any) 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.

Vertical braced excavations can be considered for the foundations. Sloped excavations may not be feasible in locations adjacent to existing utilities, pavement or structure (if any). 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.

12.2   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. 8, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
Silty Sand (SM), Sand with Silt (SP-SM), Clayey Sand (SC), Sandy Silt (ML) and Sand (SP)	C	0-10	1.5:1

[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

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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.

13.0 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.

14.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Lansing Companies 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

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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.

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15.0 REFERENCES

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Appendix A

Field Exploration

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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 locations of the borings were selected. The borings were located using existing topography and boundary features and should be considered accurate only to the degree implied by the method used.

For the previous investigation performed by Converse, a total of seven exploratory borings (BH-1 to BH-7) were drilled on December 7, 2005 across the project site, to depths of 16.5 to 51.5 feet below ground surface (bgs).

Additionally, two exploratory borings (BH-8 and BH-9) were drilled on June 3, 2019 to investigate subsurface conditions at the project site. The borings were drilled to depths of 15.8 and 16.4 feet below existing ground surface (bgs).

Two exploratory percolation test holes (PT-01 and PT-02) were drilled on June 3, 2019 to perform percolation testing. Both percolation test borings were drilled to approximately 8.0 feet below the existing ground surface (bgs).

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. Encountered materials were continuously logged by a Converse 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 in boring BH-4 (2006) at depths of 20, 25, 30, 35, 40, 45 and 50 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.

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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 and tamped. If construction is delayed, the surface may settle over time. Therefore, we recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 through A-8 (2006) and A-9 through A-12 (2019), Logs of Borings.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
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)		STRENGTH	
		Pocket Penetrometer	p
		Direct Shear	ds
		Direct Shear (single point)	ds*
CLASSIFICATION		Unconfined Compression	uc
Plasticity	pi	Triaxial Compression	tx
Grain Size Analysis	ma	Vane Shear	vs
Passing No. 200 Sieve	wa		
Sand Equivalent	se	Consolidation	c
Expansion Index	ei	Collapse Test	col
Compaction Curve	max	Resistance (R) Value	r
Hydrometer	h	Chemical Analysis	ca
Disturb	Dist.	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

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Project No.
19-81-173-01

Drawing No.
A-1

Project ID: 19-81-173-01.GPJ; Template: KEY

Log of Boring No. BH - 1

| Dates Drilled: | 12/7/2005 | Logged by: | FA | Checked By: | RJR |

Equipment: __8" HOLLOW STEM AUGER__ Driving Weight and Drop: __140 lbs / 30 in__

Ground Surface Elevation (ft): __2899__ 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.	DRIVE	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal) **SAND WITH SILT (SP-SM):** medium- to coarse-grained, some gravel, brown.						ma
					8/11/15	3	111	col
5					13/25/50 (6")	4	107	ds
		- orange brown			25/50 (6")	2	108	
10					25/50 (4")	4	106	
15					25/50 (5.5")	4	95	
20					30/50 (5")	5	96	
		End of Boring at 21.5 feet. Groundwater not encountered during drilling. Boring backfilled with soil cuttings on 12/7/05.						

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Project No. **05-81-351-01**

Drawing No. **A - 2**

Project ID: 05-81-351-01.GPJ; Template: LOG

Log of Boring No. BH - 2

Dates Drilled: _____12/7/2005_____ Logged by: _____FA_____ Checked By: ____RJR____

Equipment: ___8" HOLLOW STEM AUGER___ Driving Weight and Drop: ___140 lbs / 30 in___

Ground Surface Elevation (ft): ____2879____ 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	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal) **SILTY SAND (SM):** fine- to medium-grained, brown.						er, ca
					9/15/20	5	107	
5					7/15/21	8	113	col
					13/17/27	9	114	
10		some gravel			25/50 (3")	7	104	
15		**SAND (SP):** fine- to coarse-grained, orange brown.			25/50 (5")	1	102	

End of Boring at 16.5 feet.
Groundwater not encountered during drilling.
Boring backfilled with soil cuttings on 12/7/05.

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Project No. 05-81-351-01

Drawing No. A - 3

Log of Boring No. BH - 3

Dates Drilled:	12/7/2005	Logged by:	FA	Checked By:	RJR

Equipment: __8" HOLLOW STEM AUGER__   Driving Weight and Drop: __140 lbs / 30 in__

Ground Surface Elevation (ft): __2886__   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	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal) **SILTY SAND (SM):** fine- to medium-grained, brown.						ei
					3/5/9	4	102	
5		**SANDY SILT (ML):** fine-grained sand with clay, brown.			7/10/17	13	103	
		GRAVELLY SAND (SP): medium- to coarse-grained, light brown.			41/50 (6")	2	123	col
10		**SAND (SP):** fine-grained with gravel, pink brown.			22/50 (2")	4	107	
15		**SILTY SAND (SM):** fine- to medium-grained, brown.			16/50 (5")	5	107	

End of Boring at 16.5 feet.
Groundwater not encountered during drilling.
Boring backfilled with soil cuttings on 12/7/05.

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Project No.	Drawing No.
05-81-351-01	A - 4

Log of Boring No. BH - 4

Dates Drilled: 12/7/2005   Logged by: FA   Checked By: RJR

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

Ground Surface Elevation (ft): 2892   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.	DRIVE	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal)						max, ds
		SILTY SAND (SM): fine- to medium-grained, dark brown.			5/8/14	4	113	
5					15/17/27	6	120	c
		SAND WITH SILT (SP-SM): fine- to coarse-grained, some gravel, brown.			15/32/50 (5")	4	107	
10		**GRAVELLY SAND (SP):** medium- to coarse-grained, light gray.			30/50 (2")	2	113	
15		**SILTY SAND (SM):** fine-grained, light brown.			30/50 (3")	6	98	
20					41/50 (3")			
25		- fine- to medium-grained			50 (3")			
30		- medium- to coarse-grained, brown			50 (3")			

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Drawing No. A - 5a

Log of Boring No. BH - 4

Dates Drilled: 12/7/2005   Logged by: FA   Checked By: RJR

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

Ground Surface Elevation (ft): 2892   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	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SILTY SAND (SM): fine-grained, light brown.			50 (2")			
40		– dark brown	⊠		50 (6")			
45		– brown	⊠		50 (5")			
50			⊠		55 (6")			

End of Boring at 51.5 feet.
Groundwater not encountered during drilling.
Boring backfilled with soil cuttings on 12/7/05.

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Drawing No. A - 5b

Log of Boring No. BH - 5

| Dates Drilled: | 12/8/2005 | Logged by: | FA | Checked By: | RJR |

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

Ground Surface Elevation (ft): 2901   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/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal) **CLAYEY SAND (SC):** fine- to coarse-grained, brown.						r, ei
					27/50 (5")	13	122	
5		**SILTY SAND (SM):** fine- to medium-grained, brown.			50 (4")	7	102	col
					50 (6")	4	97	
10		**SANDY SILT (ML):** fine-grained sand with clay, light brown.			37/50 (2")	8	92	
15		**SILTY SAND (SM):** medium- to coarse-grained, brown.			50 (4")	5	98	

End of Boring at 16.5 feet.
Groundwater not encountered during drilling.
Boring backfilled with soil cuttings on 12/8/05.

Converse Consultants

APPROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Project No. 05-81-351-01

Drawing No. A - 6

Log of Boring No. BH - 6

Dates Drilled: **12/7/2005**	Logged by: **FA** Checked By: **RJR**
Equipment: **8" HOLLOW STEM AUGER**	Driving Weight and Drop: **140 lbs / 30 in**
Ground Surface Elevation (ft): **2916**	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	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal) **SILTY SAND (SM):** fine- to coarse-grained, brown.						ma
					6/15/25	3	111	
5		**GRAVELLY SAND (SP):** fine- to coarse-grained, orange brown.			25/36/37	2	112	
					18/37/50 (6")	2	125	
10		**SAND (SP):** medium- to coarse-grained, brown.			18/30/43	2	109	
15		**SANDY SILT (ML):** fine- to medium-grained sand, orange brown.			27/50 (3")	15	106	

End of Boring at 16.5 feet.
Groundwater not encountered during drilling.
Boring backfilled with soil cuttings on 12/8/05.

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APRROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
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Project No. **05-81-351-01**

Drawing No. **A - 7**

Log of Boring No. BH - 7

Dates Drilled: 12/7/2005	Logged by: FA — Checked By: RJR
Equipment: 8" HOLLOW STEM AUGER	Driving Weight and Drop: 140 lbs / 30 in
Ground Surface Elevation (ft): 2905	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	DRIVE	BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal) **SAND WITH SILT (SP-SM):** fine- to coarse-grained, some gravel, brown.			5/8/23	2	116	
5					17/37/50 (3")	1		dist.
					37/50 (3")	3	110	col
10		**SANDY SILT (ML):** fine-grained sand with caliche, brown.			40/50 (3")	12	107	
15		- fine- to medium-grained			23/50 (5")	10	103	

End of Boring at 16.5 feet.
Groundwater not encountered during drilling.
Boring backfilled with soil cuttings on 12/8/05.

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APRROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Project No. 05-81-351-01

Drawing No. A - 8

Log of Boring No. BH-8

Dates Drilled: 6/3/2019   Logged by: Catherine Nelson   Checked By: James Burnham

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

Ground Surface Elevation (ft): 2901   Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
		ALLUVIUM **SAND WITH SILT (SP-SM):** fine to coarse-grained, few gravel up to 2" in largest dimension, light brown.			12/20/28	3	109	
5		**SANDY SILT (ML):** fine to medium-grained sand, scattered gravel up to 1.5" in largest dimension, brown.			12/47/50-4"	11	110	ds
					17/50-4"	15	115	ei col
10		**SILTY SAND (SM):** fine to coarse-grained, brown.			25/50-4"	8	113	
15		- scattered gravel up to 1" in largest dimension			17/48/50-4"	7	124	
		End of boring at 16.4 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped on 06/10/2019.						

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For: Lansing Companies

Project No. 19-81-173-01

Drawing No. A-9

Log of Boring No. BH-9

Dates Drilled: 6/3/2019   Logged by: Catherine Nelson   Checked By: James Burnham

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

Ground Surface Elevation (ft): 2901   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 **SILTY SAND (SM):** fine to coarse-grained, brown.						r, ca, er ma, max
					7/10/14	4	113	col
5		**SAND WITH SILT (SP-SM):** fine to coarse-grained, brown.			7/15/26	3	119	ds
					35/50-5"	5	128	col
10		**SANDY SILT (ML):** fine-grained sand, brown.			22/48/50-3"	13	103	
15		**SILTY SAND (SM):** fine to coarse-grained, brown.			32/50-3"	7	95	

End of boring at 15.8 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 06/10/2019.

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For: Lansing Companies
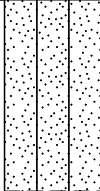
Project No. 19-81-173-01   Drawing No. A-10

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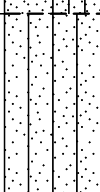
Log of Boring No. PT-01

Dates Drilled: 6/3/2019   Logged by: Catherine Nelson   Checked By: James Burnham

Equipment: 8" HOLLOW STEM AUGER   Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2877   Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES		BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		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.	DRIVE	BULK				
5		**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 2" in largest dimension, brown.		⨉				ma
		End of boring at 8.0 feet bgs. No groundwater encountered. Borehole utilized for percolation testing on 06/10/2019. Backfilled with pea-gravel and soil cuttings on 06/10/2019.						

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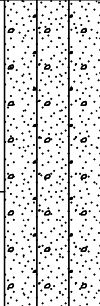For: Lansing Companies

Project No.
19-81-173-01

Drawing No.
A-11

Log of Boring No. PT-02

| Dates Drilled: | 6/3/2019 | Logged by: | Catherine Nelson | Checked By: | James Burnham |

| Equipment: | 8" HOLLOW STEM AUGER | Driving Weight and Drop: | N/A |

| Ground Surface Elevation (ft): | 2876 | Depth to Water (ft): | NOT ENCOUNTERED |

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES		BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		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.	DRIVE	BULK				
5		**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 2" in largest dimension, brown.						ei
		End of boring at 8.0 feet bgs. No groundwater encountered. Borehole utilized for percolation testing on 06/10/2019. Backfilled with pea-gravel and soil cuttings on 06/10/2019.						

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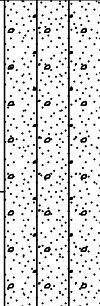| Project No.	Drawing No.
19-81-173-01	**A-12**

Project ID: 19-81-173-01.GPJ; Template: LOG

Appendix B

Laboratory Testing Program

Updated Geotechnical Investigation & Water Infiltration Test Report
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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. The test results from previous investigation (Converse, 2006) are also included.

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 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

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

Table No. B-1, Expansion Index Test Results

Boring No./Report	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-8/2019	5-10	Sandy Silt (ML)	3	Very Low
PT-02/2019	5-8	Silty Sand (SM)	2	Very Low
BH-3/2006	0-5	Silty Sand (SM)	0	Very Low
BH-5/2006	0-5	Clayey Sand (SC)	43	Low

R-value

Two representative bulk soil samples were tested for resistance value (R-value) in accordance with California Test Method CT301. The test provides a relative measure of

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soil strength for use in pavement design. The test results are shown in the following table.

Table No. B-2, R-Value Test Results

Boring No./Report	Depth (feet)	Soil Classification	Measured R-value
BH-9/2019	0-5	Silty Sand (SM)	46
BH-5/2006	0-5	Clayey Sand (SC)	16

Soil Corrosivity

One representative soil sample (2019) was tested by AP Engineering and Testing, Inc. (Pomona, CA and One representative soil sample (2006) was tested by Anaheim Laboratory (Santa Ana, CA) in accordance with California Tests 663, 622, and 617, to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials such as concrete and steel. Test results are presented on the following table.

Table No. B-3, Summary of Corrosivity Test Results

Boring No./Report	Depth (feet)	pH	Soluble Sulfates (CA 617) (percent by weight)	Soluble Chlorides (CA 622) (ppm)	Min. Resistivity (CA 663) (Ohm-cm)
BH-9/2019	0-5	9.3	0.0051	42	4,046
BH-5/2006	0-5	8.8	0.0040	22	876

Collapse

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, eight collapse tests were 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 results including collapse test are presented in the following table.

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Table No. B-4, Collapse Test Results

Boring No./Report	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-8/2019	7.5-9.0	Sandy Silt (ML)	-0.4	Slight
BH-9/2019	2.5-4.0	Silty Sand (SM)	-2.1	Moderate
BH-9/2019	7.5-9.0	Sand with Silt (SP-SM)	-0.6	Slight
BH-1/2006	2.0-3.5	Sand with Silt (SP-SM)	-0.8	Slight
BH-2/2006	5.0-6.5	Silty Sand (SM)	-0.4	Slight
BH-3/2006	7.0-8.5	Gravelly Sand (SP-P)	-0.35	Slight
*BH-4/2006	5.-6.5	Silty Sand (SM)	-0.25	Slight
BH-5/2006	5.0-6.5	Silty Sand (SM)	-3.03	Moderate
BH-7/2006	7.0-8.5	Sand with Silt (SP-SM)	-1.1	Slight

(*Result from consolidation test)

Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analyses were performed on four 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* and results are presented in the following table.

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

Boring No./ Report	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-9/2019	0-5	Silty Sand (SM)	0.0	83.0	17.0	
PT-01/2019	5-8	Silty Sand (SM)	1.0	79.0	20.0	
BH-1/2006	0-5	Sand with Silt (SP-SM)	13.8	76.6	9.6	
BH-6/2006	0-5	Silty Sand (SM)	6.9	73.5	19.6	

Maximum Dry Density and Optimum Moisture Content

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

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Table No. B-6, Laboratory Maximum Density Test Results

Boring No./ Report	Depth (feet)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
BH-9/2019	0-5	Silty Sand, Brown	133.0	6.5
BH-4/2006	0-5	Silty Sand, Dark Brown	134.5	8.0

Direct Shear

Three direct shear tests were performed on representative undisturbed samples and one on sample remolded to 90% of the laboratory maximum dry density under soaked moisture condition in accordance with ASTM Standard D3080. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.01 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 data, including sample density and moisture content, see Drawings No. B-3 and B-4, *Direct Shear Test Results,* and the following table.

Table No. B-7, Direct Shear Test Results

Boring No./Report	Depth (feet)	Soil Description	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-8/2019	5.0-6.5	Sandy Silt (ML)	28	200
BH-9/2019	5.0-6.5	Sand with Silt (SP-SM)	30	120
BH-1/2006	5.0-6.5	Sand with Silt (SP-SM)	41	400
*BH-4/2006	5.0-6.5	Silty Sand (SM)	40	350

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

Consolidation

Consolidation test (2006) was performed on one selected sample in accordance with the ASTM Standard D2435 test method. Data obtained from this test performed on a relatively undisturbed soil sample was used to evaluate the settlement characteristics of the foundation soils under load. Preparation for this test involved trimming the sample and placing the one-inch high brass ring into the test apparatus, which contained porous stones, both top and bottom, to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied

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at a constant load-increment ratio, successive loads being generally twice the preceding load. The sample was tested at field and submerged conditions. The test result is presented in Drawing No. B-5, *Consolidation Test Results*.

Sample Storage

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

U.S. SIEVE OPENING IN INCHES | U.S. SIEVE NUMBERS | HYDROMETER

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-9**	**0-5**	**SILTY SAND (SM)**					
⊠	**PT-01**	**5-8**	**SILTY SAND (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-9**	**0-5**	**4.75**	**0.687**	**0.111**		**0.0**	**83.0**	**17.0**	
⊠	**PT-01**	**5-8**	**9.5**	**0.443**	**0.11**		**1.0**	**79.0**	**20.0**	

GRAIN SIZE DISTRIBUTION RESULTS

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City of Victorville, San Bernardino County, California
For: Lansing Companies

Project No.	Drawing No.
19-81-173-01	**B-1**

The chart plots DRY DENSITY, pcf (y-axis, 90 to 150) versus WATER CONTENT, % (x-axis, 0 to 30). Curves of 100% Saturation for Specific Gravity Equal to: 2.80, 2.70, 2.60.

SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-9	0-5	SILTY SAND (SM), BROWN	D1557 -A	6.5	133.0

MOISTURE-DENSITY RELATIONSHIP RESULTS

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For: Lansing Companies

Project No. **19-81-173-01**

Drawing No. **B-2**

Project ID: 19-81-173-01.GPJ; Template: COMPACTION

SHEAR STRENGTH, psf

SURCHARGE PRESSURE, psf

BORING NO.	:	**BH-8**	DEPTH (ft)	:	**5.0-6.5**
DESCRIPTION	:	**SANDY SILT (ML)**			
COHESION (psf)	:	**200**	FRICTION ANGLE (degrees):		**28**
MOISTURE CONTENT (%)	:	**11**	DRY DENSITY (pcf)	:	**113.0**

NOTE: Ultimate Strength.
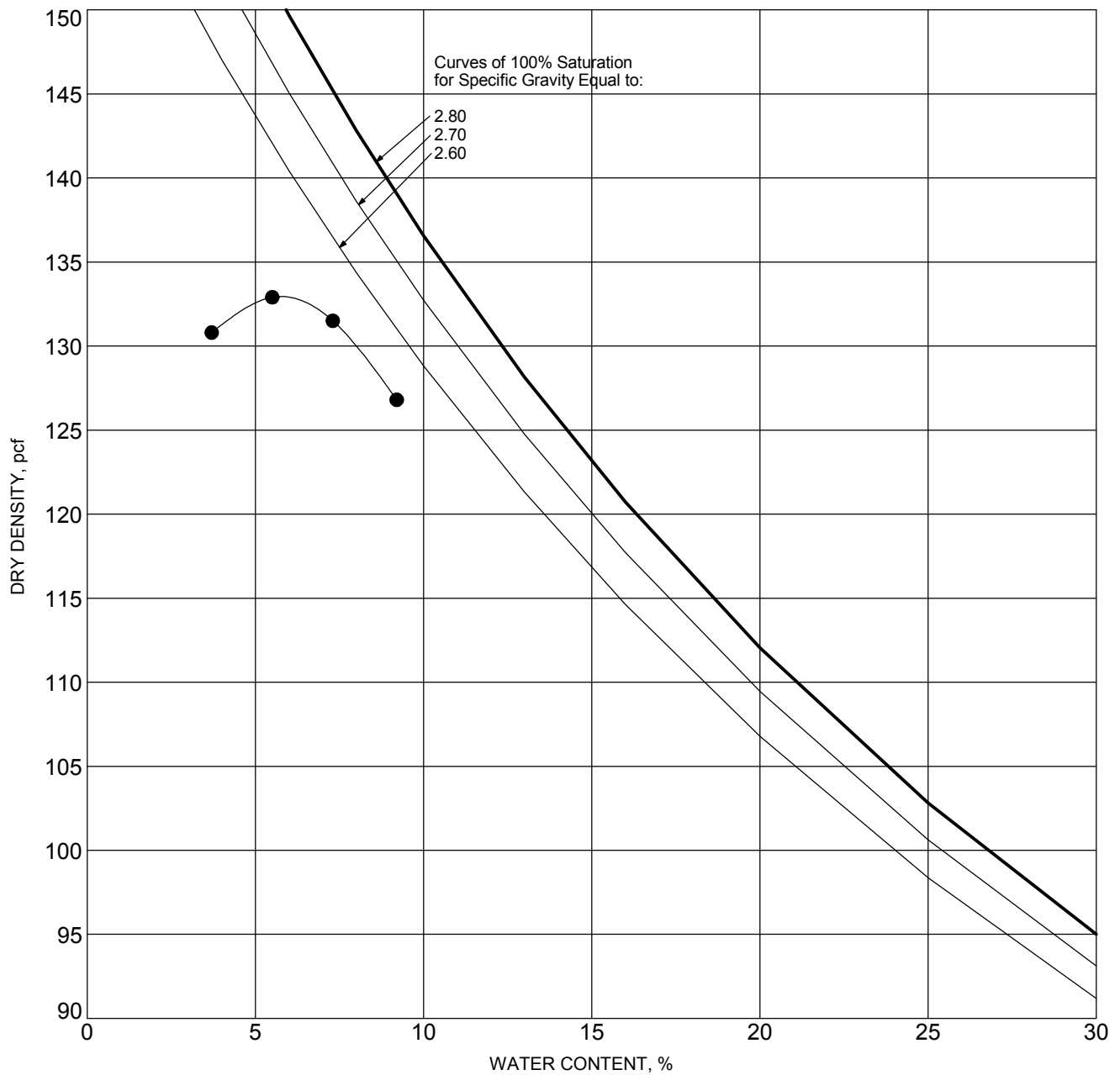
DIRECT SHEAR TEST RESULTS

Converse Consultants

Approximately 20.60-Acre Residential Development
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For: Lansing Companies

Project No.	Drawing No.
19-81-173-01	**B-3**

Project ID: 19-81-173-01.GPJ; Template: DIRECT SHEAR

BORING NO. :	**BH-9**		DEPTH (ft) :	**5.0-6.5**
DESCRIPTION :	**SAND WITH SILT (SP-SM)**			
COHESION (psf) :	**120**		FRICTION ANGLE (degrees):	**30**
MOISTURE CONTENT (%) :	**3.0**		DRY DENSITY (pcf) :	**119.0**

NOTE: Ultimate Strength.
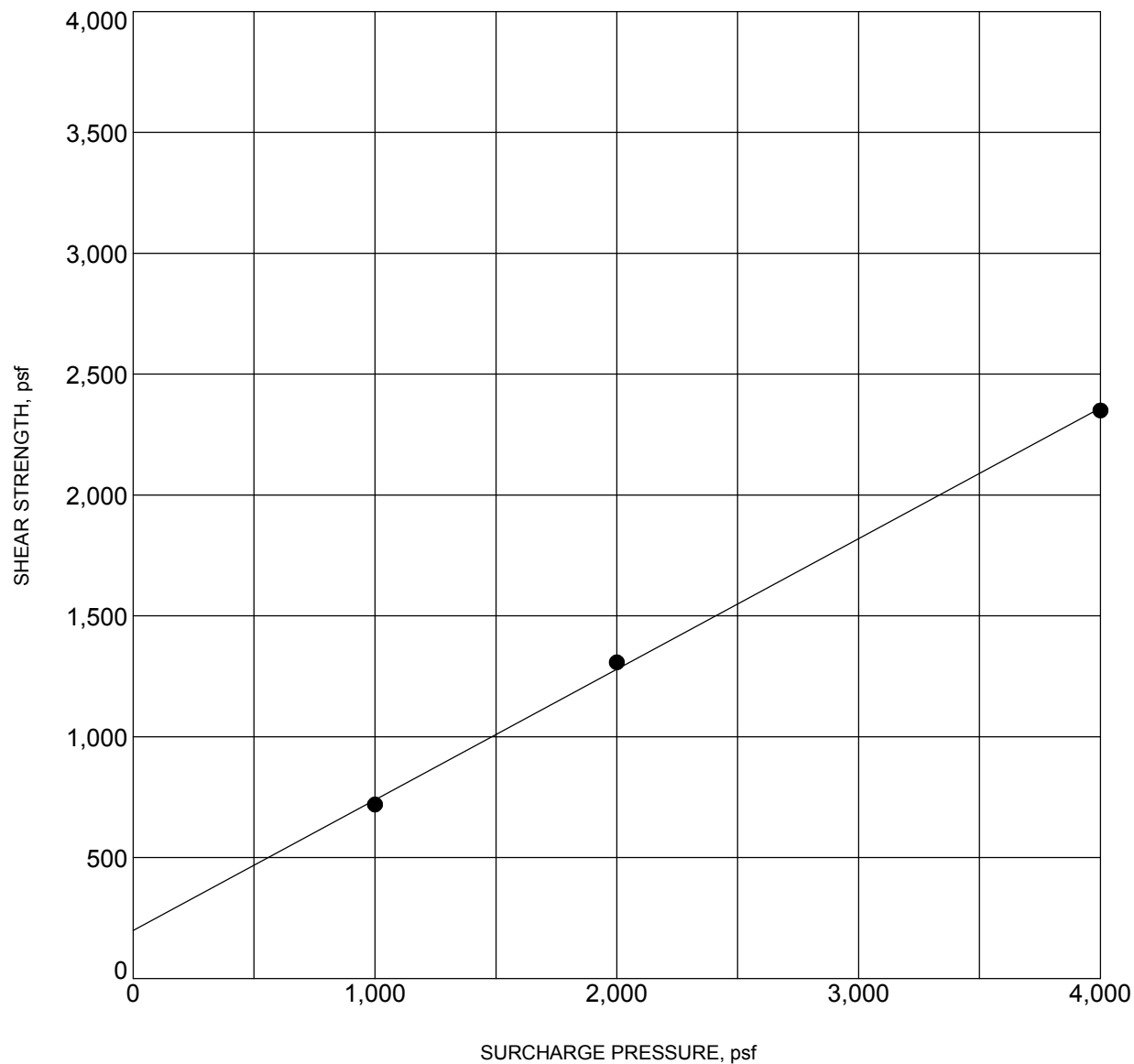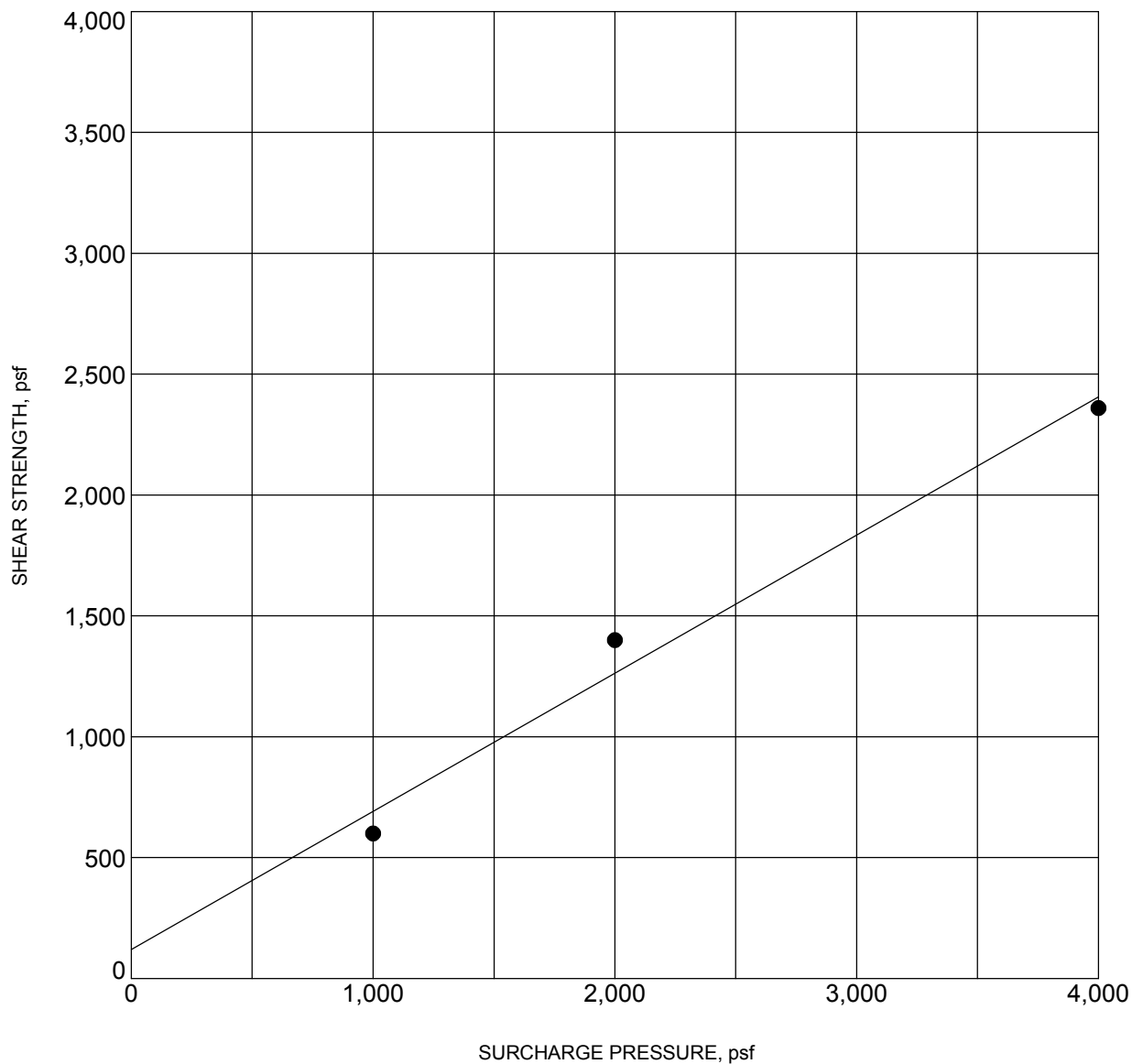
DIRECT SHEAR TEST RESULTS

Converse Consultants

Approximately 20.60-Acre Residential Development
Southeast Corner of Hopland Street and Cahuenga Road
City of Victorville, San Bernardino County, California
For: Lansing Companies

Project No.	Drawing No.
19-81-173-01	**B-4**

GRAIN SIZE DISTRIBUTION RESULTS

U.S. SIEVE OPENING IN INCHES | U.S. SIEVE NUMBERS | HYDROMETER

PERCENT FINER BY WEIGHT / GRAIN SIZE IN MILLIMETERS

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	BH - 1	0 - 5'	SAND WITH SILT (SP-SM)				2.98	19.60
⊠	BH - 6	0 - 5'	SILTY SAND (SM)					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	BH - 1	0 - 5'	25.4	1.622	0.633	0.083	13.8	76.6	9.6	
⊠	BH - 6	0 - 5'	19	1.266	0.296		6.9	73.5	19.6	

GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

APRROXIMATELY 30 - ACRE SITE
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For: Victory Ridge

Project No.	Drawing No.
05-81-351-01	B - 1

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80
2.70
2.60

DRY DENSITY, pcf

150
145
140
135
130
125
120
115
110
105
100
95
90

0   5   10   15   20   25   30

WATER CONTENT, %

SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH - 4	0 - 5'	SILTY SAND (SM)	A	8.0	134.5

MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

APRROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Project No.
05-81-351-01

Drawing No.
B - 2

SHEAR STRENGTH, psf (y-axis)

SURCHARGE PRESSURE, psf (x-axis)

BORING NO.	:	**BH - 1**	DEPTH (ft)		**5.0 - 6.5'**
DESCRIPTION	:	**SAND WITH SILT (SP-SM)**			
COHESION (psf)	:	**400**	FRICTION ANGLE (degrees)		**41**
MOISTURE CONTENT (%)	:	**3.6**	DRY DENSITY (pcf)	:	**107.4**

NOTE: Ultimate Strength.
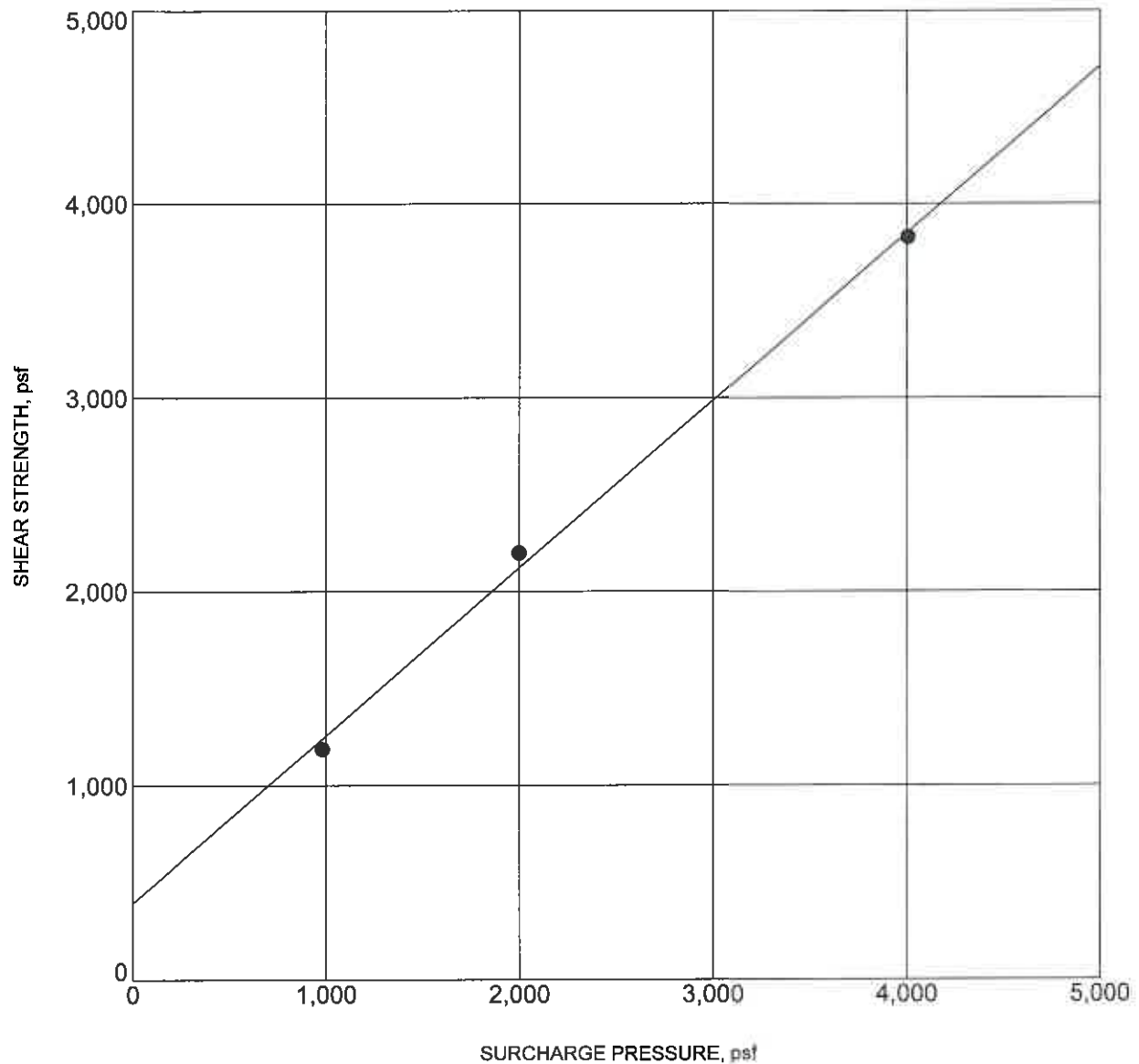
DIRECT SHEAR TEST RESULTS

 Converse Consultants

APRROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Project No.
05-81-351-01

Drawing No.
B - 3

SHEAR STRENGTH, psf

SURCHARGE PRESSURE, psf

BORING NO. :	BH - 4	DEPTH (ft)	0 - 5'
DESCRIPTION :	SILTY SAND (SM)		
COHESION (psf) :	350	FRICTION ANGLE (degrees)	40
MOISTURE CONTENT (%) :	8.5	DRY DENSITY (pcf) :	121.5

NOTE: Ultimate Strength, Sample Remolded to 90% Relative Compaction
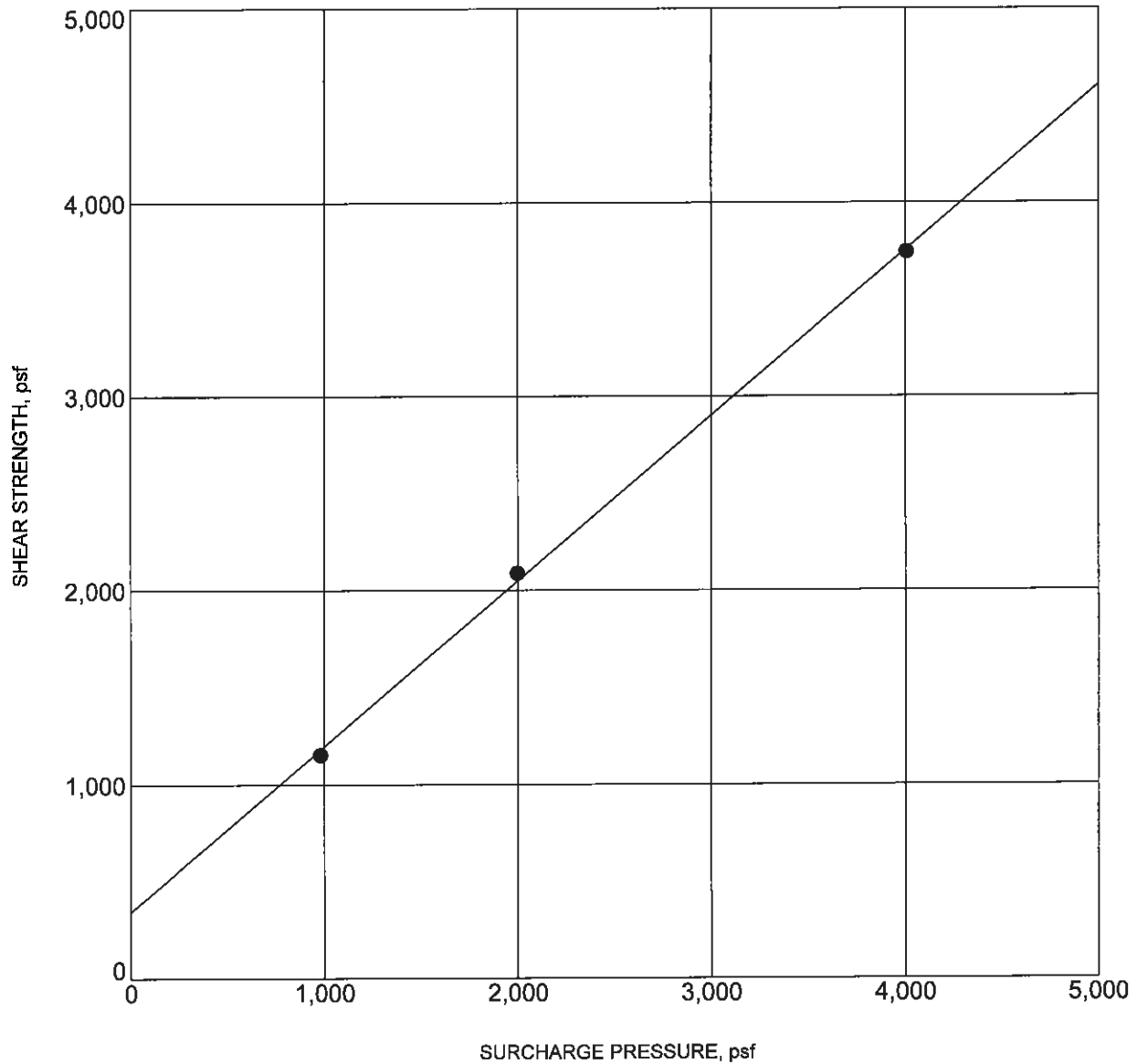
DIRECT SHEAR TEST RESULTS

Converse Consultants

APPROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Project No.
05-81-351-01

Drawing No.
B - 4

Project ID: 05-81-351-01.GPJ; Template: DIRECT SHEAR

BORING NO. :	BH - 4		DEPTH (ft) :		5.0 - 6.5'
DESCRIPTION :		SILTY SAND (SM)			
MOISTURE CONTENT (%)		DRY DENSITY (pcf)	PERCENT SATURATION		VOID RATIO
INITIAL	5.8	119.8	41		0.377
FINAL	11.5	125	100		0.319

NOTE: Solid Circles Indicate Readings After Addition of Water
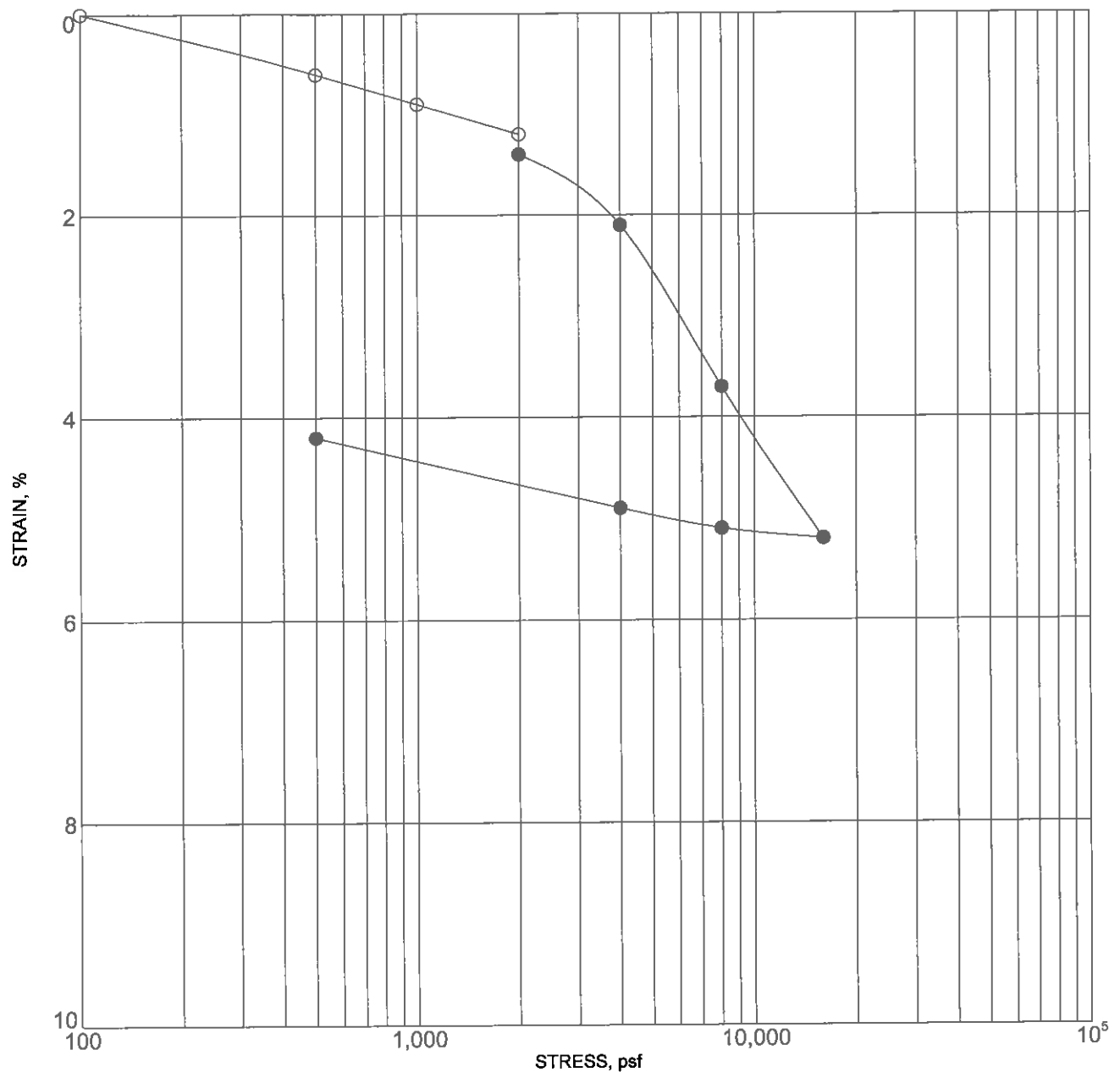
CONSOLIDATION TEST RESULTS

Converse Consultants

APPROXIMATELY 30 - ACRE SITE
City of Victorville, San Bernardino County, California
For: Victory Ridge

Project No.
05-81-351-01

Drawing No.
B - 5

Appendix C

Water Filtration Testing

Updated Geotechnical Investigation & Water Infiltration Test Report
Approximately 20.60-Acre Residential Development
Southeast Corner of Hopland Street and Cahuenga Road
City of Victorville, San Bernardino County, California
July 16, 2019
Page C-1

APPENDIX C

WATER INFILTRATION TESTING

Percolation testing was performed at two locations (PT-01 and PT-02) on June 10 and July 12, 2019 in general accordance with the San Bernardino County Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2011) for using a percolation testing method to estimate infiltration rates*.*

Upon completion of drilling the test hole, a 2-inch thick gravel layer was placed at the bottom of the hole and a 3.0-inch diameter perforated pipe was installed above the gravel to the ground surface. The boring annulus around the pipe was filled with gravel. The purpose of the pipe and gravel was to reduce the potential for erosion and caving due to the addition of water to the hole.

June 10, 2019

The test holes were presoaked by filling with water to at least 5 times the radius of the test holes. More than 6 inches of water seeped away from PT-01 in less than 25 minutes for 2 consecutive measurements, meeting the criteria for testing as "sandy soil". Less than 6 inches of water seeped away from PT-02 in less than 25 minutes for 2 consecutive measurements, meeting the criteria for testing as "soil with fines". Percolation testing was conducted immediately after presoaking. During testing, the water level and total depth of PT-01 was measured from the top of the pipe every 10 minutes for one hour. The water level and total depth of PT-02 was measured from the top of the pipe every 30 minutes for six hours. Following the completion of percolation testing, the pipes were left in the ground and the percolation test hole was backfilled with cutting soils.

July 12, 2019

The test holes were presoaked by filling with water to at least 5 times the radius of the test holes. More than 6 inches of water seeped away from PT-01 (2) and PT-02 (2) in less than 25 minutes for 2 consecutive measurements, meeting the criteria for testing as "sandy soil". Percolation testing was conducted immediately after presoaking. During testing, the water level and total depth was measured from the top of the pipe every 10 minutes for one hour. Following the completion of percolation testing, the pipes were removed, and the test hole was backfilled with soil cuttings.

Percolation rates describe the movement of water horizontally and downward into the soil from a boring. Infiltration rates describe the downward movement of water through a horizontal surface, such as the floor of a retention basin. Percolation rates are related to infiltration rates but are generally higher and require conversion before use in design. The

Updated Geotechnical Investigation & Water Infiltration Test Report
Approximately 20.60-Acre Residential Development
Southeast Corner of Hopland Street and Cahuenga Road
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percolation test data was used to estimate infiltration rates using the Porchet Inverse Borehole Method, in accordance with the San Bernardino County guidelines. A conversion factor derived from California Test 750 (Caltrans, 1986) was applied to adjust for the presence of the gravel and pipe within the borehole. A factor of safety of 3 was applied to the measured infiltration rates to account for subsurface variations, uncertainty in the test method, and future siltation. The infiltration structure designer should determine whether additional design-related safety factors are appropriate.

The measured percolation test data and calculations for conversion to infiltration rate, porosity correction, and factor of safety are shown on Plates No. 1 through 4, *Estimated Infiltration Rate from Percolation Test Data* and graphically represented on Plates No. 5 through 8, *Infiltration Rate Versus Time.* The estimated infiltration rate at the test holes and depths are presented in the following table.

Table No. C-1, Estimated Infiltration Rates

Percolation Test	Depth (feet)	Soil Type	Infiltration Rate (inches/hour)
PT-01	8	Silty Sand (SM)	1.30
PT-02	8	Sandy Silt (ML)	0.17
PT-01	10	Silty Sand (SM)	1.27
PT-02	10	Silty Sand (SM)	1.01

Based on the calculated infiltration rate during the final respective intervals in each test, we recommend an infiltration rate of 0.17 inches per hour at a depth of 8 feet bgs and 1.01 inches per hour at a depth of 10 feet bgs in the area of the basin.

Estimated Infiltration Rate from Percolation Test Data, PT-01

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-01
Personnel	Catherine Nelson
Presoak Date	6/10/2019
Test Date	6/10/2019

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	96
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.40
Porosity of Gravel, n	0.48
Porosity Correction Factor, C	0.50
Factor of Safety (FOS), F	2

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)	Corrected Infiltration Rate, I_c (inches/hr)	Infiltration Rate with FOS, I_f (inches/hr)
1	25.00	31.20	80.28	25.00	64.80	15.72	49.08	40.26	5.57	2.78	1.39
2	25.00	31.20	79.56	50.00	64.80	16.44	48.36	40.62	5.45	2.72	1.36
3	10.00	31.20	59.88	60.00	64.80	36.12	28.68	50.46	6.56	3.28	1.64
4	10.00	31.20	59.40	70.00	64.80	36.60	28.20	50.70	6.42	3.21	1.60
5	10.00	31.20	57.36	80.00	64.80	38.64	26.16	51.72	5.84	2.92	1.46
6	10.00	31.20	56.16	90.00	64.80	39.84	24.96	52.32	5.51	2.75	1.38
7	10.00	31.20	55.44	100.00	64.80	40.56	24.24	52.68	5.32	2.66	1.33
8	10.00	31.20	55.08	110.00	64.80	40.92	23.88	52.86	5.22	2.61	1.30

Recommended Design Infiltration Rate (inches/hr)	1.30

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Santa Ana Regional Water Quality Control Board Technical Guidance Document, Appendix VII, Example VII.1.

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg})))$

Porosity conversion calculations are based on the method provided in Caltrans California Test 750.

$C = n * (1 - (O / (2 * r))^2) + (I / (2 * r))^2$

$I_c = I_t * C$

$I_f = I_C * F$

Plate No.

1

Estimated Infiltration Rate from Percolation Test Data, PT-02

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-02
Personnel	Catherine Nelson
Presoak Date	6/10/2019
Test Date	6/10/2019

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	96
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.40
Porosity of Gravel, n	0.48
Porosity Correction Factor, C	0.50
Factor of Safety (FOS), F	2

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)	Corrected Infiltration Rate, I_c (inches/hr)	Infiltration Rate with FOS, I_f (inches/hr)
1	30.00	31.20	44.52	30.00	64.80	51.48	13.32	58.14	0.89	0.44	0.22
2	30.00	31.20	43.44	60.00	64.80	52.56	12.24	58.68	0.81	0.40	0.20
3	30.00	31.20	48.96	90.00	64.80	47.04	17.76	55.92	1.23	0.61	0.31
4	30.00	31.20	44.52	120.00	64.80	51.48	13.32	58.14	0.89	0.44	0.22
5	30.00	31.20	43.56	150.00	64.80	52.44	12.36	58.62	0.82	0.41	0.20
6	30.00	31.20	43.44	180.00	64.80	52.56	12.24	58.68	0.81	0.40	0.20
7	30.00	31.20	42.84	210.00	64.80	53.16	11.64	58.98	0.76	0.38	0.19
8	30.00	31.20	41.76	240.00	64.80	54.24	10.56	59.52	0.69	0.34	0.17
9	30.00	31.20	42.36	270.00	64.80	53.64	11.16	59.22	0.73	0.36	0.18
10	30.00	31.20	44.28	300.00	64.80	51.72	13.08	58.26	0.87	0.43	0.22
11	30.00	31.20	42.96	330.00	64.80	53.04	11.76	58.92	0.77	0.39	0.19
12	30.00	31.20	42.84	360.00	64.80	53.16	11.64	58.98	0.76	0.38	0.19

Recommended Design Infiltration Rate (inches/hr)   0.17

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Santa Ana Regional Water Quality Control Board Technical Guidance Document, Appendix VII, Example VII.1.

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg})))$

Porosity conversion calculations are based on the method provided in Caltrans California Test 750.

$C = n * (1 - (O / (2 * r))^2) + (I / (2 * r))^2$

$I_c = I_t * C$

$I_f = I_C * F$

Plate No.

2

Estimated Infiltration Rate from Percolation Test Data, PT-01 (2)

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-01 (2)
Personnel	Jay Burnham
Presoak Date	7/12/2019
Test Date	7/12/2019

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	120
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.40
Porosity of Gravel, n	0.48
Porosity Correction Factor, C	0.50
Factor of Safety (FOS), F	2

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)	Corrected Infiltration Rate, I_c (inches/hr)	Infiltration Rate with FOS, I_f (inches/hr)
1	25.00	78.00	113.40	25.00	42.00	6.60	35.40	24.30	6.46	3.23	1.61
2	25.00	78.00	108.48	50.00	42.00	11.52	30.48	26.76	5.09	2.54	1.27
3	10.00	78.24	94.80	60.00	41.76	25.20	16.56	33.48	5.60	2.80	1.40
4	10.00	76.80	93.60	70.00	43.20	26.40	16.80	34.80	5.48	2.74	1.37
5	10.00	78.00	93.84	80.00	42.00	26.16	15.84	34.08	5.27	2.63	1.32
6	10.00	78.00	93.36	90.00	42.00	26.64	15.36	34.32	5.07	2.53	1.27
7	10.00	78.72	93.60	100.00	41.28	26.40	14.88	33.84	4.98	2.49	1.24
8	10.00	78.00	93.36	110.00	42.00	26.64	15.36	34.32	5.07	2.53	1.27

Recommended Design Infiltration Rate (inches/hr)   1.27

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Santa Ana Regional Water Quality Control Board Technical Guidance Document, Appendix VII, Example VII.1.

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg})))$

Porosity conversion calculations are based on the method provided in Caltrans California Test 750.

$C = n * (1 - (O / (2 * r))^2) + (I / (2 * r))^2$

$I_c = I_t * C$

$I_f = I_C * F$

Plate No.
3

Estimated Infiltration Rate from Percolation Test Data, PT-02 (2)

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-02 (2)
Personnel	Jay Burnham
Presoak Date	7/12/2019
Test Date	7/12/2019

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	120
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.40
Porosity of Gravel, n	0.48
Porosity Correction Factor, C	0.50
Factor of Safety (FOS), F	2

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)	Corrected Infiltration Rate, I_c (inches/hr)	Infiltration Rate with FOS, I_f (inches/hr)
1	25.00	72.00	106.20	25.00	48.00	13.80	34.20	30.90	4.99	2.49	1.25
2	25.00	72.00	103.20	50.00	48.00	16.80	31.20	32.40	4.35	2.17	1.09
3	10.00	72.24	88.20	60.00	47.76	31.80	15.96	39.78	4.58	2.29	1.14
4	10.00	72.00	87.60	70.00	48.00	32.40	15.60	40.20	4.44	2.21	1.11
5	10.00	73.20	87.48	80.00	46.80	32.52	14.28	39.66	4.11	2.05	1.03
6	10.00	73.44	87.84	90.00	46.56	32.16	14.40	39.36	4.18	2.09	1.04
7	10.00	72.00	86.52	100.00	48.00	33.48	14.52	40.74	4.08	2.04	1.02
8	10.00	72.00	86.40	110.00	48.00	33.60	14.40	40.80	4.04	2.02	1.01

Recommended Design Infiltration Rate (inches/hr)	1.01

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Santa Ana Regional Water Quality Control Board Technical Guidance Document, Appendix VII, Example VII.1.

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg})))$

Porosity conversion calculations are based on the method provided in Caltrans California Test 750.

$C = n * (1 - (O / (2 * r))^2) + (I / (2 * r))^2$

$I_c = I_t * C$

$I_f = I_C * F$

Plate No.

4

Infiltration Rate versus Time, PT-01

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-01
Personnel	Catherine Nelson
Presoak Date	6/10/2019
Test Date	6/10/2019

Infiltration Rate Versus Time

Infiltration Rate versus Time, PT-02

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-02
Test Location	
Personnel	Catherine Nelson
Presoak Date	6/10/2019
Test Date	6/10/2019

Infiltration Rate Versus Time

Infiltration Rate versus Time, PT-01

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-01 (2)
Personnel	Jay Burnham
Presoak Date	7/12/2019
Test Date	7/12/2019

Infiltration Rate Versus Time chart showing Infiltration Rate (in/hr) versus Elapsed Time (min) for PT-01 (2).

Infiltration Rate versus Time, PT-01

Project Name	20.6-acre development
Project Number	19-81-173-01
Test Number	PT-02 (2)
Personnel	Jay Burnham
Presoak Date	7/12/2019
Test Date	7/12/2019

Infiltration Rate Versus Time