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# Appendix C

# 2019 Updated Geotechnical Report

# SNOWDROP ROAD PROJECT

FOR

**Assessment District 2018-1**
**UNINCOPORATED RANCHO CUCAMONGA, CALIFORNIA**

**PROJECT NO.: 30.30.0009**

# UPDATED GEOTECHNICAL INVESTIGATION REPORT

## SNOWDROP ROAD IMPROVEMENT PROJECT

**SNOWDROP ROAD, SANTINA DRIVE, ARCHIBALD AVENUE AND HAVEN AVENUE**
**ASSESSMENT DISTRICT 2018-1**

CITY OF RANCHO CUCAMONGA, SAN BERNARDINO COUNTY, CALIFORNIA

*CONVERSE PROJECT NO. 18-81-316-02*



*Prepared For:*

**SAN BERNARDINO COUNTY SPECIAL DISTRICTS DEPARTMENT**
222 Hospitality Lane, 2nd Floor
San Bernardino, CA 92415

*Presented By:*

**CONVERSE CONSULTANTS**
2021 Rancho Drive, Suite 1
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October 16, 2019

October 16, 2019

Mr. Siva Sivapalan, PE
Project Manager, BCE II
San Bernardino County Special Districts Department (SBCSDD)
222 Hospitality Lane, Second Floor
San Bernardino, CA 92415

Subject: **UPDATED GEOTECHNICAL INVESTIGATION REPORT**
**Snowdrop Road Improvement Project**
Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
Assessment District 2018-1
City of Rancho Cucamonga, San Bernardino County, California
Converse Project No. 18-81-316-02

Dear Mr. Sivapalan:

Converse Consultants (Converse) is pleased to submit this updated geotechnical investigation report for the design and construction of the Snowdrop Road Improvement Project, along Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue, in the City of Rancho Cucamonga, San Bernardino County, California. This report was prepared in accordance with our revised proposal dated October 1, 2019 and authorization to proceed by email on October 3, 2019.

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

We appreciate the opportunity to be of service to the San Bernardino County Special Districts Department (SBCSDD). Should you have any questions, please do not hesitate to contact us at 909-796-0544.

**CONVERSE CONSULTANTS**

Hashmi Quazi, PhD, GE, PE
Principal Engineer

Dist: 3/Addressee
HSQ/RLG/ZA/kvg

## PROFESSIONAL CERTIFICATION

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

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

REGISTERED GEOLOGIST
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## APPENDICES

# 1.0 INTRODUCTION

This updated geotechnical investigation report was prepared to provide design and construction of the proposed improvements for the Snowdrop Road Improvements project, in the City of Rancho Cucamonga, San Bernardino County, California. The approximate location of the proposed street improvements is shown in Figure No. 1, *Approximate Project Area Map.*

The purposes of this investigation were to evaluate the nature and pertinent engineering properties of the subsurface materials along the project limits and to provide recommendations regarding general site grading, flexible pavement design, storm drain design parameters, retaining wall inter-block design parameters and construction.

This report is prepared for the project site described herein and is intended for use solely by San Bernardino County Special Districts Department and their designated project team. If provided to other parties, this report be used for information on factual data only. Other parties should be responsible for making their own interpretations of the data contained in this report.

# 2.0 PROJECT BACKGROUND/DESCRIPTION

Based on the County's Work Order No. 18407-904 dated December 12, 2018, Converse Consultants prepared a geotechnical investigation report (Converse, 2019) for design and construction of approximately 300 linear feet of Snowdrop Road beginning approximately 0.4 miles west of the northern termination of Haven Avenue and continuing 300 feet to the west. Plans provided by you (prepared by Webb Associates) on September 17, 2019 indicate 2 storm drains about 15 feet deep have been added to the project. The CHJ's report (CHJ, 2014) was prepared for approximately 2-mile of roadway improvements. It included 9 retaining walls (inter block) and 12 storm drains. The size of the storm drains varies from 18" to 60".

At present, the roadway is paved and unpaved. We understand that undocumented fill has been placed in various locations to rebuild the roadway over time.

Based on the referenced improvement plans by Albert A. Webb Associates, the project consists of design and construction of approximately 2-miles of roadway improvements beginning at a portion of the north end Archibald Avenue and continuing east along Snow Drop east to a portion of the north end Haven Avenue. It includes 12 storm drains, up to approximately 15 feet deep, 5 retaining walls (inter block), up to approximately 18 feet high. The table below summarizes the proposed facilities to be constructed.


# Approximate Project Area Map

| | | |
|---|---|---|
| Project: | Snowdrop Road Improvement Project | |
| Location: | Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue | Project No |
| | City of Rancho Cucamonga, San Bernardino County, California | 18-81-316-02 |
| For: | San Bernardino County Special Districts Department | |

**Converse Consultants**

FIGURE NO.
**1**

**Table No. 1, Proposed Facilities to be constructed**

| Type of Facility | Length (ft) | Depth (ft) or Height (ft) | Stations |
|---|---|---|---|
| Archibald Ave. | 1,800 | N.A. | 2+00 to 20+00 |
| Santina Dr. | 2,000 | N.A. | 20+00 to 40+00 |
| Snow Drop Dr. | 4,500 | N.A. | 40+00 to 85+00 |
| Haven Ave. | 2.300 | N.A. | 85+00 to 108+50 |
| Retaining Wall No. 1 | 110 | 10 | 36+80 to 37+90 |
| Retaining Wall No. 2 | 190 | 12 | 41+00 to 42+90 |
| Retaining Wall No. 3 | 455 | 11 | 43+90 to 48+45 |
| Retaining Wall No. 4 | 205 | 18 | 50+55 to 52+60 |
| Retaining Wall No. 6 | 80 | 6 | 30+10 to 30+90 |
| Storm Drain B | 25 | 6 | N.A. |
| Storm Drain C | 25 | 5 | N.A. |
| Storm Drain D | 80 | 7 | N.A. |
| Storm Drain E | 15 | 4 | N.A. |
| Storm Drain F | 110 | 7 | N.A. |
| Storm Drain G | 35 | 8 | N.A. |
| Storm Drain H | 70 | 11 | N.A. |
| Storm Drain I | 25 | 7 | N.A. |
| Storm Drain J | 175 | 15 | N.A. |
| Storm Drain K | 140 | 14 | N.A. |
| Storm Drain M | 70 | 6 | N.A. |
| Storm Drain N | 75 | 9 | N.A. |

(N.A. = not applicable)

Cut slopes up to approximately 30 feet high and fill slopes up to approximately 25 feet high. Cut slopes are proposed at a maximum slope ratio of 1.5:1 horizontal to vertical (H:V) and fill slopes at a maximum slope ratio of 2:1 H:V. At present the roadway is paved and unpaved. The roadway is bounded on both sides by vacant land with trees and shrubs. We understand that undocumented fill has been placed to rebuild the roadway over time.

# 3.0 SCOPE OF WORK

The scope of Converse's investigation included the tasks described in the following sections.

## *3.1 Project Set-up*

The project set-up consisted of the following tasks.

- Conducted a site reconnaissance with you and ensured that backhoe and personnel access to all test pit and seismic refraction line locations were available.
- Notified Underground Service Alert (USA) at least 48 hours prior to trenching to clear the locations of conflict with underground utilities.
- Engaged a California-licensed backhoe operator to trench the test pits.
- Engaged a California-licensed geophysicist to perform a seismic refraction survey.

## *3.2 Subsurface Exploration*

Six exploratory test pits (TP-01 through TP-06) were excavated to investigate the subsurface conditions along the proposed road alignment on October 7, 2019. The test pits were excavated to depths of approximately 6.0 feet to 10.0 feet below the existing ground surface (bgs). Converse also previously drilled four exploratory borings (Converse, 2019) to depths of approximately 11.0 feet to 16.5 feet below the existing ground surface (bgs) on the site in December 2018, as reported in the referenced geotechnical report.

CHJ Consultants (CHJ) previously drilled sixteen exploratory borings and excavated four test pits on the site in 2014, as reported in the referenced geotechnical report.

Based on our initial review of the project site geology we anticipated encountering shallow bedrock within portions of the road alignment. Since this shallow bedrock may impact the design and excavatability of the subsurface material, we retained Terra Geosciences to conduct seismic refraction surveys (attached in Appendix C), on October 4 and 6, 2019, with four seismic refraction lines in order to obtain a velocity profile of the subsurface materials in various locations of suspect shallow bedrock that may affect construction.

The seismic refraction survey investigated the subsurface by generating arrival time and offset distance information to determine the path and velocity of an elastic disturbance in the ground. Shot, hammer, weight drop or some comparable method of putting impulsive energy into the ground creates the disturbance. Detectors are laid out in a line at regular intervals to measure the first arrival energy and the time of its arrival. The data was plotted in time-distance graphs to calculate velocity of and depth to layers. The velocities indicated in the report were as follows: V1 layer from about 1 foot to 12 feet, 1,238 to 1,618 ft/sec; V2 layer from about 5 feet to 31 feet, 1,801 to 2,700 ft/sec; and V3 layer from about 7 feet to 31 feet, 2,346 to 4,714 ft/sec.

The approximate locations of the exploratory test pits, borings and seismic refraction survey lines by Converse as well as the exploratory borings and test pits by CHJ are shown on Figures No. 2a through 2k *Approximate Boring, Test Pit and Seismic Refraction Locations Map*. A detailed discussion of the subsurface exploration is presented in Appendix A, *Field Exploration*.

### *3.3 Laboratory Testing*

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

- Expansion Index (ASTM D4829)
- Sand equivalent (ASTM D2419)
- Soil corrosivity (California Test Methods 643, 422, and 417)
- Grain size analysis (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.

### *3.4 Analysis and Report Preparation*

Data obtained from the present field exploration and laboratory testing program as well as previous field exploration and laboratory testing by Converse and CHJ was assembled and evaluated. Geotechnical analyses of the compiled data were performed, followed by the preparation of this updated report to present our findings, conclusions, and recommendations for the proposed project.

## 4.0 SUBSURFACE CONDITIONS

The various elements of the subsurface conditions observed by Converse are presented below.

### *4.1 Subsurface Profile*

Based on the current and previous exploratory trenches and borings as well as laboratory test results, the subsurface soil at the site consisted of artificial fill, topsoil, alluvial soils, and bedrock with approximate thicknesses as shown in the following table.

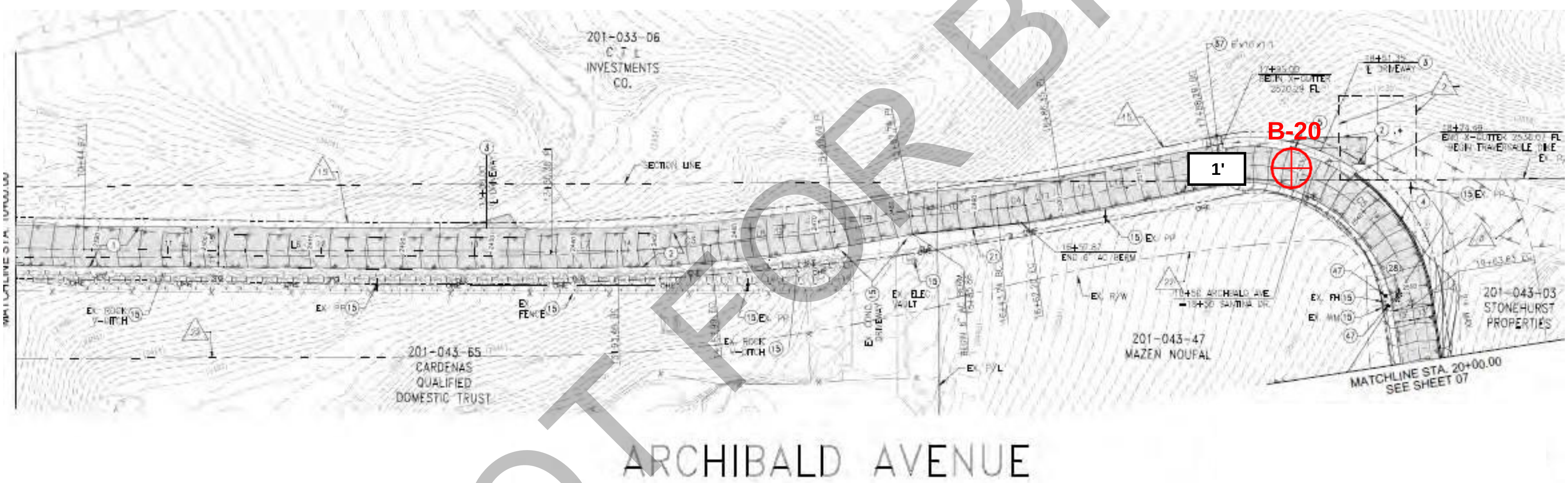
## EXPLANATION

**B-20** — Number and Approximate Location of Exploratory Boring (CHJ)

**1'** — Estimated Depth of Over-Excavation (ft)

N

120'






# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvem ent Project

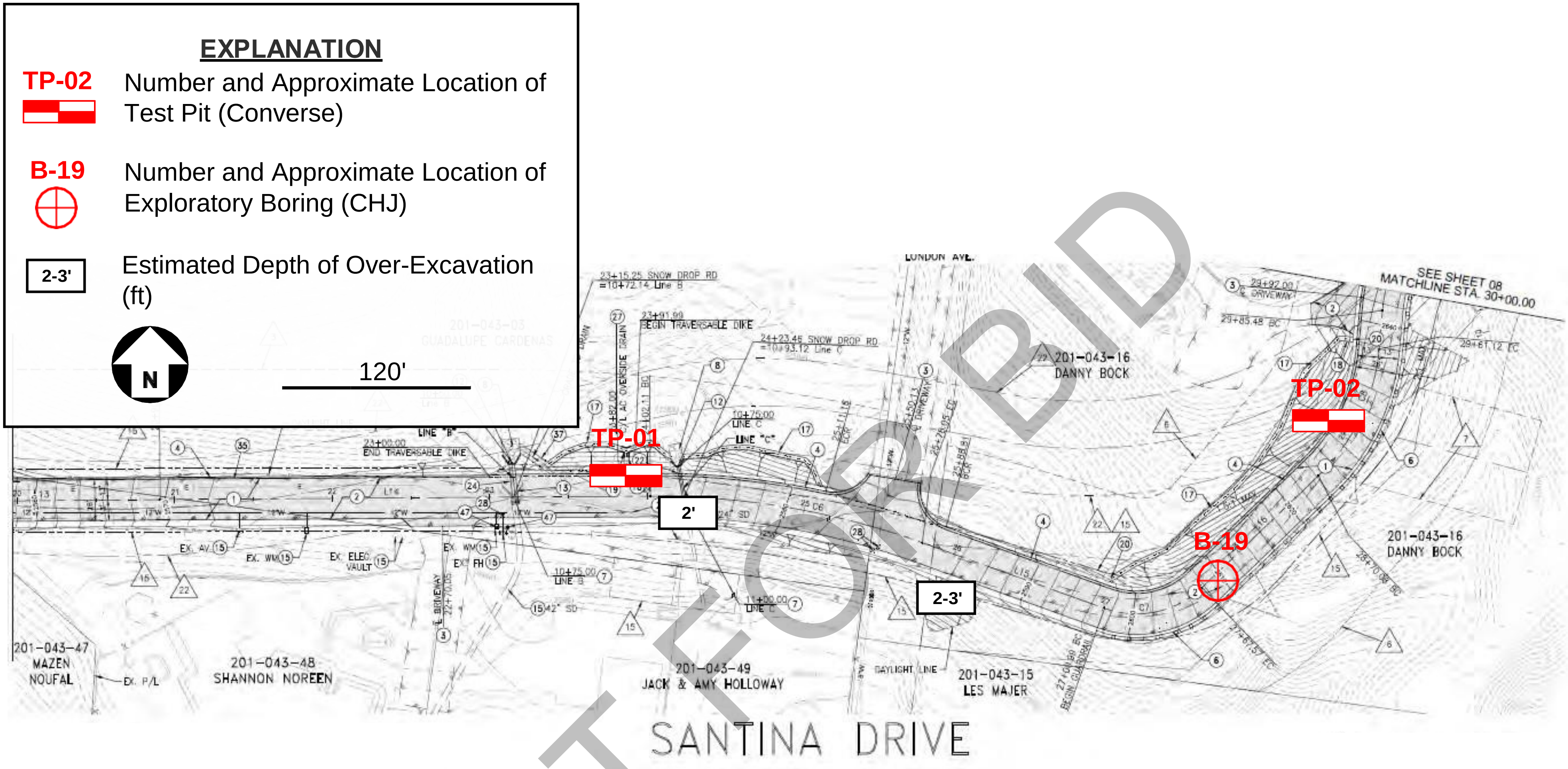
Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California

For: San Bernardino County Special Districts Department

Project No.
18-81-316-02

**Converse Consultants**

Figure No.
**2a**





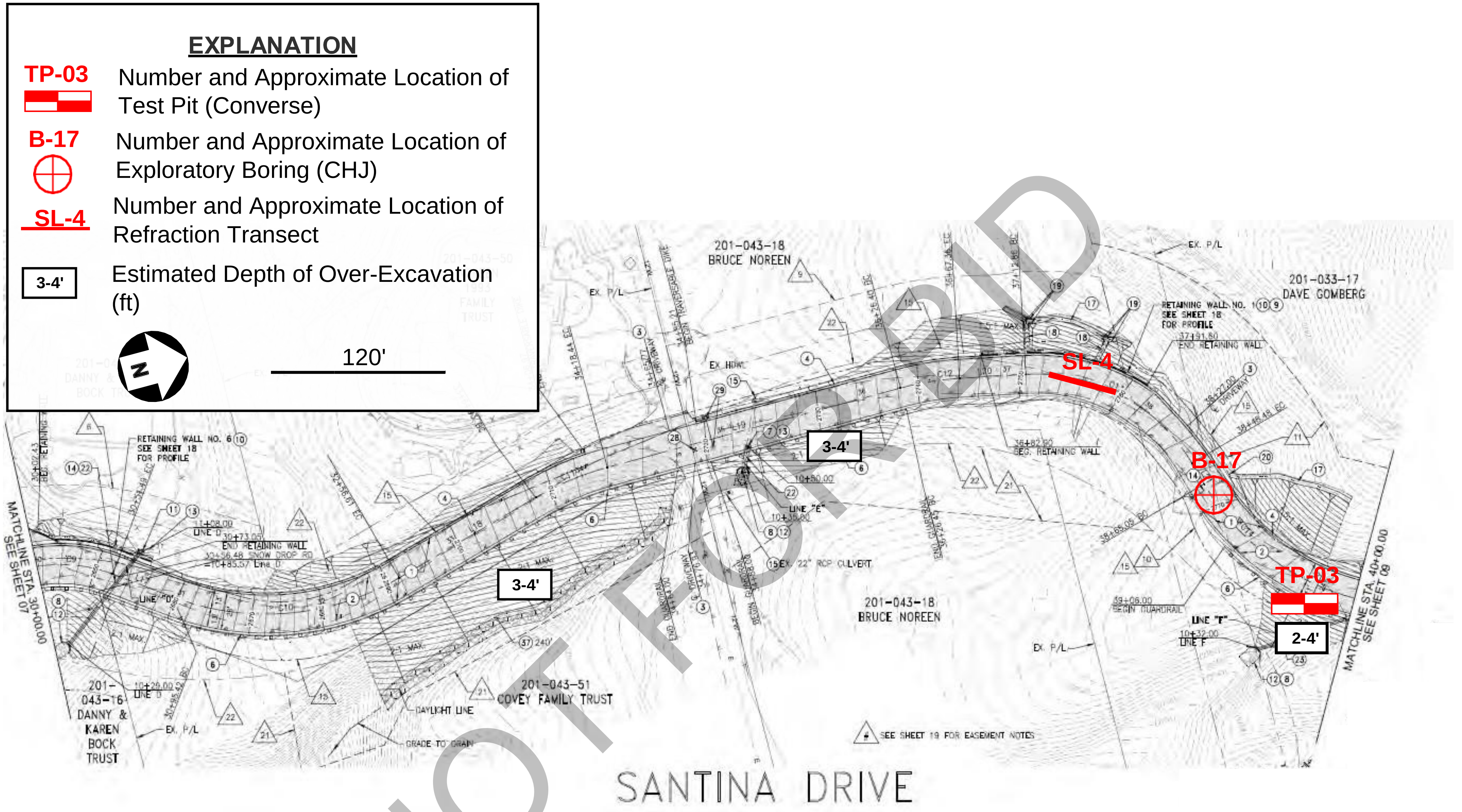

# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvem ent Project
Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No.
18-81-316-02

**Converse Consultants**

Figure No.
**2b**



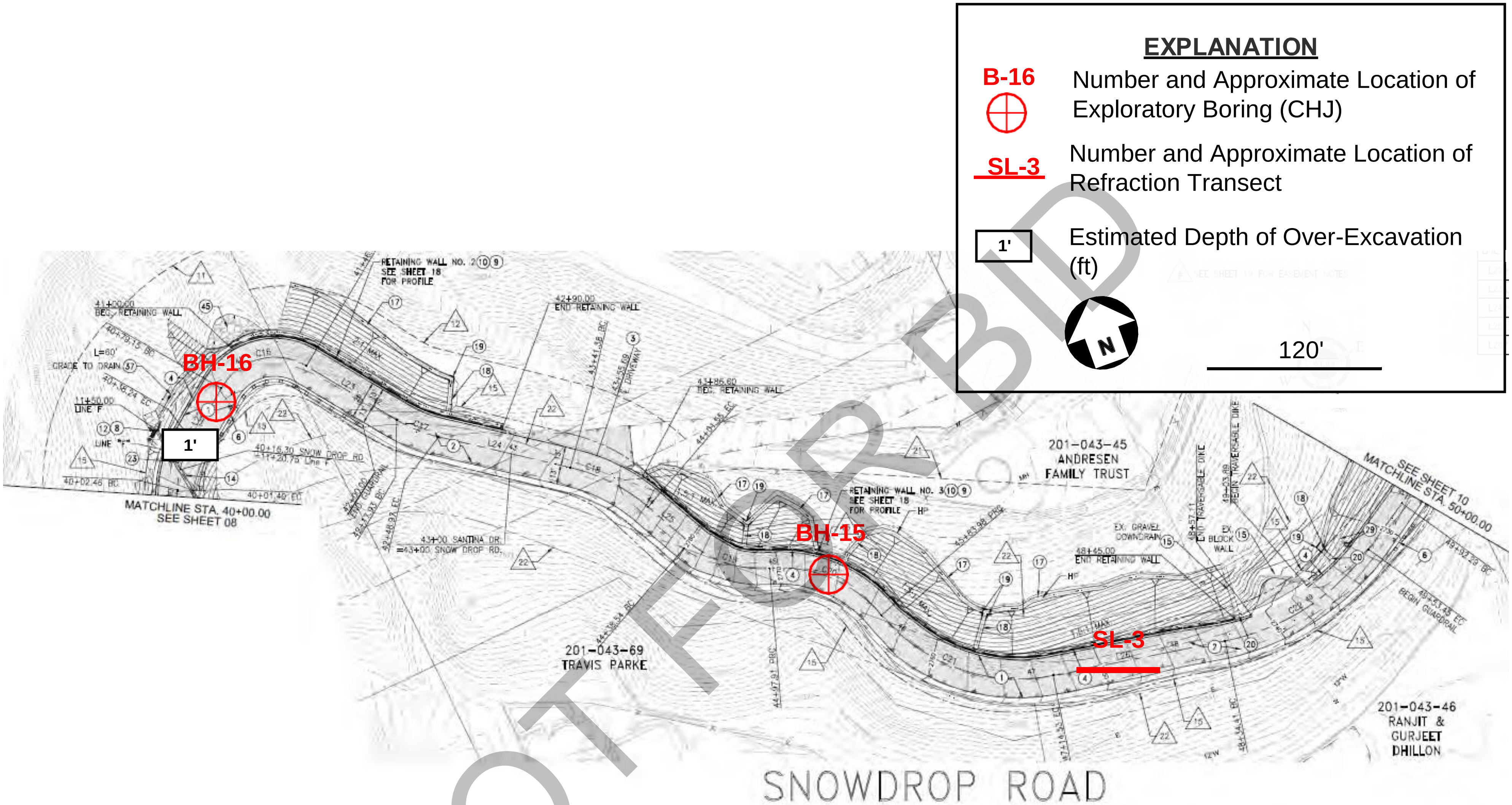

# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvem ent Project
Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No. 18-81-316-02

Converse Consultants

Figure No. 2c






# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

| | |
|---|---|
| Project: | Snowdrop Road Improvem ent Project |
| Location: | Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue |
| | City of Rancho Cucamonga, San Bernardino County, California |
| For: | San Bernardino County Special Districts Department |

Project No.
18-81-316-02

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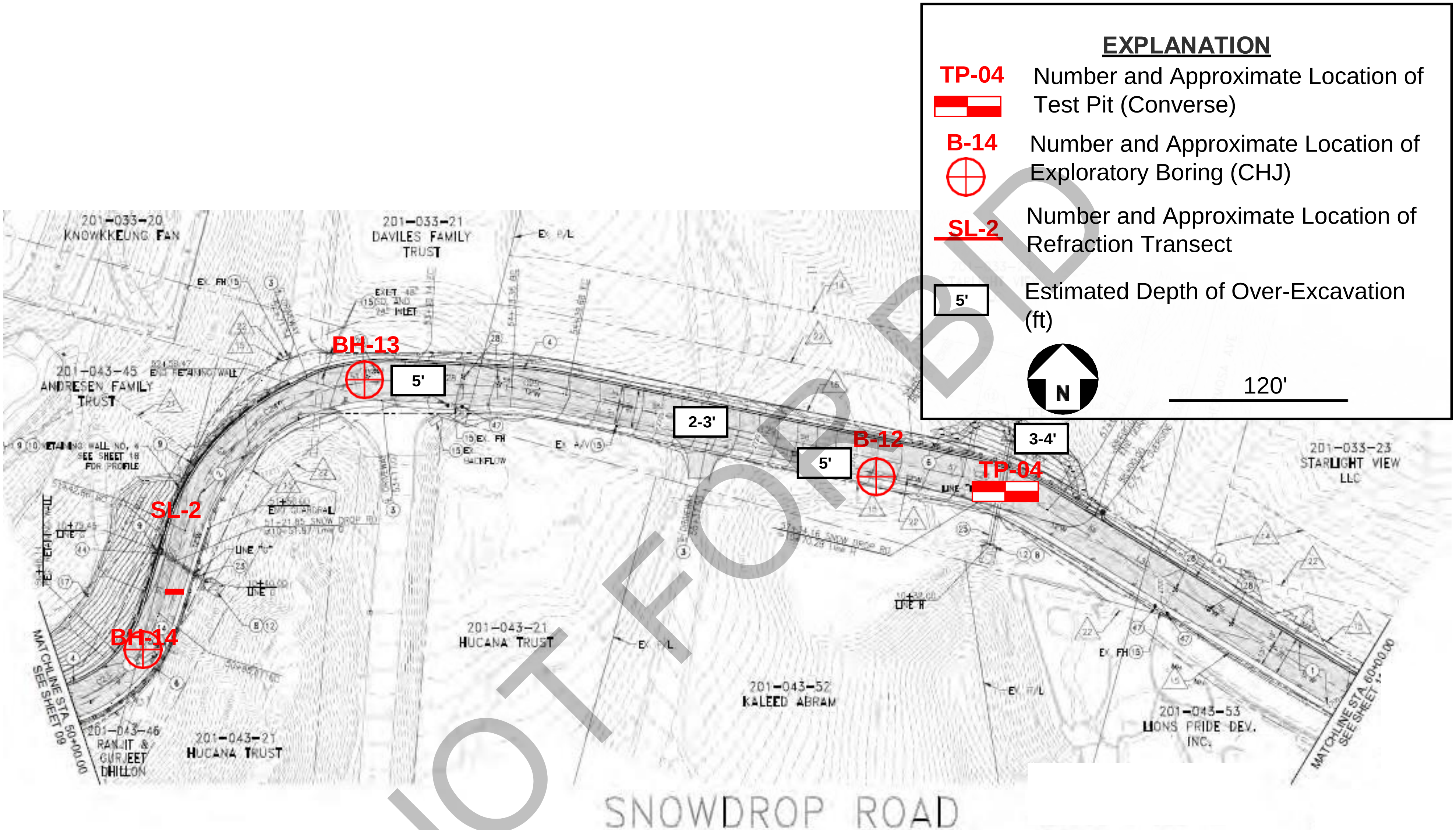
Figure No.
**2d**






## Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvem ent Project
Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

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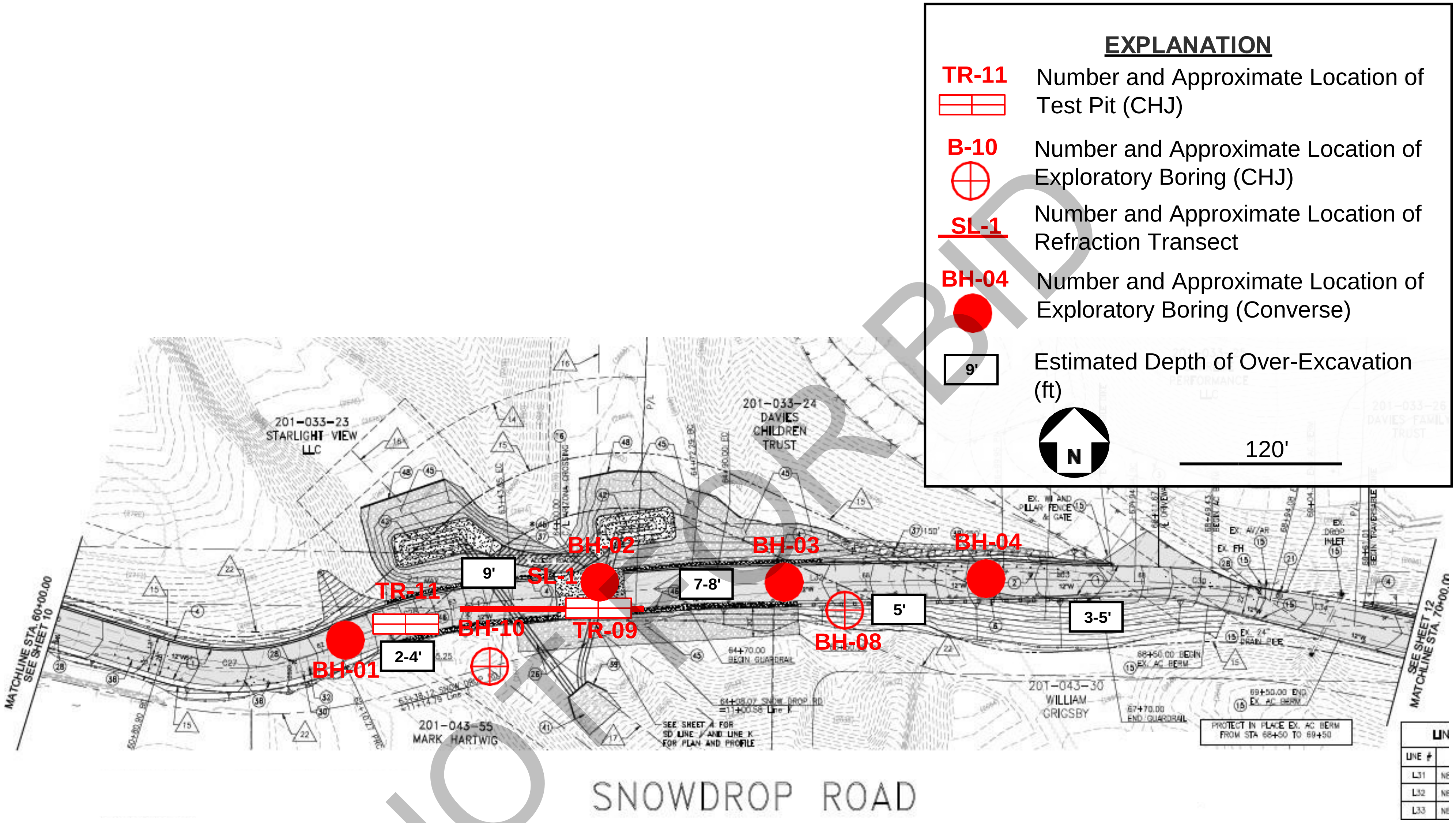
Project No.
18-81-316-02

Figure No.
**2e**






## Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvement Project

Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California

For: San Bernardino County Special Districts Department

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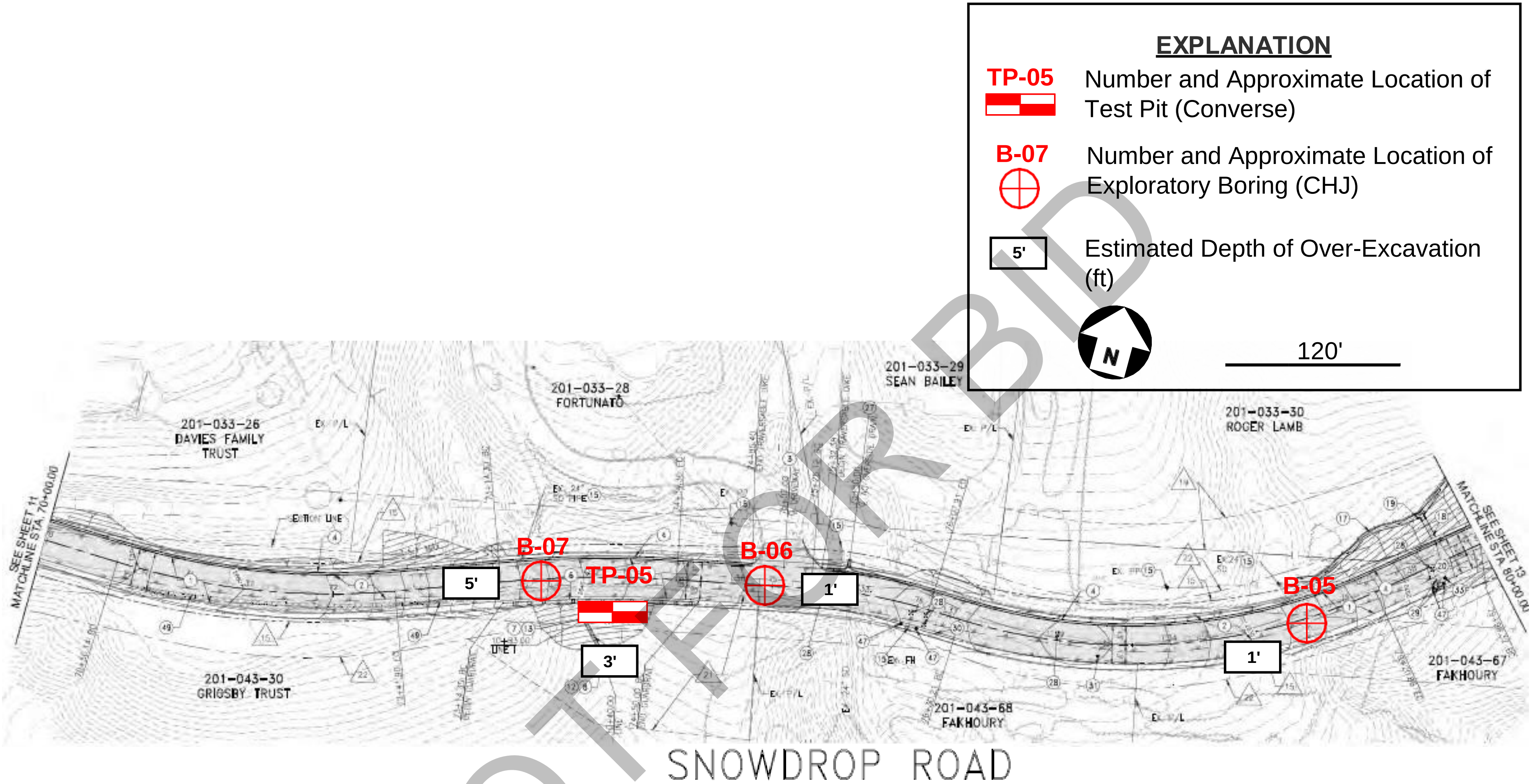
**erse Consultants**

Project No.
18-81-316-02

Figure No.
**2f**






# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

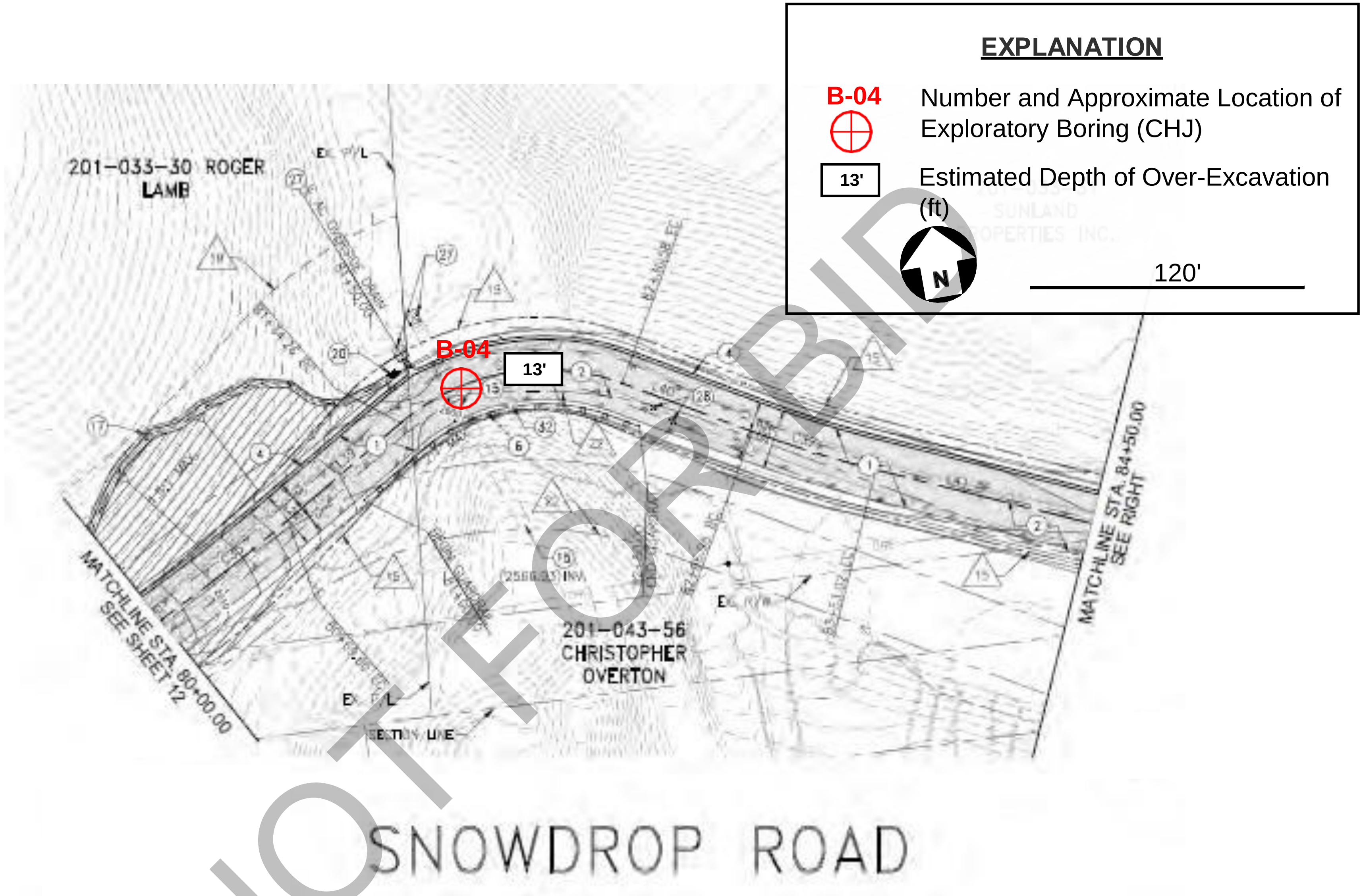
| | |
|---|---|
| Project: | Snowdrop Road Improvem ent Project |
| Location: | Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue |
| | City of Rancho Cucamonga, San Bernardino County, California |
| ~~For:~~ | ~~San Bernardino County Special Districts Department~~ |

Project No.
18-81-316-02

Converse Consultants

Figure No.

2g




## Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvem ent Project
Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No.
18-81-316-02

**Converse Consultants**

Figure No.
**2h**

## EXPLANATION

**B-03** Number and Approximate Location of Exploratory Boring (CHJ)

**1-2'** Estimated Depth of Over-Excavation (ft)

N

120'


NOT FOR BID

# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

| | |
|---|---|
| Project: | Snowdrop Road Improvem ent Project |
| Location: | Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue |
| | City of Rancho Cucamonga, San Bernardino County, California |
| For: | San Bernardino County Special Districts Department |

Project No.
18-81-316-02

**Converse Consultants**

Figure No.
**2i**

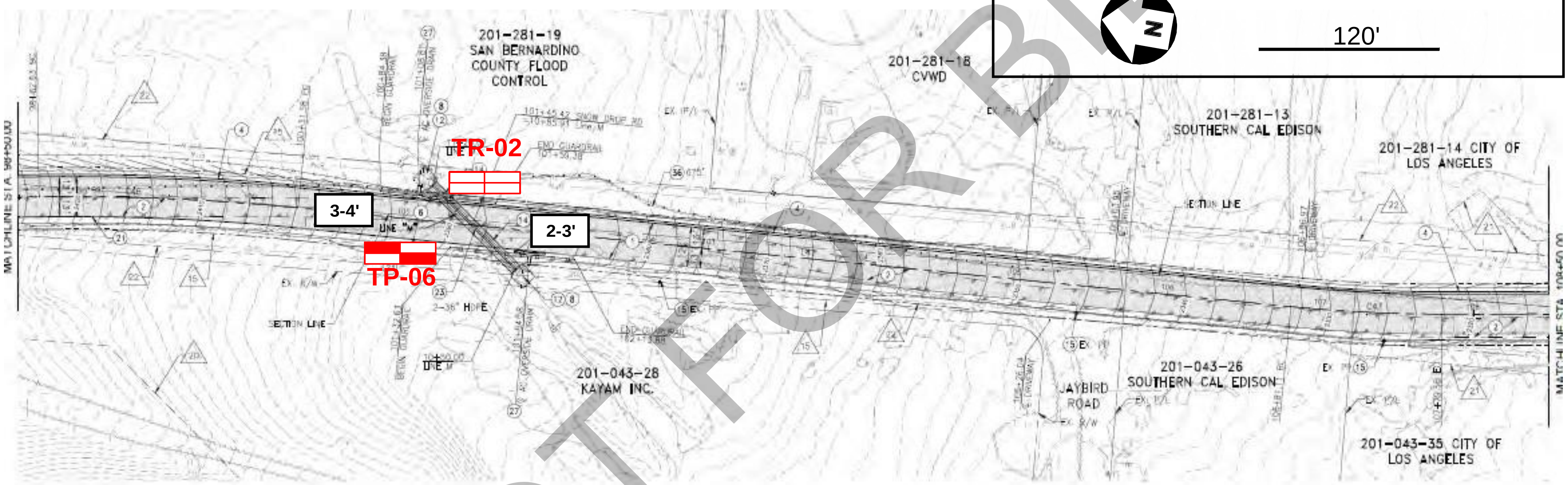
**EXPLANATION**

**TP-06** Number and Approximate Location of Test Pit (Converse)

**TR-02** Number and Approximate Location of Test Pit (CHJ)

**3-4'** Estimated Depth of Over-Excavation (ft)

120'




HAVEN AVENUE



# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

Project: Snowdrop Road Improvem ent Project

Location: Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California

For: San Bernardino County Special Districts Department

Project No.
18-81-316-02

Converse Consultants

Figure No.
**2j**

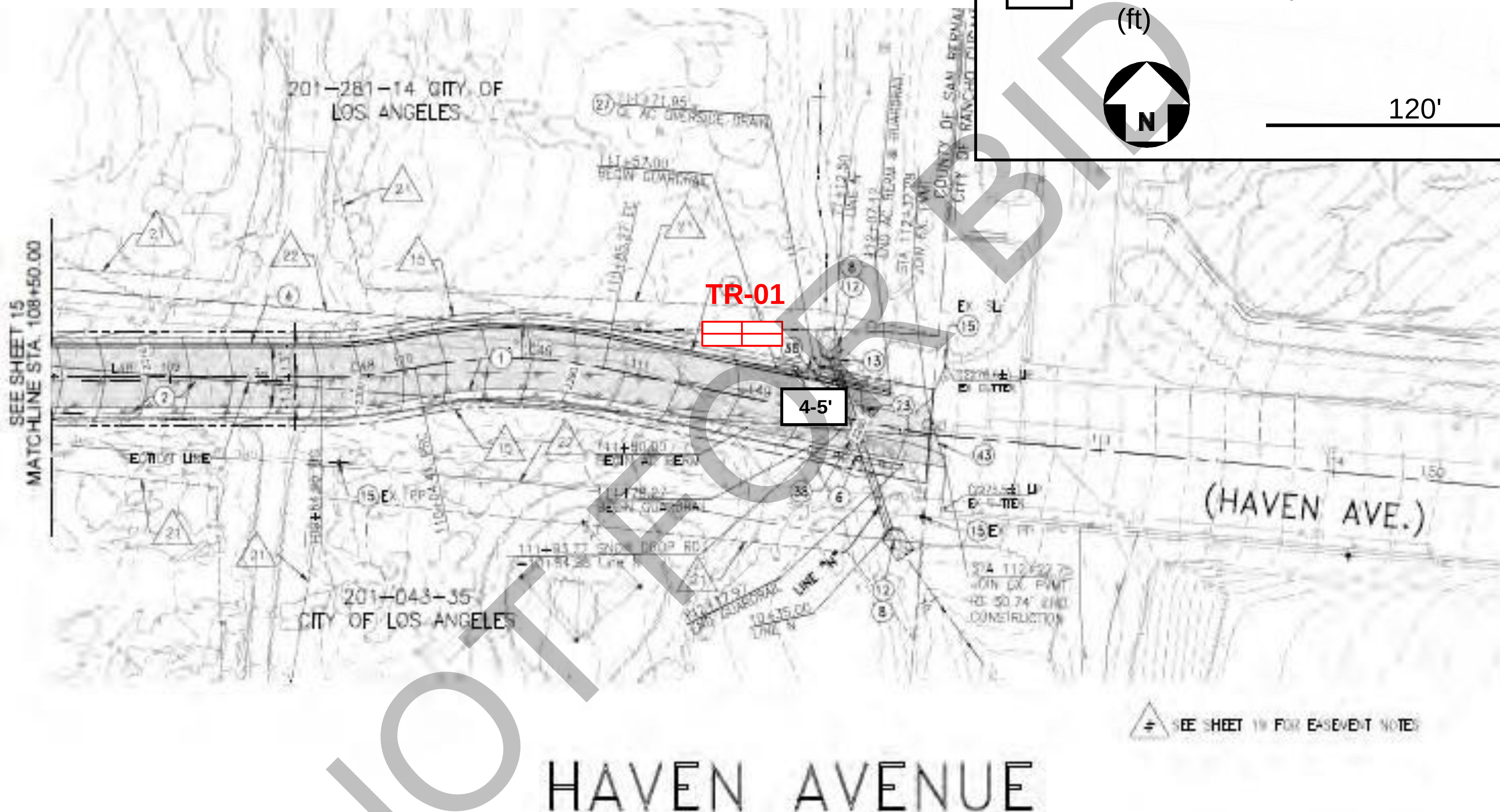
## EXPLANATION

**TR-01** Number and Approximate Location of Test Pit (CHJ)

**4-5'** Estimated Depth of Over-Excavation (ft)

N

120'






# Approximate Boring, Test Pit, and Seismic Refraction Locations Map

| | |
|---|---|
| Project: | Snowdrop Road Improvement Project |
| Location: | Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue<br>City of Rancho Cucamonga, San Bernardino County, California |
| For: | San Bernardino County Special Districts Department |

Project No.
18-81-316-02

**Converse Consultants**

Figure No.
**2k**

**Table No. 2, Subsurface Profile (Current Converse Test Pits)**

| Subsurface | Test Pit | | | | | |
|---|---|---|---|---|---|---|
| | TP-01 | TP-02 | TP-03 | TP-04 | TP-05 | TP-06 |
| **Asphalt Concrete / Aggregate Base** | Not Encountered | Not Encountered | Not Encountered | 3.0” AC / No AB | Not Encountered | Not Encountered |
| **Artificial Fill** | Not Encountered | Not Encountered | Not Encountered | Not Encountered | Not Encountered | Not Encountered |
| **Alluvial Fan** | 0.0’ to 10.0’ | 0.0’ to 1.0’ | 0.0’ to 10.0’ | 0.0’ to 6.0’ | 0.0’ to 1.0’ | 0.0’ to 6.0’ |
| **Bedrock** | Not Encountered | 1.0’ to 6.0’ | Not Encountered | 6.0'-10.0’ | 1.0’ to 6.0’ | Not Encountered |

For a detailed description of the subsurface materials encountered in the current exploratory test pits, see Drawing Nos. A-2 through A-7, *Log of Test Pits*, in Appendix A, Field Exploration.

**Table No. 3, Subsurface Profile (Previous Converse Borings)**

| Subsurface | Boring | | | |
|---|---|---|---|---|
| | BH-01 | BH-02 | BH-03 | BH-04 |
| **Asphalt Concrete / Aggregate Base** | Not Encountered | Not Encountered | Not Encountered | 3.0” AC / No AB |
| **Artificial Fill** | Surface to 2.5’ | Surface to 7.5’ | Surface to 6.0’ | 0.25’ to 2.5’ |
| **Alluvium** | 2.5’ to 10.0’ | 7.5’ to 15.0’ | 6.0’ to 11’ | 2.5’ to 16.5’ |
| **Bedrock** | 10.0’ to 15.4’ | 15.0’ to 15.9’ | Refusal at 11’ on Bedrock | Not Encountered |

For a detailed description of the subsurface materials encountered in the previous exploratory borings, see Drawing Nos. A-2 through A-5, *Log of Borings*, in Appendix A-1, Field Exploration.

### *4.2 Subsurface Variations*

Based on the results of our subsurface exploration, variations in the continuity and nature of subsurface conditions should be anticipated. Due to the variations in the nature and depositional characteristics of earth materials, care should be exercised in extrapolating or interpolating subsurface soil conditions between or beyond the exploration location.

### *4.3 Excavatability*

Based on the attached seismic refraction survey investigation, by Terra Geosciences (Appendix C) the subsurface bedrock and soil materials at the project are expected to be excavatable to proposed vertical depths of construction, of up to approximately 15 feet to 20 feet, by conventional heavy-duty earth moving and trenching equipment. <u>Based on trenching and drilling in some areas, trench excavations may be difficult due to the presence of gravel, cobbles and possible boulders within some of the alluvial soils and local areas within the bedrock, below a depth of approximately 6 feet, near station 29+00, and 11 feet, near station 65+00.</u>

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.

### *4.4 Groundwater*

Groundwater was not encountered during this investigation, to the maximum explored depth of 10.0 feet bgs or in previous investigations by Converse to depths of 16.5 feet bgs and CHJ to depths of 31.5 feet bgs

Regional databases were reviewed to estimate expected groundwater conditions in the vicinity of the project site. No relevant groundwater data was found in either the Geotracker (SWRCB, 2019) or National Water Information System (USGS, 2019) databases.

Based on the groundwater data reviewed, the current and historical groundwater levels are deeper than 50 feet bgs. Dewatering is not expected to be required during the construction of the project. There is a possibility of water seepage within fractures in the bedrock or along alluvial soil and bedrock contacts due to seasonal precipitation.

## 5.0 GEOLOGIC SETTING

The regional and local geology are discussed in the following subsections.

### *5.1 Regional Geology*

The project is located at the base of the San Gabriel Mountains within the northwestern boundary of the Peninsular Ranges Geomorphic Province of Southern California.

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

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

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

### *5.2 Local Geology*

The project is located at the base of the San Gabriel Mountains. Regional mapping (Morton and Miller, 2006) indicates that the project is underlain by Holocene and Pleistocene-aged alluvial fan deposits and metamorphic bedrock consisting of granitic gneiss which is generally moderately fractured and moderately hard to hard. The alluvium and alluvial fan deposits consist of unconsolidated to slightly consolidated sand, silty sand and gravelly sand with cobbles and boulders. Non engineered fill also exists along the existing roadways which consist of silty sand and clayey sand with some gravel and cobbles.

## 6.0 FAULTING AND SEISMICITY

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

### *6.1 Faulting*

The project has two potions of the Archibald Avenue and one portion of Haven Avenue alignments where the active Cucamonga fault crosses them, as stated in the referenced geotechnical report by CHJ. There are no known active faults projecting toward or extending across The Snow Drop alignment portions the project. The potential for surface rupture resulting from the movement of the Cucamonga fault is a possibility on only the Archibald Avenue and Haven Avenue alignments. Since the proposed retaining walls are not known within active fault areas, the chance of surface rupture is considered low.

The project 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. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the project.

### *6.2 CBC Seismic Design Parameters*

Mapped acceleration parameters based on the 2016 California Building Code and ASCE 7-10 and generalized site coordinates 34.1721°N latitude and 117.5861°W longitude are provided in the following table. These parameters were determined using the ATC Hazards online calculator.

**Table No. 4, CBC Seismic Parameters**

| Seismic Parameters | |
|---|---|
| Site Coordinates | 34.1721 N, 117.5861 W |
| Site Class | D |
| Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_s$ | 3.117g |
| Mapped 1-second Spectral Response Acceleration, $S_1$ | 1.133g |
| 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, $S_{MS}$ | 3.117g |
| MCE 1-second period Spectral Response Acceleration, $S_{M1}$ | 1.7g |
| Design Spectral Response Acceleration for short period $S_{DS}$ | 2.078g |
| Design Spectral Response Acceleration for 1-second period, $S_{D1}$ | 1.133g |
| Maximum Peak Ground Acceleration, $PGA_M$ | 1.217g |

### *6.3 Secondary Effects of Seismic Activity*

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

***Surface Fault Rupture:*** The potential for surface rupture resulting from the movement of the Cucamonga fault is a possibility on only the Archibald Avenue and Haven Avenue alignments but would only be limited to rupture within the asphalt paving. There are no known active faults projecting toward or extending through the Snow Drop Road portion of the project, where the retaining walls are proposed, therefore surface rupture in these is considered low.

***Liquefaction:*** Liquefaction is defined as the phenomenon in which a cohesionless soil mass suffers a substantial reduction in its shear strength due to the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction generally occurs in submerged granular soils and non-plastic silts located within 50 feet of the ground surface during or after strong ground shaking. There are several general requirements for liquefaction to occur. They are as follows.

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

The project is not located within an area mapped as susceptible to liquefaction by San Bernardino County (San Bernardino County, 2010b). Due to the absence of shallow groundwater, the risk of liquefaction is considered low.

***Landslides:*** Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. Due to the relatively bedrock type in the sloping areas of the project and the relatively flat nature of the soil units in the other portions the proposed configuration of the project, the risk of landsliding is considered low.

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

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

***Seiches:*** Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Seiching is possible at the east end of the project if a seismic event coincides with high water levels within the Cactus Basins.

***Earthquake-Induced Flooding***: Dams or other water-retaining structures may fail as a result of large earthquakes, resulting in flooding. The project is not located in an area designated for risk of dam inundation by San Bernardino County (San Bernardino County, 2010a). The risk for earthquake-induced flooding at the project is considered low.

# 7.0 LABORATORY TEST RESULTS

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 are presented below.

## *7.1 Physical Testing*

- Expansion Index – Three representative bulk soil sample from the upper 10 feet of the site materials was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result indicated expansion index of 0 to 4, corresponding to very low expansion potential.
- Sand Equivalent – Two representative bulk soil samples were tested to evaluate sand equivalent (SE) in accordance with the ASTM Standard D2419 test method. The measured sand equivalents ranged from 15 to 24.
- Grain Size Analysis – Three representative samples were tested to determine their relative grain size distributions in accordance with the ASTM Standard D6913. Test results are graphically presented in Drawings No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship test of two representative soil samples were conducted in accordance to ASTM Standard D1557. The results are presented in Drawings No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry densities ranged from 126.9 (with rock correction 129.7) to 134.5 pounds per cubic feet (pcf), with an optimum moisture contents ranging from 9.0 to 11.0 (with rock correction 9.9) percent.
- Direct Shear – Two direct shear tests (TP-02@1'-3' and TP-04@6'-10') were performed on soil samples remolded to 90 percent of the maximum dry density and optimum moisture content under soaked moisture conditions in accordance with the ASTM D3080 method. Results of the direct shear tests are presented in Drawings No. B-3 and B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

## *7.2 Chemical Testing - Corrosivity Evaluation*

Two representative soil samples were 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 the project soils when placed in contact with common pipe materials. These tests were performed by AP

Engineering and Testing, Inc. (Pomona, CA) in accordance with California Tests 643, 422, and 417. The test results are summarized below and are presented in Appendix B, *Laboratory Testing Program.*

- The pH measurements of the samples tested were 7.0 and 7.4.
- The sulfate contents of the samples tested were 0.0033 and 0.0055 percent by weight (33 and 55 ppm).
- The chloride concentrations of the samples tested were 32 and 34 ppm.
- The minimum electrical resistivities when saturated were 5,593 and 6,678 ohm-cm.

## 8.0 EARTHWORK RECOMMENDATIONS

Earthwork for the project will include grading, retaining wall inter-block construction, trench excavation, pipe subgrade preparation, pipeline bedding placement, and trench backfill following the placement of the storm drain, as well as roadway pavement construction.

### *8.1 General*

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

All debris, deleterious material and surficial soils containing roots and perishable materials (if any) should be stripped and removed from the project. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

### *8.2 Over-excavation/Removal*

In fill or shallow cut areas, which are underlain by artificial fill or topsoil, may be prone to future settlement under the surcharge of pipe, foundation, and/or fill loads. These materials should be overexcavated to competent alluvial soils or bedrock and replaced with compacted fill soils. In cut areas, deeper excavation may be required below finish grade, if artificial fill or topsoil are exposed at grade. Overexcavations should also extend at least 2.0 feet below the lowest proposed footings, within the proposed retaining wall areas. Within pavement areas overecavations can be limited to 5 feet below existing grade. However, localized, deeper overexcavation could be encountered where recommended by the geotechnical consultant based on observations during grading. The estimated depths to unsuitable materials are indicated on the Approximate Boring, Test Pit and Seismic Refraction Location Maps (Figures 2a through 2k).

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

The overexcavation below the footings should be uniform. The overexcavation should extend to at least 2 feet beyond the footprint of the footings and at least 1 foot beyond the edge of the pavement. The overexcavation bottom should be scarified and compacted as described in Section 8.8, *Compacted Fill Placement*.

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

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

### *8.3 Slope Stability and Fill Slope Construction*

Cut constructed at a slope ratio of 1.5:1 H:V in bedrock and cut slopes in alluvial soils or fill slopes constructed at a slope ratio of 2:1 H:V should be grossly stable.

Overexcavation of unsuitable soils and a 15-foot wide fill key should be excavated into competent alluvial soils or bedrock at the toes of fill and fill-over-cut slopes. The bottom of the fill keys should be tilted at 2 percent back into the slope.

### *8.4 Pipeline Subgrade Preparation*

The final subgrade surface should be level, firm, uniform, free of loose materials, and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles, larger than 3 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 sub-grade 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.

### *8.5 Pipe Bedding*

Bedding is defined as the material supporting and surrounding the pipe to one foot above the pipe. Pipe bedding should follow the guideline presented in General Permit

Conditions and Trench Specifications, San Bernardino County, 2017. Additional information for pipe bedding is provided as below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate, or crushed rock may be used as pipe bedding material. The sand equivalents of the tested soils vary from 15 to 24. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. Based on laboratory test results, the soils at the project may be suitable for use as bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

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

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

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

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

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

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

Where,

F = Bedding Material
B = Surrounding Native and/or Fill Soils
*D*15(F) = Particle size through which 15% of bedding material will pass
*D*85(B) = Particle size through which 85% of surrounding soil will pass
*D*50(F) = Particle size through which 50% of bedding material will pass
*D*50(B) = Particle size through which 50% of surrounding soil will pass

### *8.6 Backfill Materials*

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. Excavated soils should be processed, including cleaning roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. Screening may be required to remove oversized particles from some on-site soils. 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 should be 20 or less.
- Plasticity index of 10 or less.
- Contain less than 30 percent by weight retained on ¾-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Imported soils, if used as fill, should be predominantly granular and meet the above criteria. Any imported fill should be tested and approved by geotechnical representative prior to delivery to the project.

### *8.7 Backfill Recommendations Behind Walls*

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 compaction should be conducted in such a way within a horizontal distance of 5 feet from the wall so that any overstress will not transfer to 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.

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

All fill placed 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 finish grade underneath pavement should be compacted to at least 95 percent of the laboratory maximum dry density.

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

### *8.9 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 should follow the guideline presented in General Permit Conditions and Trench Specifications, San Bernardino County, 2017. Besides, additional trench backfill recommendations are presented below.

- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse-grained soil, and between optimum and 2 percent above optimum for fine-grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The 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.
- Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.
- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.

### *8.10 Retaining Walls Drainage*

The recommended lateral earth pressure values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures. Wall drainage may be provided by free-draining gravel surrounded by synthetic filter fabric or by prefabricated, synthetic drain panels or weep holes. In either case, drainage should be collected by perforated pipes and directed to a sump, storm drain, or other suitable location for disposal. We recommend drain rock should consist of durable stone having 100 percent passing the 1-inch sieve and less than 5 percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a minimum flow rate of 110 gallons per minute per square foot of fabric, and a minimum puncture strength of 110 pounds.

### *8.11 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. Based on our previous experience in the other projects in close vicinity of this site, for the 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 20 feet of soils and bedrock is estimated to range from approximately 5 to 10 percent within soil areas and 0 to 5 percent in bedrock areas. An average value of 3 to 8 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 is estimated to be approximately 0.1 foot to 0.15 foot, within soil areas and approximately 0.1 foot to 0.15 foot, within bedrock areas

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.

## 9.0 DESIGN RECOMMENDATIONS

General design recommendations, resistance to lateral loads, pipe design parameters, bearing pressures, and soil corrosivity are discussed in the following subsections.

### *9.1 General*

The various design recommendations provided in this section are based on the assumptions that the above earthwork recommendations will be implemented. Design parameters are presented in the sections below.

### *9.2 Retaining Walls Foundation Design Parameters*

Design parameters related to retaining walls are presented in the following sections.

#### 9.2.1 Soil parameters

Soil parameters for each retaining wall are presented below.

**Table No. 5, Soil Parameters for Retaining Walls**

| Soil Parameter | Retaining Wall 1 | Retaining Wall 2 | Retaining Wall 3 | Retaining Wall 4 | Retaining Wall 6 |
|---|---|---|---|---|---|
| Unit weight of Soil, $\gamma$ (pcf) | 125 | 120 | 120 | 130 | 120 |
| Angle of internal friction of soils, $\phi$ | 35 | 28 | 28 | 35 | 35 |
| Soil cohesion, c (psf) | 0 | 50 | 50 | 0 | 0 |

#### 9.2.2 Bearing Capacity

The proposed retaining walls will be supported on continuous (strip) footings. The design of the shallow continuous footings should be based on the recommended parameters presented in the Table No. 6, *Recommend Foundation Parameters*.

**Table No. 6, Recommended Foundation Parameters**

| Parameter | Retaining Wall Foundation (2 & 3) | Retaining Wall Foundation (1, 4 and 6) |
|---|---|---|
| Minimum continuous footing width | 18 inches | 18 inches |
| Minimum continuous footing depth of embedment below lowest adjacent grade to the top of footing | 18 inches | 18 inches |
| Allowable net bearing capacity | 2,000 psf | 2,500 psf |

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

The footing dimensions and reinforcement should be based on structural design. The allowable bearing capacity can be increased by 500 psf with each foot of additional embedment and 100 psf with each foot of additional width up to a maximum of 3,000 psf.

The net allowable bearing values indicated above are for the dead load 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 loading, which will include loading induced by wind or seismic forces.

### 9.2.3 Lateral 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.

**Table No. 7, Recommended Foundation Parameters**

| Retaining Wall | Active Earth Pressure (psf/ft depth) | | At-Rest Earth Pressure (psf/ft depth) | Passive Earth Pressure (psf/ft depth) | Coefficient of Friction | *Seismic Earth Pressure (pcf) |
|---|---|---|---|---|---|---|
| | Level | 1.5:1 Slope | Level | | | |
| 1 | 38 | 84 | 56 | 300 | 0.40 | |
| 2 | 44 | 99 | 64 | 280 | 0.35 | |
| 3 | 44 | 99 | 64 | 280 | 0.35 | 36H |
| 4 | 38 | 88 | 56 | 315 | 0.40 | |
| 6 | 38 | 88 | 56 | 315 | 0.40 | |

(* Wall greater than 6 feet in height)

Active earth pressures assume no surcharge and no hydrostatic pressure. If water pressure is allowed to build up behind the structure, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the structure.

Resistance to lateral loads can be assumed to be provided by friction acting at the base of foundations and by passive earth pressure. A factor of safety of 1.5 was applied in calculating passive earth pressure. These lateral resistances may be increased by 33 percent for seismic forces. Due to the low overburden stress of the soil at shallow depth, the upper one foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

Vertical and lateral bearing values 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.

The equivalent fluid seismic pressure was calculated using Seed and Whitman (1970) procedure. The seismic force applied to the wall is based on a horizontal seismic acceleration coefficient equal to one-third of the peak ground acceleration in accordance with Caltrans Bridge Design Specifications (Caltrans, 2004). An equivalent fluid seismic pressure presented in the above table may be assumed under active loading conditions at the top of an inverted triangle pressure distribution where H is the height of the backfill behind the wall. Under at-rest conditions, the active equivalent fluid seismic pressure should be increased by 30 percent.

### *9.3 Pipe Design Parameters*

Structural design of pipeline 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. 8a, *Soil Parameters for Pipe Design (Storm Drain B to G)* and 8b, *Soil Parameters for Pipe Design (Storm Drain H to N)*.

Where pipelines are connecting to rigid structures near, or at their 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.

**Table No. 8a, Soil Parameters for Pipe Design (Storm Drain B to G)**

| Soil Parameters | Value | | | | | |
|---|---|---|---|---|---|---|
| | Strom Drain B | Strom Drain C | Strom Drain D | Strom Drain E | Strom Drain F | Strom Drain G |
| Unit weight of compacted backfill (assuming 92% average relative compaction), $\gamma$ | 130 pcf | 130 pcf | 135 pcf | 135 pcf | 130 pcf | 135 pcf |
| Angle of internal friction of soils, $\phi$ | 31 | 31 | 35 | 35 | 28 | 35 |
| Soil cohesion, c | 50 pcf | 50 pcf | 0 pcf | 0 pcf | 50 pcf | 0 pcf |
| Coefficient of friction between HDPE pipe and native soils, fs | 0.25 | 0.25 | 0.25 | 0.25 | 0.25 | 0.25 |
| Bearing pressure against Alluvial Soils | 2,000 psf | 2,000 psf | 2,500 psf | 2,500 psf | 2,000 psf | 2,500 psf |
| Coefficient of passive earth pressure, Kp | 3.12 | 3.12 | 3.69 | 3.69 | 2.77 | 3.69 |
| Coefficient of active earth pressure, Ka | 0.32 | 0.32 | 0.27 | 0.27 | 0.36 | 0.27 |
| Modulus of Soil Reaction, E' | 1,500 psi | 1,500 psi | 1,500 psi | 1,500 psi | 1,500 psi | 1,500 psi |

**Table No. 8b, Soil Parameters for Pipe Design (Storm Drain H to N)**

| Soil Parameters | Value | | | | | |
|---|---|---|---|---|---|---|
| | Strom Drain H | Strom Drain I | Strom Drain J | Strom Drain K | Strom Drain M | Strom Drain N |
| Unit weight of compacted backfill (assuming 92% average relative compaction), $\gamma$ | 135 pcf | 130 pcf | 130 pcf | 130 pcf | 132 pcf | 132 pcf |
| Angle of internal friction of soils, $\phi$ | 35 | 28 | 31 | 31 | 33 | 33 |

| Soil Parameters | Value | | | | | |
|---|---|---|---|---|---|---|
| | Strom Drain H | Strom Drain I | Strom Drain J | Strom Drain K | Strom Drain M | Strom Drain N |
| Soil cohesion, c | 0 pcf | 50 pcf | 50 pcf | 50 pcf | 0 pcf | 0 pcf |
| Coefficient of friction between pipe and native soils, fs | 0.25 | 0.25 | 0.25 | 0.25 | 0.25 | 0.25 |
| Bearing pressure against Alluvial Soils | 2,500 psf | 2,000 psf | 2,000 psf | 2,000 psf | 2,000 psf | 2,000 psf |
| Coefficient of passive earth pressure, Kp | 3.69 | 2.77 | 3.12 | 3.12 | 3.39 | 3.39 |
| Coefficient of active earth pressure, Ka | 0.27 | 0.36 | 0.32 | 0.32 | 0.29 | 0.29 |
| Modulus of Soil Reaction, E' | 1,500 psi | 1,500 psi | 1,500 psi | 1,500 psi | 1,500 psi | 1,500 psi |

### *9.4 Soil Corrosivity*

The results of chemical testing of four representative soil samples were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel (if present). The test results are presented in Appendix B, *Laboratory Testing Program* and are discussed below.

The sulfate content of the sampled soil corresponds to American Concrete Institute (ACI) exposure category S0 for this sulfate concentration (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 will be exposed to moisture from precipitation and irrigation. Based on the project location and the results of chloride testing of the site soils, we do not anticipate concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

The minimum electrical resistivities when saturated were 5,593 and 6,678 ohm-cm. These values indicate that the tested soils are moderately corrosive to ferrous metals in contact with the soil (Romanoff, 1957).

According to the Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive if the pH is 5.5 or less, or chloride content is 500 parts per million (ppm) or greater, or sulfate content is 1,500 ppm or greater, or resistivity less than 2,000 ohm- cm. Based on the tested results, the project soils are not considered corrosive.

<u>Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the soils.</u>

### *9.5 Asphalt Concrete Pavement*

One representative soil sample (Converse, 2019) and three representative soil samples (CHJ, 2014) were tested to determine the R-value of the subgrade soils. The tested R-values were 11, 35, 41, 51 and 65. For pavement design, a medina R-value of 35 and design Traffic Index of 6 was utilized.

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

**Table No. 9, Recommended Preliminary Pavement Sections**

| R-value 35 | Traffic Index (TI) | Pavement Section | | |
|---|---|---|---|---|
| | | Option 1 | | Option 2 |
| | | Asphalt Concrete (inches) | Aggregate Base (inches) | Full AC Section (inches) |
| | 6 | 4.0 | 6.0 | 7.0 |

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

Aggregate base materials should be moisture conditioned as needed to near optimum moisture content and compacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557) for support of new pavement sections.

Base materials should conform to Section 200-2 of the Greenbook, 2018, or as required by the County of San Bernardino, and should be placed in accordance with Section 301-2 of the Greenbook.

In order to lengthen the life span of the pavement, the top portion of HMA surface layer may be replaced with equivalent gap-graded Rubberized Hot Mix Asphalt (RHMA-G) and/or a rubberized stress absorbing membrane interlayer (SAMI-R) may be placed below the RHMA-G surface layer. The RHMA-G thickness should have a minimum of 0.1 feet and a maximum of 0.2 feet. (Caltrans HDM, Topic 631). The RHMA-G and SAMI-R will reduce the occurrence of reflective cracking of the pavement surface.

Asphalt concrete materials should conform to Section 203 of the Greenbook, 2018 and should be placed in accordance with Section 302-5 of the Greenbook, or as required by the County of San Bernardino.

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

# 10.0 CONSTRUCTION RECOMMENDATIONS

Construction recommendations for the project are presented below.

## *10.1 General*

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

Sloped excavations may not be feasible in locations adjacent to existing utilities or structures, including utilities, channels, or other improvements. Recommendations pertaining to temporary excavations are presented in this section.

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, current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the owner's representative and the competent person employed by the contractor in accordance with regulations. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

## *10.2 Temporary Sloped Excavations*

Temporary open-cut trenches may be constructed with side slopes as recommended in the table below. 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. 10, Slope Ratios for Temporary Excavations**

| Soil Type | OSHA Soil Type | Depth of Cut (feet) | Recommended Maximum Slope (Horizontal:Vertical)[1] |
|---|---|---|---|
| Silty Sand with (SM), Silty Sand with Gravel (SM), Gravel with Sand (GP-GM), Clayey Sand (SC), Sandy Silt (ML) and Silty Clay with Sand (CL) | C | 0-10 | 1.5:1 |
| | | 10-20 | 2:1 |
| Bedrock (excavated as Silty Sand and Clayey Sand) | A | 0-20 | 3/4:1 |

[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 as necessary to protect the workers in the excavation.

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

### *10.3 Shoring Design*

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

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

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

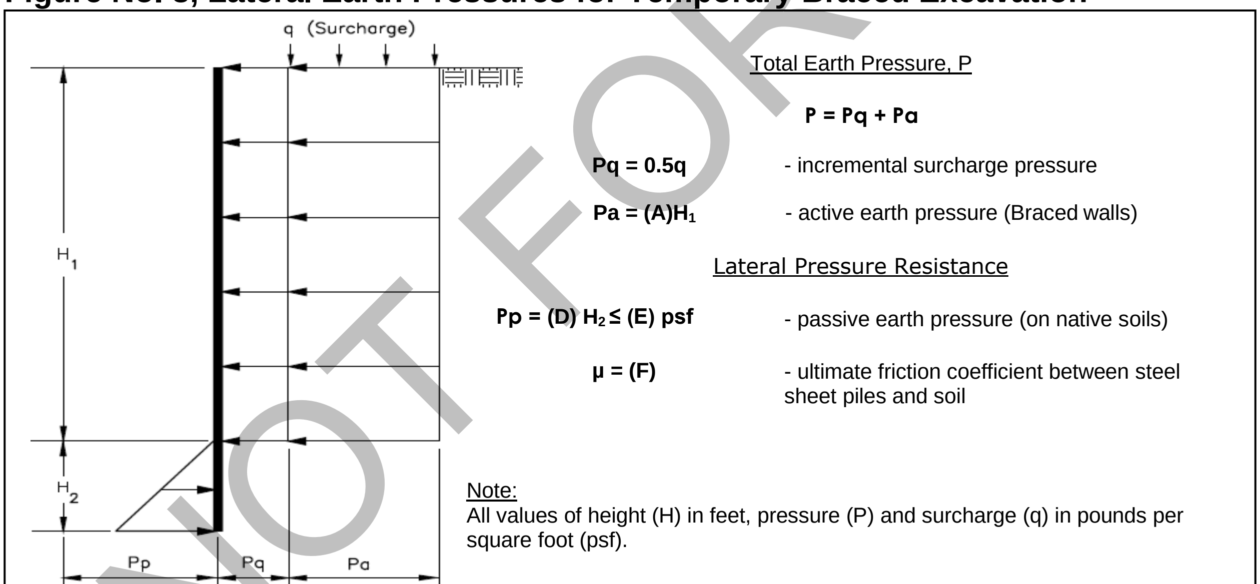
### Table No. 11, Lateral Earth Pressures for Temporary Shoring

| Lateral Resistance Soil Parameters* | Snowdrop Road |
|---|---|
| Active Earth Pressure (Braced Shoring) (psf) (A) | 24 |
| Active Earth Pressure (Cantilever Shoring) (psf) (B) | 40 |
| At-Rest Earth Pressure (Cantilever Shoring) (psf) (C) | 60 |
| Passive earth pressure (psf per foot of depth) (D) | 250 |
| Maximum allowable bearing pressure against native soils (psf) (E) | 2,000 |
| Coefficient of friction between sheet pile and native soils, fs (F) | 0.25 |

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

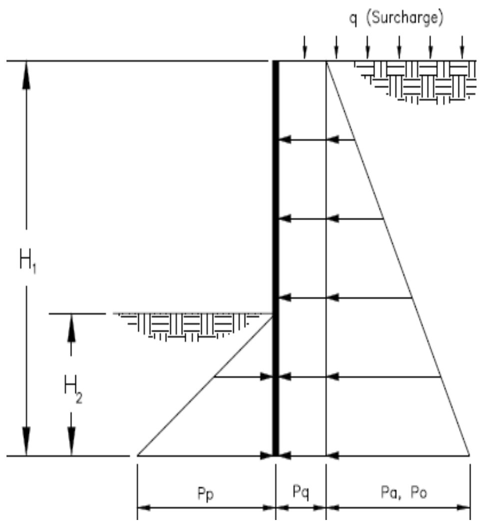
Restrained (braced) shoring systems should be designed based on Figure No. 3, *Lateral Earth Pressures for Temporary Braced Excavation* to support a uniform rectangular lateral earth pressure.

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




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

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




Total Earth Pressure, P

**P = Pq + Pa, Po**

**Pq = 0.5q** - incremental surcharge pressure

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

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

Lateral Pressure Resistance

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

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

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

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

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

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

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

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

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

## 11.0 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. Testing should be performed to determine density and moisture during the project 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.

## 12.0 CLOSURE

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

Design recommendations given in this report are based on the assumption that the recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the actual soil 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.

## 13.0 REFERENCES

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# Appendix A

## Field Exploration

# APPENDIX A

## FIELD EXPLORATION

Our investigation included field reconnaissance and a subsurface exploration program consisting of performing soil test pits. During the field reconnaissance, the surface conditions were noted, and the pits were marked at locations selected by Converse. The approximate pit locations were established in the field by reference to street centerlines, and other visible features and should be considered accurate only to the degree implied by the method used to mark them in the field.

Six exploratory test pits (TP-01 through TP-06) were excavated using a backhoe equipped with 24-inch wide bucket to investigate the subsurface conditions along the proposed road alignment on October 7, 2019. The test pits were excavated to depths of approximately 6.0 feet to 10.0 feet below the existing ground surface (bgs). Converse also previously drilled four exploratory borings, from about Station 63+00 to 67+00 to depths of approximately 11.0 feet to 16.5 feet below the existing ground surface (bgs) on the site in December 2018, as reported in the referenced geotechnical report.

Encountered earth materials were continuously logged by a Converse geologist and visually classified in the field in accordance with the Unified Soil Classification System. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

Soils and rocks encountered in the pits were logged by a Converse geologist and were classified in the field by visual examination in accordance with the Unified Soil Classification System (ASTM Standard 2488). The field descriptions presented on the test pit logs have been modified where appropriate to reflect laboratory test results.

Representative bulk samples were collected from selected depths within the test pits. The samples were obtained from the excavated soil and placed in large plastic bags for delivery to our laboratory. Test pits were backfilled with excavated soil and tamped. The ground surface at the test pit locations may settle over time. If construction is delayed, we recommend the owner monitor the test pit locations and backfill any depressions that occur or provide protection around the test pit locations to prevent trip and fall injuries from occurring.

For a key to soil symbols and terminology used in the test pits, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. Logs of the test pits are presented in Drawings No. A-2 through A-7, *Log of Test Pits.*

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS: GRAPH | SYMBOLS: LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS | GRAVEL AND GRAVELLY SOILS | 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 |
| | MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | SAND AND SANDY SOILS | 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 |
| | MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS | 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 |
| MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | 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

## TEST PIT SYMBOLS

SAMPLE TYPE

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

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

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

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

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

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

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

## UNIFIED SOIL CLASSIFICATION AND KEY TO TEST PIT SYMBOLS

# Log of Test Pit No. TP-1

Dates Drilled: 10/7/2019  Logged by: William Buckley  Checked By: Bob Gregorek

Equipment: Backhoe  Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2578  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 | Description | SAMPLES: DRIVE | SAMPLES: BULK | BLOWS | MOISTURE | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):** | | | | | | |
| | | **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" in largest dimension, light brown. | | | | | | |
| | | - few gravel and cobbles up to 5" in largest dimension, orangish-brown | | | | | | ma |
| 5 | | | | | | | | |
| | | - few gravel and cobbles up to 12" in largest dimension, brown | | | | | | |
| 10 | | | | | | | | |
| | | End of test pit at 10.0 feet bgs.<br>No groundwater encountered.<br>Test pit backfilled with soil cuttings, tamped and wheel rolled for compaction on 10/7/19. | | | | | | |

# Log of Test Pit No. TP-2

# Log of Test Pit No. TP-3

Dates Drilled: 10/7/2019 Logged by: William Buckley Checked By: Bob Gregorek

Equipment: Backhoe Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2625 Depth to Water (ft): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):**<br>**SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" in largest dimension, light brown. | | | | | | ei,max,ds |
| | | **BEDROCK:**<br>**GRANULITIC GNEISS (Pm):** severely weathered, no apparent bedding, low hardness, grayish-brown. | | | | | | |
| 5 | | - moderately weathered, moderate hardness, light grayish-brown, sampled as Silty Sand (SM). | | | | | | |
| | | End of test pit at 6.0 feet bgs due to refusal on bedrock.<br>No groundwater encountered.<br>Test pit backfilled with soil cuttings, tamped and wheel rolled for compaction on 10/7/19. | | | | | | |

# Log of Test Pit No. TP-4

**Converse Consultants** City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

**18-81-316-02 A-3**

# Log of Test Pit No. TP-5

Dates Drilled: 10/7/2019 Logged by: William Buckley Checked By: Bob Gregorek

Equipment: Backhoe Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2783 Depth to Water (ft): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):**<br>**SILTY SAND (SM):** fine to medium-grained, some clay, reddish-brown. | | | | | | |
| | | - light brown | | | | | | |
| 5 | | | | | | | | ei,se<br>ca,er,ma |
| 10 | | | | | | | | |
| | | End of test pit at 10.0 feet bgs.<br>No groundwater encountered.<br>Test pit backfilled with soil cuttings, tamped and wheel rolled for compaction on 10/7/19. | | | | | | |

# Log of Test Pit No. TP-6

**Converse Consultants** City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

**18-81-316-02 A-4**

# Log of Test Pit No. TP-7

Dates Drilled: 10/7/2019 | Logged by: William Buckley | Checked By: Bob Gregorek

Equipment: Backhoe | Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2741 | Depth to Water (ft): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):**<br>**SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" in largest dimension, light brown. | | | | | | |
| 5 | | | | | | | | |
| | | **BEDROCK:**<br>**GRANULITIC GNEISS (Pm):** moderate to severely weathered, no apparent bedding, moderate hardness, light grayish-brown, sampled as Silty Sand (SM). | | | | | | ei,se,ca,er ma,max |
| 10 | | | | | | | | |
| | | End of test pit at 10.0 feet bgs.<br>No groundwater encountered.<br>Test pit backfilled with soil cuttings, tamped and wheel rolled for compaction on 10/7/19. | | | | | | |

# Log of Test Pit No. TP-9

Dates Drilled: 10/7/2019 | Logged by: William Buckley | Checked By: Bob Gregorek

Equipment: Backhoe | Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2641 | Depth to Water (ft): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **<u>VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):</u>**<br>**SILTY SAND (SM):** fine to coarse-grained, some clay, few gravel and cobbles up to 12" in largest dimension, brown. | | | | | | |
| 5 | | **<u>BEDROCK:</u>**<br>**GRANULITIC GNEISS (Pm):** moderate to severely weathered, no apparent bedding, moderate hardness, light grayish-brown. | | | | | | |
| | | End of test pit at 5.0 feet bgs due to refusal on bedrock.<br>No groundwater encountered.<br>Test pit backfilled with soil cuttings, tamped and wheel rolled for compaction on 10/7/19. | | | | | | |

# Log of Test Pit No. TP-10

**Converse Consultants** City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

**18-81-316-02** **A-6**

# Log of Test Pit No. TP-11

Dates Drilled: 10/7/2019 Logged by: William Buckley Checked By: Bob Gregorek

Equipment: Backhoe Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 2383 Depth to Water (ft): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **YOUNG ALLUVIAL FAN DEPOSITS (Qyf):**<br>**SILTY SAND (SM):** fine to coarse-grained, few gravel, cobbles and boulders up to 36" in largest dimension, brown. | | | | | | |
| 5 | | | | | | | | |
| | | End of test pit at 6.0 feet bgs due to refusal on cobbles and boulders.<br>No groundwater encountered.<br>Test pit backfilled with soil cuttings, tamped and wheel rolled for compaction on 10/7/19. | | | | | | |

# Log of Test Pit No. TP-12

# Appendix A-1

Field Exploration, Converse Consultants (1/22/19)

# Log of Boring No. BH-1

Dates Drilled: 12/21/2018 | Logged by: William Buckley | Checked By: James Burnham

Equipment: 6" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2698 | 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) | Description | SAMPLES: DRIVE | SAMPLES: BULK | BLOWS | MOISTURE | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|
| 0–3 | **ARTIFICIAL FILL** **SILTY SAND with GRAVEL (SM):** fine to coarse-grained, gravel and cobbles up to 5" in largest dimension, light reddish-brown. | | | | | | |
| 3–10 | **ALLUVIUM** **SILTY SAND with GRAVEL (SM):** fine to coarse-grained, gravel up to 2" in largest dimension, reddish-brown. | | | | | | |
| 5 | | ■ | ▨ | 25/12/12 | 9 | 94 | ei,ma,max col |
| | | ■ | ▨ | 43/50-4" | 6 | 112 | |
| 10 | **BEDROCK: DECOMPOSED GRANITE (Dg):** severely to completely weathered, no apparent bedding, orangish-brown. | ■ | | 4/31/50 | 5 | 123 | |
| 15 | | ■ | | 50-5" | 9 | 115 | |

End of boring at 15.4 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 12/21/2018.

# Log of Boring No. BH-2

ty of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

| Project No. | Drawing No. |
| --- | --- |
| **18-81-316-01** | **A-2** |

# Log of Boring No. BH-3

Dates Drilled: 12/21/2018 Logged by: William Buckley Checked By: James Burnham

Equipment: 6" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2673 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 | Description | SAMPLES: DRIVE | SAMPLES: BULK | BLOWS | MOISTURE | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **ARTIFICIAL FILL** **SILTY SAND (SM):** fine to coarse-grained, gravel and cobbles up to 5" in largest dimension, light reddish-brown. | | | | | | |
| 5 | | | | | 15-50-5.5" | 5 | 112 | r |
| | | | | | 50-4" | | | NR |
| | | **ALLUVIUM** **SILTY SAND (SM):** fine to coarse-grained, gravel up to 2" in largest dimension, light brown. | | | | | | |
| 10 | | | | | 50/28/22 | 5 | 115 | |
| 15 | | **BEDROCK: DECOMPOSED GRANITE (Dg):** severely to completely weathered, no apparent bedding, orangish-brown. | | | 35/50-5.5" | 3 | 128 | |

End of boring at 15.9 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 12/21/2018.

# Log of Boring No. BH-4

Converse Consultants

Snowdrop Road Improvement Project
Snowdrop Road, 0.4 Miles West of Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

| Project No. | Drawing No. |
|---|---|
| 18-81-316-01 | A-3 |

# Log of Boring No. BH-5

Dates Drilled: 12/21/2018 Logged by: William Buckley Checked By: James Burnham

Equipment: 6" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2666 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) | Description | SAMPLES: DRIVE | SAMPLES: BULK | BLOWS | MOISTURE | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|
| 0–4 | **ARTIFICIAL FILL** **SILTY SAND with GRAVEL (SM):** fine to coarse-grained, gravel and cobbles up to 5" in largest dimension, brown. | | | | | | ei,r,ca,er, ma,max |
| | | | | 32/50-4.5" | 5 | 108 | dist. |
| 5 | | | | 37/8/10 | 7 | 110 | col |
| 4–8 | **ALLUVIUM** **SILTY SAND with GRAVEL (SM):** fine to coarse-grained, gravel up to 2" in largest dimension, brown. | | | | | | |
| 10 | **BEDROCK: DECOMPOSED GRANITE (Dg):** severely to completely weathered, no apparent bedding, orangish-brown. | | | 50-4" | 5 | 112 | |

End of boring at 11.0 feet bgs due to refusal on bedrock.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped on 12/21/2018.

# Log of Boring No. BH-6

y of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

| Project No. | Drawing No. |
|---|---|
| 18-81-316-01 | A-4 |

# Log of Boring No. BH-7

Dates Drilled: 12/21/2018 Logged by: William Buckley Checked By: James Burnham

Equipment: 6" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2667 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 | Description | SAMPLES DRIVE | SAMPLES BULK | BLOWS | MOISTURE | DRY UNIT WT. (pcf) | OTHER |
|---|---|---|---|---|---|---|---|---|
| | | **3" ASPHALT CONCRETE / NO AGGREGATE BASE** | | | | | | |
| | | **ARTIFICIAL FILL** **SILTY SAND with GRAVEL (SM):** fine to coarse-grained, gravel up to 2" in largest dimension, light reddish-brown. | ■ | ▨ | 36/28/40 | 4 | 119 | |
| 5 | | **ALLUVIUM** **SILTY SAND with GRAVEL (SM):** fine to coarse-grained, gravel up to 3" in largest dimension, brown. | ■ | ▨ | 28/50-5.5" | 8 | 116 | |
| | | | ■ | ▨ | 20/21/28 | 6 | 124 | |
| 10 | | - orangish-brown | ■ | | 23/23/13 | 4 | 123 | |
| 15 | | | ■ | | 6/6/7 | 8 | 95 | |

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings, tamped and surface patched with cold asphalt concrete on 12/21/2018.

# Log of Boring No. BH-8

y of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

| Project No. | Drawing No. |
|---|---|
| 18-81-316-01 | A-5 |

# Appendix A-2

Field Exploration, CHJ Consultants (3/17/14)

# KEY TO LOGS

**LEGENO OF LAB/FIRLD** TESTS:

AL Atterberg Limit (ASTM D4318)

Blows A measure of the penetration resistance of soil expressed as the number of hammer blows required to advance the indicated sampler 6 inches (or less if noted). Samplers are driven with an automatic hammer that drops a **140-pound** weight 30 inches for each blow. After the required seating, samplers are advanced up to 18 inches ahead **of the boring, providing** up to three sets of blows per drive.

Bulk Indicates Disturbed or Bulk Sample

Cor. Chemical/Corrosivity Tests **(Caltrans 417, 422 and 643)**

Dist. Indicates **Disturbed** Sample

DS Direct Shear Test (ASTM **D3080)**

MDC Maximum Density - Optimum Moisture Determination (ASTM D1557)

N.R. Indicates No Recovery of Sample

Ring Indicates Relatively Undisturbed Ring Sample. Relatively undisturbed ring samples are obtained with a modified **California** sampler (3.0" O.D. and 2.42" I.D.) lined with rings driven with a 140-pound weight falling 30 inches.

RV **R-value (CT 301)**

SA Sieve Analysis (ASTM D422)

SE Sand Equivalent Test (ASTM D2419)

Enclosure "B" (2 of 3)
**Job No. 14095-3**

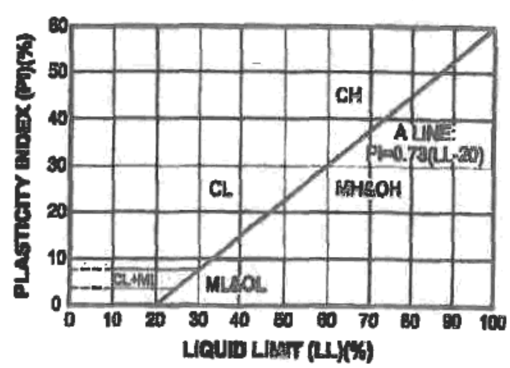
# U9(FíED 9O1L CŒã$JFJCùEŒ öYãTEK

| UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART | | | |
|---|---|---|---|
| COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size) | | | |
| GRAVELS More than 50% of coarse fraction larger than No.4 sieve size | Clean Gravels (Less than 5% 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 (More than 12% fines) | | |
| | | GM | Silty gravels, gravel-sand-silt mixtures |
| | | GC | Clayey gravels, gravel-sand-clay mixtures |
| SANDS 50% or more of coarse fraction smaller than No.4 sieve size | Clean Sands (Less than 5% fines) | | |
| | | SW | Well-graded sands, gravelly sands, little or no fines |
| | | SP | Poorly graded sands, gravelly sands, little or no fines |
| | Sands with fines (More than 12% fines) | | |
| | | SM | Silty sands, sand-silt mixtures |
| | | SC | Clayey sands, sand-clay mixtures |
| FINE-GRAINED SOILS (50% or more 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 of 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 50% or greater | | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts |
| | | CH | Inorganic clays of high plasticity, fat clays |
| | | OH | plasticity. |
| HIGHLY ORGANIC SOILS | | PT | Peat and other highly organic soils |

| LABORATORY CLASSIFICATION CRITERIA | |
|---|---|
| GW $C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$ between 1 and 3 | |
| GP Not meeting all gradation requirements for GW | |
| GM Atterberg limits below "A" line or P.I. less than 4 | Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols. |
| GC "A" | |
| SW $C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$ between 1 and 3 | |
| SP Not meeting all gradation requirements for SW | |
| SM Atterberg limits below "A" line or P.I. less than 4 | Limits plotting in shaded zone |
| SC Atterberg limits above "A" line with P.I. greater than 7 | of dual symbols. |

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size). Coarse-grained soils are classified as follows:
Less than 5 percent........................................GW, GP, SW, SP
More than 12 percent......................................GM, GC, SM, SC
5 to 12 percent..........................Borderline cases requiring dual symbols

**PLASTICITY CHART**



# SOILCONEbTENCY

## Coninactness of Granular Soils

| **Description** | **Approximate Relative Density (%)** |
|---|---|
| Very Loose | **0-15** |
| Loose | **i5-40** |
| Medium Dense | 40-70 |
| Dense | 70-85 |
| Very Dense | 85-100 |

## ConslmncvofPlasdcSolh

| Description | Approximate Shear Strength **(psf)** |
|---|---|
| Very Soft | sdaD250 |
| Soft | 250-500 |
| Medium Stiff | |
| Stiff | 1,000-2,000 |
| Very Stiff | 2,000M,€KB |
| Hard | Momthan4,AD |

# EXPLORATORY TRENCH NO. 1

Date Excavated: 2/2t)/I4

Client: County of San Bemardino Special Services Department

Equipment: Rubber Tire Backhoe

Bucket Six: 18" Bucket

Surface Eevation(ft):

Logged by: JMcK

**Station** No.: N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DENSITY | SAMPLES BULK | RELATIVE COMP. (%) | FIELD MOISTURE (%) | DRY UNIT WT. (pcf) | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | **Asphalt Concrete, 4-1/** | Fill | | | | | | |
| | | **Aggreg** **M)** ate Base, 4" | | | | | | | |
| 1 | | (S SitySand, finetocoarse, witiigravelandboulders, dark gray brown | | | | | 3.3 | | os,›uz›c |
| 2 | | | | | | | | | |
| 3 | | | | | | | | | |
| 4 | | **d, e, with cobbl ders to** (SP) Sao gae to coars esandboul 24" ia size, Few **slit** | Native-Qy£ | | | | | | |
| 5 | | | | | | | | | |
| | | **END OF TRENCH** | | | | | | | |
| 6 | | **.5' - 4.5'** | | | | | | | |
| 7 | | NO REFUSAL, NO GROUNDWATER<br>SLIGHT CAVING, NO BEDROCK<br>EILL TO 3 | | | | | | | |
| 8 | | | | | | | | | |
| 9 | | | | | | | | | |

TRENCH_LOG_10_FT 14095-3.GPJ CHJ.GDT 3/18/14

# EXPLORAZ'ORY TRENCH NO.2

Date Excavated: 2/20/14

Client: County of **San** Bernardino Special Services Department

Equipment: Rubber Tire Backhoe

Bucket Size: 18" Bucket

see m»ation(ft):

Logged by: JMcK

Station No.: N/A

| DEPTH (ft) | GRAPHIC LOG | WSUALCIASSIMCAMON | REMARKS | SAMPLES DENSITY | SAMPLES BULK | RELATIVE COMP. (%) | FIELD MOISTURE (%) | DRY UNIT WT. (pcf) | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | Asphalt Concrete, 5-1/2" | Fill | | | | | | |
| 1 | | (GP-GM) Gravel with sand, silt, and b size | | | | | 3.4 | | |
| | | (SM) Silty Sand, fine to medi l um, b ack | | | | | | | |
| 2 | | (SP) Sand, fine to coarse, with gravel and boulders to 30 in size", few silt | Native - Qyf | | | | | | |
| 3 | | | | | | | | | |
| 4 | | | | | | | | | |
| 5 | | | | | | | | | |
| | | END OF TRENCH | | | | | | | |
| 6 | | | | | | | | | |
| | | NO REFUSAL, NO GROUNDWATER<br>SLIGHT CAVING, NO BEDROCK<br>FILL TO 2.0' - **2.25'** | | | | | | | |
| 7 | | | | | | | | | |
| 8 | | | | | | | | | |
| 9 | | | | | | | | | |

TRENCH LOG 10_FT 14095-3.GPJ CHJ.GDT 3/18/14

# EXPLORATORY BORING NO. 3

Date Drilled: 2/18/14 cxe«t: CountyorsanB«nardaospecia s«vicesDepaitment

Equipnmit: CME 75 Tnick Rig Driving Weight / Drop: 140 lbs./ 30 N

Surface Elevation(fl): Logged by: JMcK Measured Depth to Water(ft): N/A

| DEPTH | GRAPHIC | VISUAL CLASSIFICATION | ARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS / 6 IN. | FIELD MOIs1in (egg) | DRY UNITWT | TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | **(CL) Silty Clay, with sand, fine, red orange with weathered angular bedrock fragments** | Native | | | | 11.0 | | |
| | | | | | | | 10.3 | 113 | Ring |
| | | | Pm Weathered | | | 21 25 | | | |
| 5 | | | | | | | | | |
| | | | | | | 40 | 10.3 | 110 | |
| | | Gneiss bedrock returned as (ML) Sandy Silt, tine, with clay, light brow Bedrock has angular cla•4s 1" to 4", and is highly weathered, cemented and dense | P'• | | | | 1.3 | | |
| 10 | | | | | | 18 50 | 2.7 | 115 | Ring |
| 15 | | | | | | 50/1" | 7.1 5.5 | | |
| 20 | | | | | | | | | |
| 25 | | | | | | 50 | N.R. | N.R. | Ring |

DNI3 OF BORING

NO REFUSAL, NO CAVING
NO GROUNDWATER, NO FI£L
GNEISS BEDROCK AT 7'

30

# EXPLORATORY BORING NO. 4

Date Drilled: **2/21/14**

Client: Gounty of San Bemardino Special Services Department

**Equipment:** CME 75 Truck Rig

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

Surface Eevation(fl):

**Logged by: JMcK**

Measured Depth to Water(ft): N/A

| DEPTH (ft) | GBAPI-iIC | VISUAL CLASSIFICATION | | SAMPLES | BLOWS/6 IN. | FIELD MOISTURE (%) | DRY IPU | LABPI ELD TESTS |
|---|---|---|---|---|---|---|---|---|
| | | gAspaait concrete, 5-l/8• <br> (8M) Silty Sand, fine, with gravel to 2.5", gray brown | Fill | | 5<br>8 | 4.5 | Dist. | Ring |
| 5 | | brMn klayey »3 nne, with silt and gmvel to I", gray | | | 7<br>14<br>12 | 2.3<br>5.8 | 119 | SA, MDC, Ring |
| 10 | | (S $ S e to medium with coarse, with gravel | | | 12<br>14<br>11 | 5.8<br>6.6 | 114 | Ring |
| | | **END OF BORING** | **Auger Chatter** | | | | | |

—15

NO CAVING, NO BEDROCK
NO GROUNDWATER, EILL TO 13'
REFUSAL ON BOULDER AT 13'

20

25 -

- 30



**SNOWDROP**
**RANCHOCUCAMONG CALHDIUflA**

Job No. Enclosure

14095-3 B-4

EXPLORATORY BORING NO. 5



**SNOWDROP**
**RANCHOCUCAMONG CALHDIUflA**

Job No. Enclosure

# EXPL€tRATORY BORING NO. 5

Date Drilled: 2/21/14

Equipment: CME 75 Zruck Rig

Client: County of San Bernardino Special Services Department

Driving Weight / Drop: 140 lbs. / 30 in

Surface Elevation(ft):

Logged by: fMcK

Measured Depth to Water(ft): N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | ARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | FIELD MOISTURE (%) | | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | SA |
| | | | | | | | 6.8 | 121 | DS, Rjjjg |
| 5 | | | | | | | | | |
| | | | | | | | 5.4 | 119 | Riag |
| | | Gness Bedrock returned as (SM) Stt\y F'••n*, fine to aodium, féw gravel **to 3/4"** | !'* | | | 26 50/5" | 5.2 5.8 | 112 | Ring |
| 10 | | | | | | | | | |
| 15 | | NO RRFUSALi NO CAVING NO GROUNDWATER, NO FILL GNEISS BEDROCK AT 8' | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

# E LORATORYBOMNGNO.6

Date Drilled: 2/21/14

Client: County of San Beinardino Special Services Deparonent

Equipment: **CME 75 Truck Rig**

140 lbsJ 30 in.

**Driving Weight / Drop:**

**Logged by: JMcK**

Surface Elevation(ft):

Measured Depth to Water(ft): N/A

| DEPTH (fl) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | MOIMOE | DRYER. | TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | **Asphalt** Concrete, 2-lf8" | Base on | | | | | | |
| | | Aggreg4t Base, 4" | Native soi | | | | | | |
| | | (SC) Clayey °•as-dp fine to oudiuin, with silk brown | Qa | | | t4 17 2t | 8.2 4.9 | 121 | AL, Cor., SA DS, Ring |
| 5 | | (SU fiilty 8and, fire to i aium with coane, with clay and gravel to 1/2", mottled brown to light brown | | | | 9 12 13 | 7.5 6.9 | 116 | Ring |
| 10 | | | | | | 9 11 13 | 5.5 | 105 | Ring |
| 15 | | | | | | 21 50/4.5" | 10.8 NJ.7 | 114 | Ring |
| | | °Gneiss bedrock returned as (SC) Clayey Sand, fine to medium, stiff, brown | | | | | | | |
| 20 | | | | | | 35 14 | 13.2 | 121 | Ri9g |

AND OF BORING

NO REFUSED, NO CAVING
NO GROUNDWATER, NO FILL
GNEISS BF-DROCKAT 17'

25

# EXPLORATORI BORING NO. 7

*Date Driiled:* **2/18/14**

Equipment: CME 75 Truck Rig

**Client:** County of San Bernardino Special Services Department

Driving Weight / Drop: 140 Ibs./ 30 in.

Surface Elevation(fl):

Logged by: JMcK

Measuied Depth to Water(S): N/A

| DEPTH (g) | GRAPHIC L ŒÌ | VISUAL CLASSIFICATION | MARKS | SAMPLES DRIVE | SAMPLES BULKS | BLOWS/6 IN. | FIELD MOISTURE (%) | DRY UNIT | TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | ’\Asphalt Concreta, 1-1/4" | Fill | | | | | | |
| | | (ML) Sandy Silt, fine, with gravel to 1/4", light brown | | | | 14 13 | 5.7 | 110 | Rit¡g |
| | | | | | | | 8,4 | 120 | Ring |
| | | | Qa | | | | | | |
| | | (sM) 8ilty x•»* n-, with giavel to lf2", brown | | | | | 6.1 | | |
| 10 | | | | | | 21 | 3.5 | ]2j | gittg |
| | | | Pm | | | | | | |
| | | (ifielss beiÍfock, **fOIiBIed** | | | | | | | |
| 15 | | | | | | 25 50f5" | ]7 | Düt | |

Em ur aonwo

NO REFUSAL, NO CAVING
NO GROUNDWATER, FILL TO 8'

20 GNEISS BEDROCKAT 14'

CHUGDT

30 -

# EXPLORATORY BORING NO.8

Date Drilled: 2/18/14

Cheat: Conaty o:£Saa Bemardiao Special Services Depaztzaeot

B;ptipmmt: CMS 75 Tnick Rig

Dciviog Weigbt / Drop: 140lbs./30 ia.

Surface Elevation(ft):

Logged by: JMcK

Measuiwl Depth to Water(ft): N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | FIELD MOISTURE (%) | DRY UNIT WT. (pcf) | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | (SM) Silty Sand, fine with medium, light brown | Fill | | | | 4.1 | | |
| | | | | | | 17<br>30<br>43 | 2.f | 125 | DS, Ring |
| 5 | | (SM) Silty Sand, fine with medium, trace gravel to 1/2", blown | Native - Qvof | | | 34<br>50 | 5.0<br>4.7 | 134 | Ring |
| | | | | | | 34 | 4.7 | 123 | Ring |
| 15 | | (SM) Silty Sand, fine to mwlium, with angular gravel to 1", brown | | | | 13<br>22<br>30 | 3.7<br>6.5 | 124 | Ring |
| | | END OF BORING<br><br>NO REFUSAL, NO CAVING<br>NO GROUNDWATER, NO BEDROCK<br>FILL TO 5' | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

10331-3 14095-3.GPJ CHJ.GDT 3/18/14

# EXPLORATORY TRENCH NO. 9

Date Excavated: 2/20/14

Client: County of San Beniaidino Special Services Department

Equipment: Rubber Tire Bnnkhng

Bucket Size: 18" Bucket

Surface Elevation(fl):

Logged by: JMcK

**Station** No.: N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | MARK | DRNsMY | BULK | M LATT COMC (%) | FIELD MOISTURE (%) | TES7S |
|---|---|---|---|---|---|---|---|---|
| | | (xM) Siity can.fine to i **alum, with** graver to a", light brown | ° • - 9• | | | | ° | |
| 1 | | | | | | | | |
| 2 | | | | | | | | |
| 3 | | Debris Bow— • ~~ts~~ returned as (SM) ailty - , Anne **to medium with** coarse, with cobbles and boulders to 42" | 9•f | | | | | |
| 4 | | | | | | | | |
| 6 | | | | | | | | |
| 7 | | END OF TRENCH | | | | | | |
| 8 | | NO REFUSAL, NO GROUNDWATER<br>MODERAM CAVING, NO BEDROCK<br>NO FILL | | | | | | |

s

NOT FOR BID

TRENCH LOO 10 FT 1409M.GPJ CHJ GDT

# EXPLORATORY BORING NO. 10

Date **Drilled: 2/18/14**

Ilquipment: CME 75 Truck Rig

**Client: County of** San **Bemardino Special Services** Department

Driving Weight /Drop: 140 lbs./ 30 in.

Surface Elevation(ft):

Logged by: JMcK

Measured Depth to Water(ff): N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | FIELD MOISTURE (%) | DRY UNIT WT. (pcf) | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | (SM) Silty Sand, fine with medium, trace gravel to 2", light brown | Fill | | X | | 2.9 | | |
| | | | | X | | 50 | | 121 | Ring |
| 5 | | | | X | | 40<br>49 | | 125 | Ring |
| | | | C1en6'm<br>7' | | | | | | |
| | | (SM) Silty Sand, fine with medium, trace gravel to 1/2", red brown | Native - Qa | | X | | 5.2 | | |
| 10 | | | Rock in shoe | X | | 11<br>14<br>17 | 2.8 | Dist. | Ring |
| | | (SM) Silty Sand, fine to medium, with gravel, light brown | | | X | | 4.0 | | |
| 15 | | | | X | | 30<br>50/4" | 2.2 | 118 | Ring |
| | | **END OF BORING**<br><br>NO REFUSAL, NO CAVING<br>NO GROUNDWATER, NO BEDROCK<br>EILL TO 9' | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |



10331-3 14095-3.GPJ CHJ.GDT 3/18/14

# EXPLORATORY FRENCH NO. 11

Date Excavated: 2/20/14

Client: County of San Bemardino special Services Departinmt

Equipment: Rubber Tire il8nkhog

Buciœt Size: 18" *Bucket*

Surface Elevation(A):

**Logged by: JMcK**

Station No.: N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARK | SAMPLES DEWY | COMP. (%) | ø FB LD | " MOISTURg (%) | S 1S3I |
|---|---|---|---|---|---|---|---|---|
| | | (SM) Silty Sand, fine to coarse, with gravel, bedded, light | Native - Qa | | | | | |
| 1 | | | | | | | | |
| 2 | | | | | | | | |
| 3 | | | | | | | | |
| 4 | | Detiriz *le* ur sœhments, (SM) Stlty Sand, line to coaise, with giavel, cobbles to 8" in size, and few boulders to 20" in size, light brown | | | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |
| 8 | | INIT Ul•'iltBNCH | | | | | | |

NO REFUSAL, NO GROUNDWATER
NO CAVING??, NO BEDROCK
NO EILL

NOT FOR BID

14 0BS4 spJC sjbgt

CHJ

SNOWDROP
mmcuocvcrowc*c o

Job No. 14095-3

ilnclosure B-11

# EXPLORATORY BORING NO.12

Date Drilled: 2/18/14
Clieot: Couoty o:FSao Bemazdiao Special Services Depaztazsztt

Equipment: CME 75 Tnick Rig
Dzivizig Weigkt / Dzop: t40 lbs./ 30 io.

Surface Elevation(A):
Logged by: JMcK
Measured Depth to Water(A): NfA

| EPTH (ft) | HIC | VISUAL CLASSOICATION | MARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | MOISTURB (%) | DRY ł P [TWT. bS | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | .; | taM) auiy and. ri•e, witn clay, light brown | Fill | | | | 7.4 | | |
| | | | Drive on | | | 50f4" | 4.5 | g6 | gittg |
| 5 | | (SM) Silty S'and, Dae ii •um.. witD clay, txt down | | | | | 7.7 | | |
| | | | | | | 24 35 42 | N.R. | N.R | IUng |
| 10 | | (SM) Silty Sand, fine to coarse, with clay and angular gravel to 1/2", red brown | | | | 3 5 | 13.0 9.7 | 116 | Ring |
| 15 | | (SC) Clayey Satid, fire to coarse, brown | | | | 4 5 | 16.8 | 112 | Ring |
| 20 | | (SC) Cilayey sand, fine to coarse, with silt, red blows, plastic | Native - Qvof | | | 8 6 | 17.6 | 113 | Ring |
| 25 | | | | | | 6 | 15.5 | 106 | DS, Ring |
| | | | | | | 7 7 | | | |
| 30 | | (SM) Stlty '»tt+, lime a co witb clay, browo | | | | 3 16 | | | |

END OF BORING

NO REFUSAL, NO CAVING
NO GROUNDWATER, NO BEDROCK
FILL TO 20'

10331-3 14095-3.GPJ CHJ GDT 3/18/14

CHJ

SNOWDROP

CAMONG C IUflA 20 10.7 It7 géjg

# EXPLORATORY BORING NO.13

Job No. Enclosure
was-3 B-i2

# EXPLORATORY BORING NO. 13

Date Drilled: 2/18/14

Equipœent: CME 75 Truck Rig

Client: County of San Bemardino Special Services Departiiient

Driving Weight / Drop: 140 1bs./ 30 in.

Surface Elevation(ft):

Logged by: JMcK

Measured Depth to Water(ft): N/A

| DRPTH (A) | LOG | VISUAL CLASSIFICATION | | | BULK | BLOW8 /6 IN. | MOISTURE (%) | DRY UNIT WT. | LABFIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | (SM) Silty Sand, fine to medium with clay and weathered gravel to 1/2", light brown | Fill | | | 12 17 20 | 8.7 | 120 | Ring |
| 5 - | | | | | | 5 6 7 | 7.9 | (II | Ring |
| | | (SM) Silty Sand fine to coarse, brown | Native - Qa | | | 16 11 10 | 10.6 8.3 | 108 | Ring |
| —10 | | | | | | | | | |
| 15 - | | | | | | 15 22 | 6.8 | 131 | Ring |
| 20 | | | | | | 5 6 | 6.5 | 112 | Rigg |
| - 25 | | | | | | 4 | 4.1 | t21 | Gag |
| | | | | | | 46 50/3" | 10.3 | 113 | Ring |
| 30 | | (SM) Silly Sand, fine to coaise, with gravel ana combles, light brown | | | | | | | |

HM]OFBOWNG
NOREFU NOCA G
NOGROUNDWA NOBEDRO€K
WLLTO10'

10331-3 14005-3.GPJ CH GDT 3f B14

**EXPLORATORY BORING NO. 14**

# EXPLORATORY BORING NO.14

Date Drilled: 2/18/14

Client: County of San Bemardino Special ServiceS Department

Equipment: CME 75 Truck Rig

**Dziviag Weigbt / Dzop: 140 lbs.**/ *TO!zi.*

Surface Elevation(ft):

**£oggedby: JMcK**

Measured Depth to water(ft): N/A

| DEPTH (ft) | LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6IN. | FIELD | DRY WT. | LAB TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | yAsptialt Concrete, 2-1/2" | Asphalt on Native/Pm | | | | | | AC, Ring |
| | | Garcias bedrock, fo1i•* - and weathered, retunis as (SM) Silty Sand, line with medium, with clay, light brown | | | | 18 28 **50/4"** | SP | **129** | SA |
| 5 | | | **Auger Chatter** | | | **50/4"** | | | |
| | | | | | | t7 | 3.I | j3j | Ri,,g |
| 10 | | | | | | 30 | 5.5 | 116 | **Ring** |
| 1:3 | | NO REFUSAL, NO CAVING<br>NO GROUNDWATER, NO FILL<br>GNEISS BEDROCK AT **0.21'** | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

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NCHO CUCAMONGA, CALIFORNIA

Job No. Enclosure
**14095-3 B-14**

EXPLORATORY BORING NO.15

# EXPLORATORY BORING NO. 15

Date Drilled: 2/21/14

Equipment: CME 75 Truck **Rig**

*Ck* enL *C:J une* of San Bemardino Special Seivices Department

**Drivi9g Weight** / Drep: 140 lbs./ 30 in.

**Logged by: JMcK**

Measured Depth to Water(ft): N/A

| DEPTH (g) | GRAPlaC kOG | WSUALCLASSIMCAMON | MARKS | SAMPLES DRIVE | SAMPLES BUL_K | BLOWSf6 IN. | MOISTURB (%) | DRY UNIT WT. | LAB/£ŒLD ÆST |
|---|---|---|---|---|---|---|---|---|---|
| | | \Aapbfilt C'anctet», 1-3/4" | Fill | | X | | 4.4 | | |
| | | (Slv{) Sftty Sazid, gae witb o›edium, Few gravel to 1/2", light brown | | X | | 6 7 | 3.6 | 109 | Ring |
| 5 | | | | X | | 6 6 | 4.2 | 99 | |
| 10 | | Gneıss bedrock returned as (SM) Sılty Sand, fine, lıght brown | Native - Pm | X | | 50/5" | 2.8 | 113 | Ring |
| | | | | | X | | 2.7 | | |
| | | | | X | | 50/5" | 2.2 | 119 | Ring |

**EMDUFBURING**

NO REFUSAL, NO CAVING
NO GROUNDWATER, NO EILL
GNEISS BEDROCK AT 10'

20

25



CHJ.GDT

# EXPLORATORV BORING NO. 16

Date Drilled: 2/21/14

Equipment: CME 75 Truck Rig

Surface Elevation(ft):

Client: County of San Beniardino Special Services Department

Driving Weight /Drop: 140 lbs./ 30 in.

**Logged by: JMcK**

Measured Depth to Water(fl): N/A

| GRAPHIC | VISUAL CLASSIFICATION | REMARKS | SAMPLES BULK | BLOWS/6 IN. | F IBLE | | LAB TESTS |
|---|---|---|---|---|---|---|---|
| | **Fill material, 3.5"** pA, pbatt concrete, I-7f6 | Fill Native - Pm | | | 6.0 | | AL, SA DS, Ring |
| | (SC) Clayey Sand, fine to coarse, with silt, yellow brown | | | 13 12 | 6.4 | 111 | |
| 5 | | | | 12 IS 12 | 6.7 | f1 | ) tjg |
| 10 | **Gosiss bedrock zetumod as (SM) SJty Saad, ttae to zixxiiuziz, witk angular gravel to 3/4", light browo, moderately weathered** | Auger Chatter | | | | | SNOWDROP |
| 15 | | | | | | | |

i:IND OF BURIIsIW

NO3TTUS NOCA G
NOGRO WA #IRLTO0.45'
SB R KAT7'

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Bjn
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3s 2.5

# EXPLORATORV BORING NO. 16



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# EXPLORATORY BORING NO. 17

Date **milled: 2/21/14**

Client: Coimty of San Beniaidino Special Services Department

Equipment: CME 75 Tnick Rig

Driving Weight / Drop: 140lbs. / 30 N

Surface Elevation(ft):

Logged by: JMcK

Measured Depth to Water(A): N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | FIELD MOISTURE (%) | DRY UNIT WT. (pcf) | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | Asphalt Concrete, 2-1/4" | Base on Native Pm | | | | | | |
| | | Aggregate Base, 3-7/8" | | X | | 22 | 5.0 | 118 | Ring |
| | | Gneiss as (SM) Silty Sand, fine with medium, light brown | | | | | | | |
| 5 | | | | X | | 50f5" | 5.1 | 107 | Ring |
| | | END OF BORING | | | | | | | |
| 10 | | NO REFUSAL, NO CAVI1'IG<br>NO GROUNDWATER, NO FILL<br>GNEISS BEDROCK AT 6" | | | | | | | |
| 15 | | | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |



10331-3 14095-3.GPJ CHJ.GDT 3/18/14

# EXPLI3RATORY BORING NO. 18

Dae **chilled: 2/21/14**

Clieon Gounty of San Beniardino Special Services Department

**E‹juipomnt:** CME 75 Tnick Rig

Driving Weight / Drop: 140 lbs. / 30 in.

**Surface Elevation(ft):**

Logged by: JMcK

Measured Depth to Water (ft): N/A

| DEPTH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | FIELD MOISTURE (%) | DRY UNIT WT. (pcf) | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | **Asphalt Concrete, 2-1/8"** | **Base on** | | | | | | |
| | | **Aggrgat** Base, | **Native Pm** | | | 20 | 5.4 6.2 | 119 | Ring |
| | | **Gneiss ed as (SM) Silty Sand, fine with medium, with angular gravel to 1", light brown** | | | | 50/3" | | | |
| 5 | | | | | | | | | |
| | | **END OF BORING** | | | | | | | |
| 10 | | NO REFUSAL, NO CAVING<br>NO GROUNDWATER, NO EILL<br>GNEISS BEDROCK AT 6" | | | | | | | |
| 15 | | | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |



10331-3 14095-3.GPJ CHJ.GDT 3/18/14

CHJ

SNOW DROP
RANCHO CUCAMONGA, CALIFORNIA

Job No. £lnclosure
ices-i B-18

# EXPLORATORY BORING NO. 19

Date Ddfleé: &21l4

**Clicat Co«aty o£San Bemardiao Special Services Depaztzaeat**

Equipacot: CNfl3 75 Tzuck Ifig

**Driving Wei@t/ Drop: 140 the./ 30 ia.**

Surface Elevatiozt(ft): **MqMby: P4cK** Measured Depth to Water(ft): NfA

| DEPTH (fl) | GRAPIRC ***G | VISUAL CLASSIFICATION | REMARKS | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | RCELD MOSIMCE | o eY | LAB/FIELD TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | a9gular gravel to **1/2", light** brown | | | | 48 50/4.5" | 4.7 | 110 | Ring |
| 5 | | | | | | | | | |
| | | | | | | 50/S" | N.IL | H.jL | Ring |

k ill US BtIRlNG

NO REFUSAL, NO CAVING
NO GROUNDWATER, NO FILL
GNEISS BEDROCK AT SURFACE

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140B5a.GPJ

# EXPLORATORY BORING NO. 19

# EXPLORATORY BORING NO. 2ti

Date Drilled: 2/21/14

Equipment: CME 13 Truck Rig

Surface E1evati‹xi(ft):

Client: County of San Bemardino Special Services Department

Driving Weight / **Drop: 140** lbs./ 30 U

**Logged by: JMcK**

Measured Depth to Water(ff): N/A

| D BPH (ft) | GRAPHIC LOG | VISUAL CLASSIFICATION | | SAMPLES DRIVE | SAMPLES BULK | BLOWS/6 IN. | MOISTURE (%) | DRY UNIT | TESTS |
|---|---|---|---|---|---|---|---|---|---|
| | | **(SM) Silty Sand, fine** to medium with coarse, with clay, gravel, and weatered clasts of bedrock to **3", dark brown** | Native Qvof | X | X | **18** )4 17 | 12.1 | 119 | Ring |
| 5 | | | | | | | | | |
| 10 | | Clayey 8and, **fine** to coarse, few gravel, brown | | X | | 27 | 8.8 | j j4 | gjgg |
| | | **Cfaeiss bedrock zeamted as {She) Sztty Saod, lfas, wittt aogulaz giavel to 3/4", ligbt browa** | += | | | | | | |
| 15 | | | | X | | 50/5" | 6.6 | 90 | Ring |
| | | kND OF BORING | | | | | | | |
| | | NO REFUSAL, NO CAVING<br>NO GROUNDWATER, NO FILL<br>GNRISS BEDROCK AT 13' | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |



31 14005•3.GPJ CHJ.GDT

# Appendix B

Laboratory Testing Program

# APPENDIX B

## LABORATORY TESTING PROGRAM

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

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

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

### **Expansion Index**

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

**Table No. B-1, Expansion Index Test Results**

| Test Pit No. | Depth (feet) | Soil Description | Expansion Index | Expansion Potential |
|---|---|---|---|---|
| TP-02 | 1-3 | Silty Sand (SM) | 0 | Very Low |
| TP-03 | 3-10 | Silty Sand (SM) | 0 | Very Low |
| TP-04 | 6-10 | Silty Sand (SM) | 4 | Very Low |

### **Sand Equivalent**

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

**Table No. B-2, Sand Equivalent Test Results**

| Test Pit No. | Depth (feet) | Soil Description | Sand Equivalent |
|---|---|---|---|
| TP-03 | 3-10 | Silty Sand (SM) | 15 |
| TP-04 | 6-10 | Silty Sand (SM) | 24 |

## Soil Corrosivity

Two representative soil samples were tested by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Tests 643, 422, and 417, 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 the soils when placed in contact with common construction materials such as concrete and steel (if present). Test results are presented on the following table.

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

| Boring No. | Depth (feet) | pH | Soluble Sulfates (CA 417) (percent by weight) | Soluble Chlorides (CA 422) (ppm) | Min. Resistivity (CA 643) (Ohm-cm) |
|---|---|---|---|---|---|
| TP-03 | 3-10 | 7.0 | 0.0055 | 34 | 5,593 |
| TP-04 | 6-10 | 7.4 | 0.0033 | 32 | 6,678 |

## Grain-Size Analysis

To assist in classification of soils, three mechanical grain-size analyses were performed on selected samples in general accordance with the ASTM D6913 method. Grain-size curves are shown in Drawings No. B-1, *Grain Size Distribution Results* and are presented below.

**Table No. B-4, Grain Size Distribution Test Results**

| Boring No. | Depth (ft) | Soil Classification | % Gravel | % Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|
| TP-01 | 1-3 | Silty Sand (SM) | 10.0 | 65.0 | 25.0 | |
| TP-03 | 3-10 | Silty Sand (SM) | 0.0 | 63.0 | 37.0 | |
| TP-04 | 6-10 | Silty Sand (SM) | 11.0 | 66.0 | 23.0 | |

## 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 tests were conducted in accordance with ASTM Standard D1557 method. Test results are presented on Drawings No. B-2, *Moisture-Density Relationship Results,* and summarized in the following table.

**Table No. B-5, Laboratory Maximum Density Test Results**

| Boring No. | Depth (feet) | Soil Description | Maximum Dry Density (pcf) | Optimum Moisture (%) |
|---|---|---|---|---|
| TP-02 | 1-3 | Silty Sand (SM), Grayish Brown | 134.5 | 9.0 |
| TP-04 | 6-10 | Silty Sand (SM), Light Grayish Brown | 126.9 (129.7*) | 11.0 (9.9*) |

(* Rock correction: TP-04 = 10.22%)

## Direct Shear

Two direct shear tests were performed on soil samples remolded to 90 percent of the maximum dry density and optimum moisture content under soaked moisture conditions in accordance with the ASTM D3080 method. In order to prepare remolded samples, laboratory maximum dry density of soil was utilized. For each test, three samples contained in a brass sampler ring were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test results, including sample density and moisture content, see Drawings No. B-3 and B-4, *Direct Shear Test Results,* and in the following table.

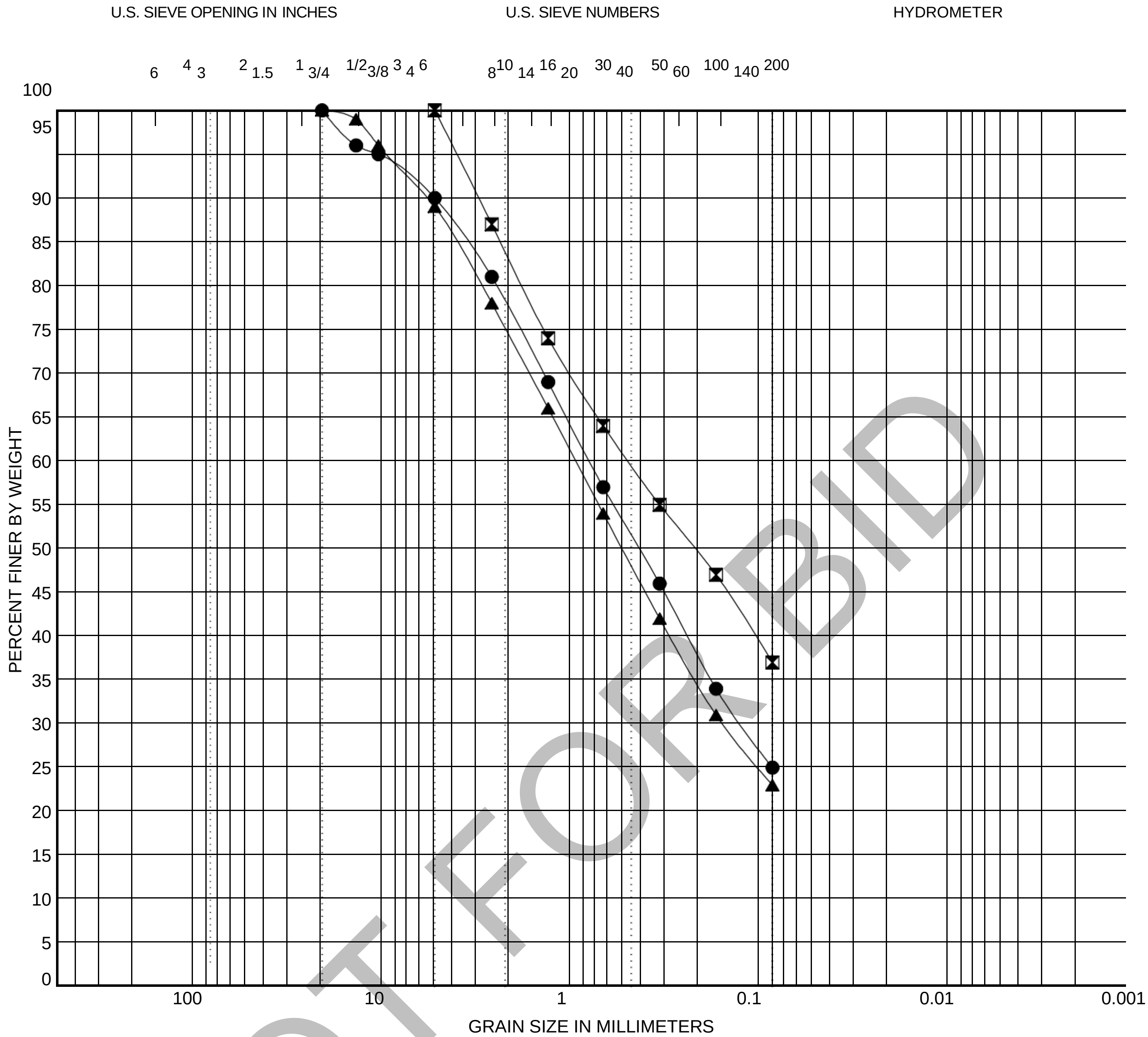
**Table No. B-6, Direct Shear Test Results**

| Boring No. | Depth (feet) | Soil Description | Ultimate Strength Parameters | |
|---|---|---|---|---|
| | | | Friction Angle (degrees) | Cohesion (psf) |
| *TP-02 | 1-3 | Silty Sand (SM) | 39 | 60 |
| *TP-04 | 6-10 | Silty Sand (SM) | 33 | 110 |

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

## 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.




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Boring No. | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **TP-01** | **1-3** | **SILTY SAND (SM)** | | | | | |
| ⧗ | **TP-03** | **3-10** | **SILTY SAND (SM)** | | | | | |
| ▲ | **TP-04** | **6-10** | **SILTY SAND (SM)** | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| | Boring No. | Depth (ft) | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **TP-01** | **1-3** | **19** | **0.711** | **0.11** | | **10.0** | **65.0** | **25.0** | |
| ⧗ | **TP-03** | **3-10** | **4.75** | **0.441** | | | **0.0** | **63.0** | **37.0** | |
| ▲ | **TP-04** | **6-10** | **19** | **0.841** | **0.138** | | **11.0** | **66.0** | **23.0** | |
| | | | | | | | | | | |
| | | | | | | | | | | |

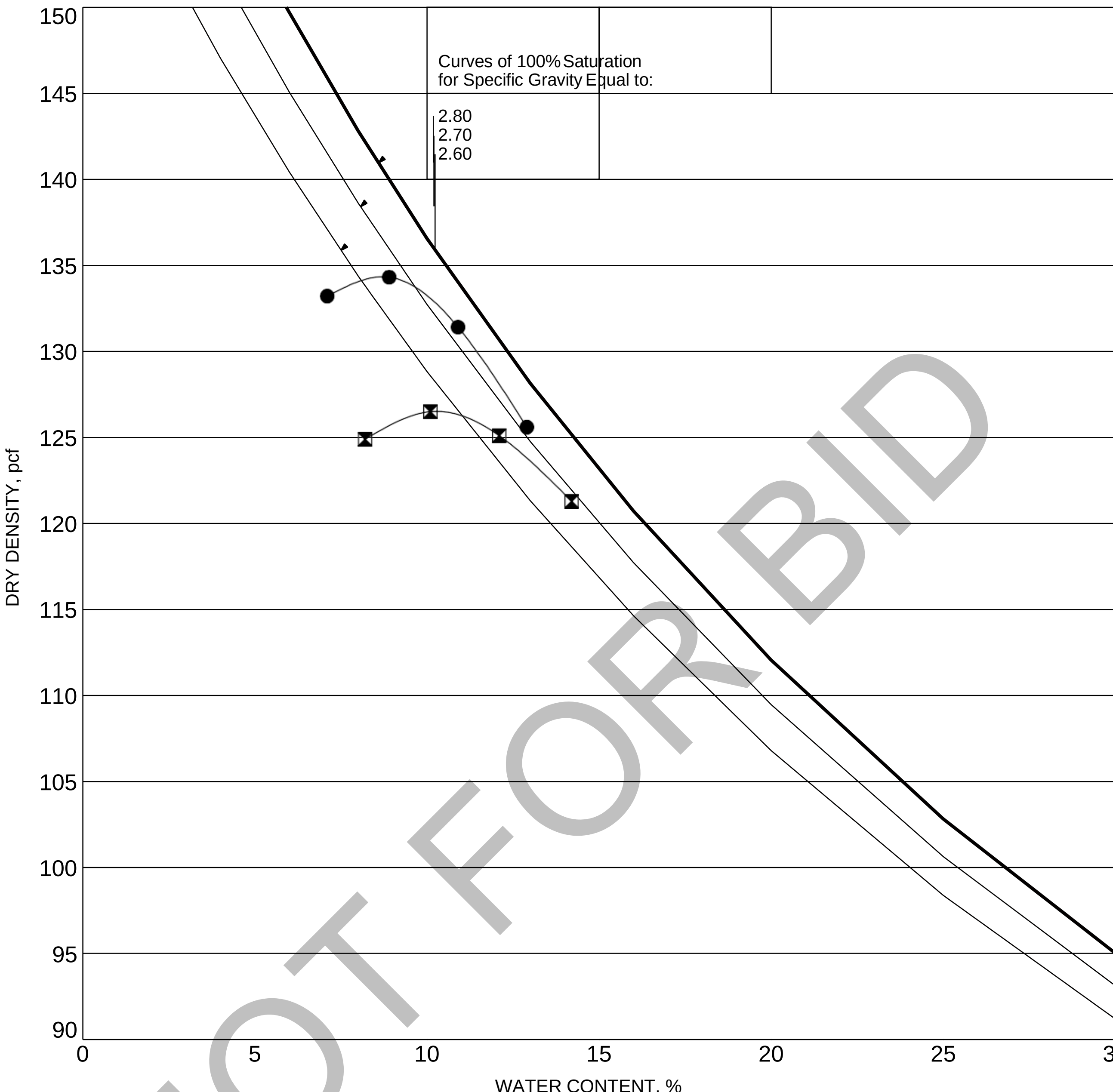
# GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Snowdrop Road Inprovement Project
Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

| Project No. | Drawing No. |
|---|---|
| **18-81-316-02** | **B-1** |

Project ID: 18-81-316-02.GPJ; Template: GRAIN SIZE






| SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf |
|---|---|---|---|---|---|---|
| ● | TP-02 | 1-3 | SILTY SAND (SM), GRAYISH-BROWN | D1557 - A | 9.0 | 134.5 |
| ⧗ | TP-04 | 6-10 | SILTY SAND (SM), LIGHT GRAYISH-BROWN | D1557 - B | 11.0 (9.9*) | 126.9 (129.7*) |
| | | | | | | |
| | | | | | | |
| | | | | | | |

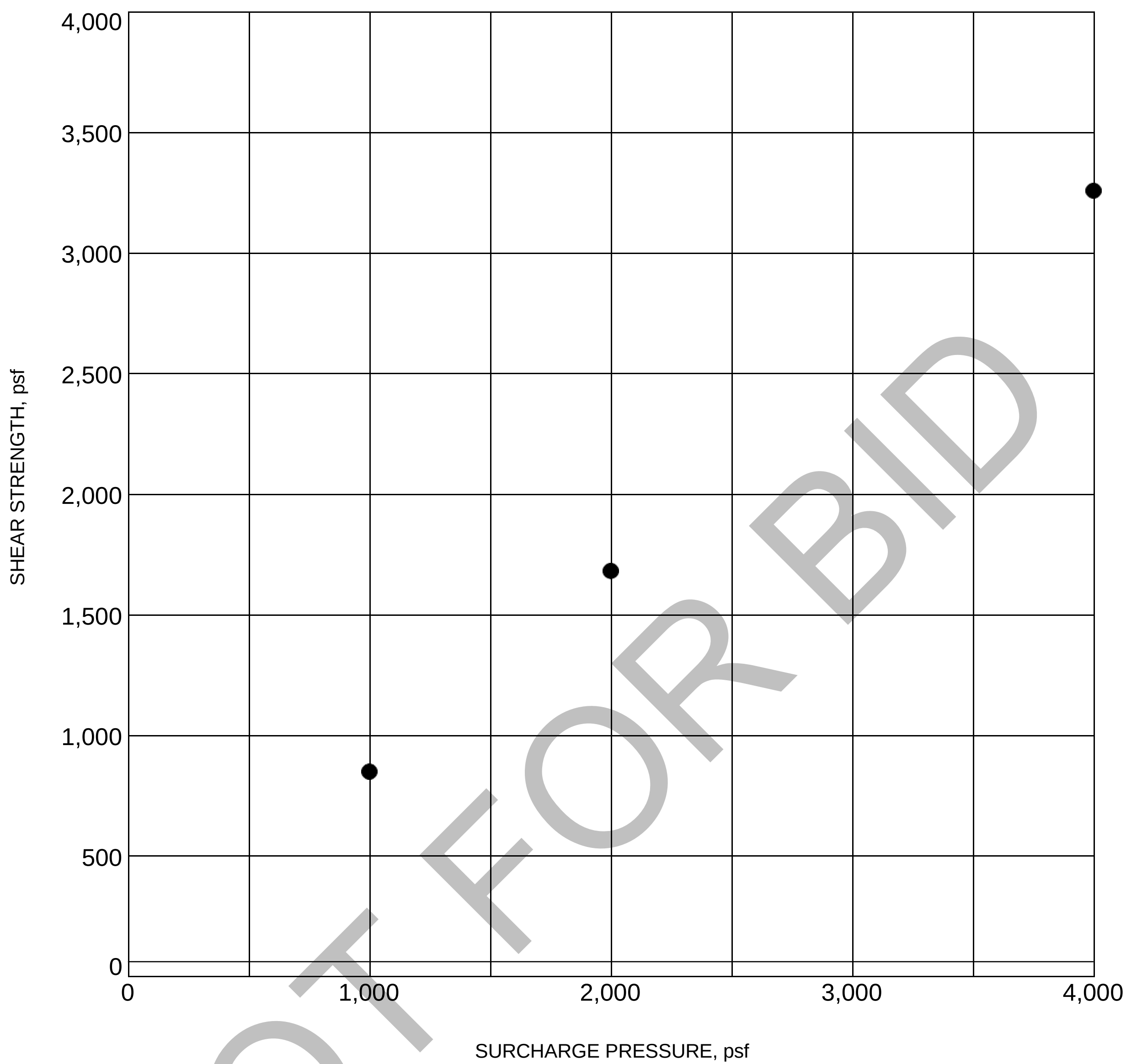
**(*Rock correction = TP-04 = 10.22%)**

# MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Snowdrop Road Inprovement Project
Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

Project No. 18-81-316-02
Drawing No. B-2






| | | | |
|---|---|---|---|
| BORING NO. : | **TP-02** | DEPTH (ft) : | **1-3** |
| DESCRIPTION : **SILTY SAND (SM)** | | | |
| COHESION (psf) : | **60.0** | FRICTION ANGLE (degrees): | **39.0** |
| MOISTURE CONTENT (%) : | **9.0** | DRY DENSITY (pcf) : | **121.0** |

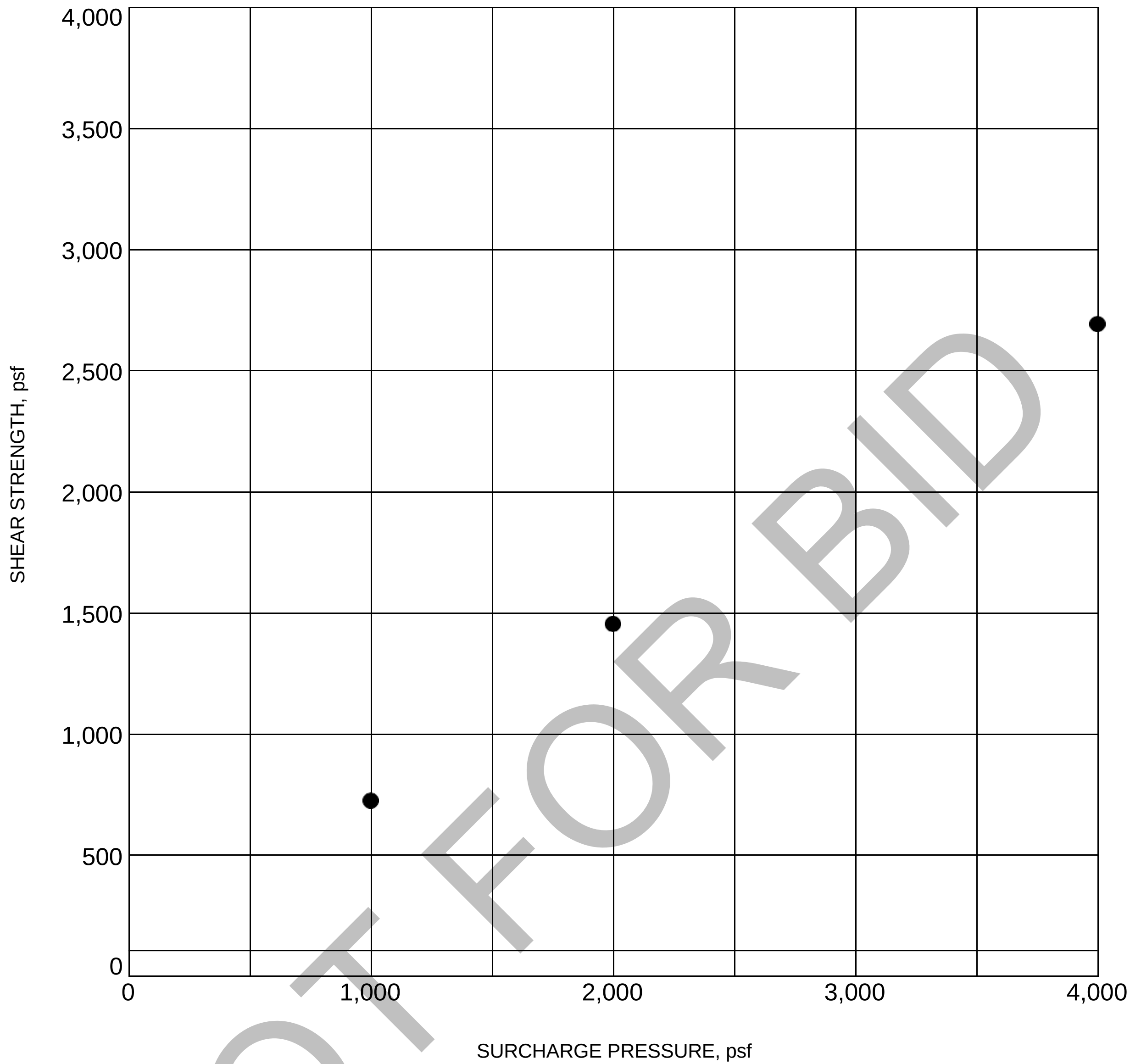
NOTE: Ultimate Strength.

# DIRECT SHEAR TEST RESULTS

Converse Consultants

Snowdrop Road Inprovement Project
Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

| Project No. | Drawing No. |
|---|---|
| **18-81-316-02** | **B-3** |






| | | | |
|---|---|---|---|
| BORING NO. : | **TP-04** | DEPTH (ft) : | **6-10** |
| DESCRIPTION : **SILTY SAND (SM)** | | | |
| COHESION (psf) : | **110.0** | FRICTION ANGLE (degrees): | **33.0** |
| MOISTURE CONTENT (%) : | **10.0** | DRY DENSITY (pcf) : | **115.0** |

NOTE: Ultimate Strength.

# DIRECT SHEAR TEST RESULTS

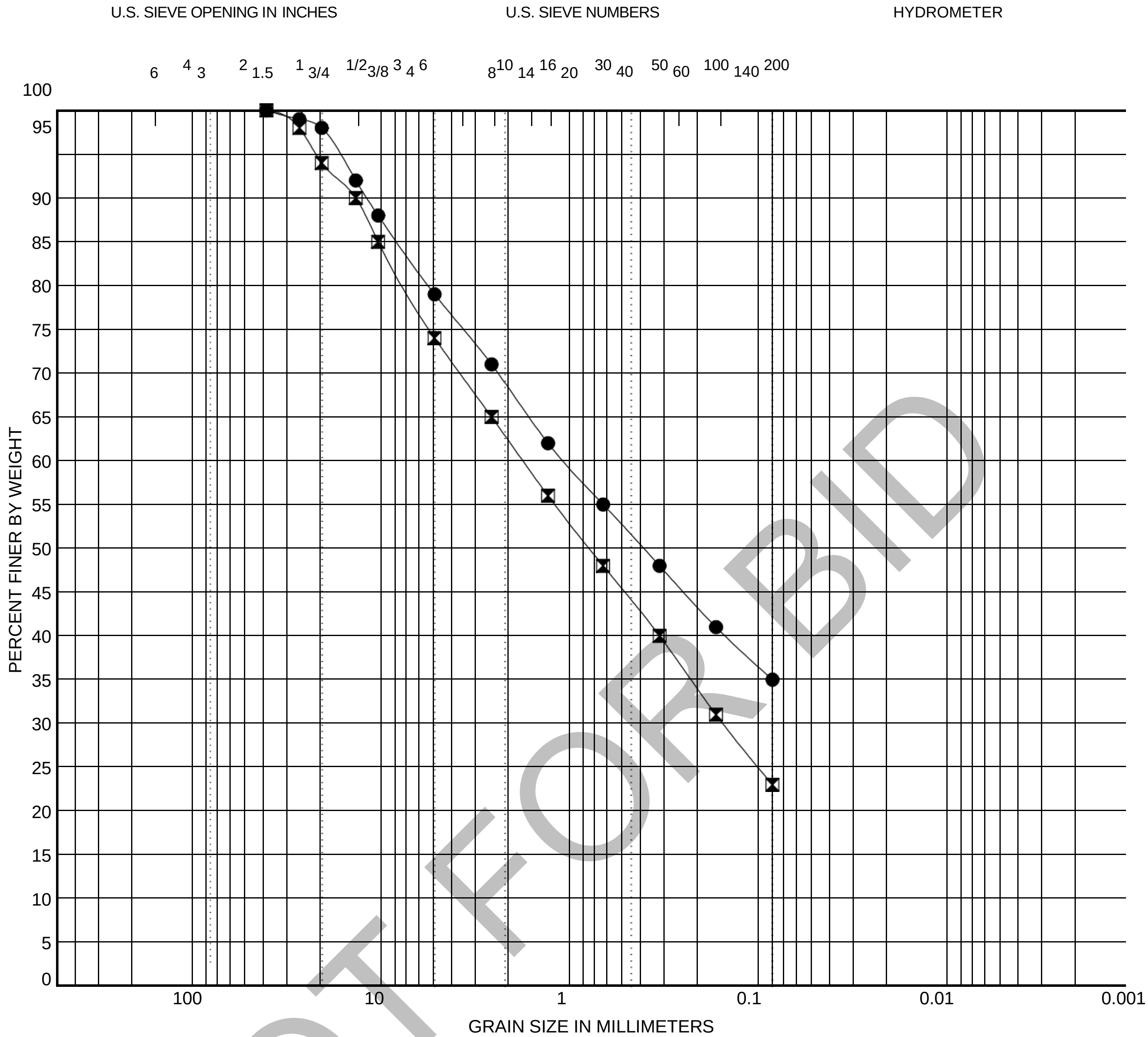
Converse Consultants

Snowdrop Road Inprovement Project
Snowdrop Road, Santina Drive, Archibald Avenue and Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
San Bernardino County Special Districts Department

| Project No. | Drawing No. |
|---|---|
| **18-81-316-02** | **B-4** |

# Appendix B-1

Laboratory Testing, Converse Consultants (1/22/19)



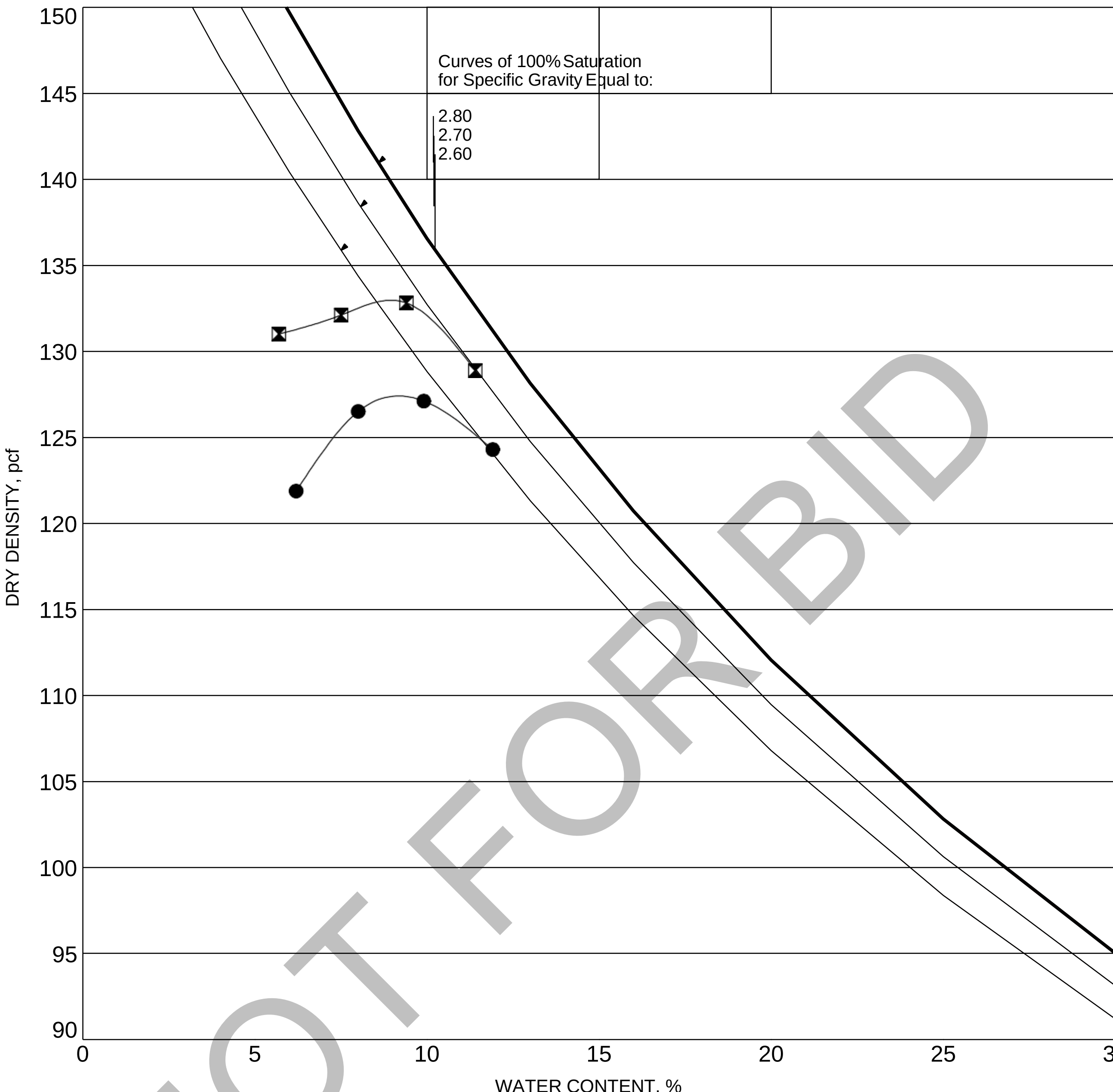



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Boring No. | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **BH-01** | **5-10** | **SILTY SAND WITH GRAVEL (SM)** | | | | | |
| ⧗ | **BH-03** | **0-5** | **SILTY SAND WITH GRAVEL (SM)** | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| | Boring No. | Depth (ft) | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **BH-01** | **5-10** | **37.5** | **0.973** | | | **21.0** | **44.0** | **35.0** | |
| ⧗ | **BH-03** | **0-5** | **37.5** | **1.606** | **0.138** | | **26.0** | **51.0** | **23.0** | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| | | | | | | | | | | |

# GRAIN SIZE DISTRIBUTION RESULTS






| SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf |
|---|---|---|---|---|---|---|
| ● | *BH-01 | 5-10 | SILTY SAND WITH GRAVEL (SM), REDDISH-BROWN | D1557 - B | 7.9 | 131.2 |
| ⧗ | *BH-03 | 0-5 | SILTY SAND WITH GRAVEL (SM), BROWN | D1557 - B | 7.2 | 138.8 |
| | | | | | | |
| | | | | | | |
| | | | | | | |

(* = rock correction; BH-01 = 11.43% rock and BH-03 = 19.01% rock)

# MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

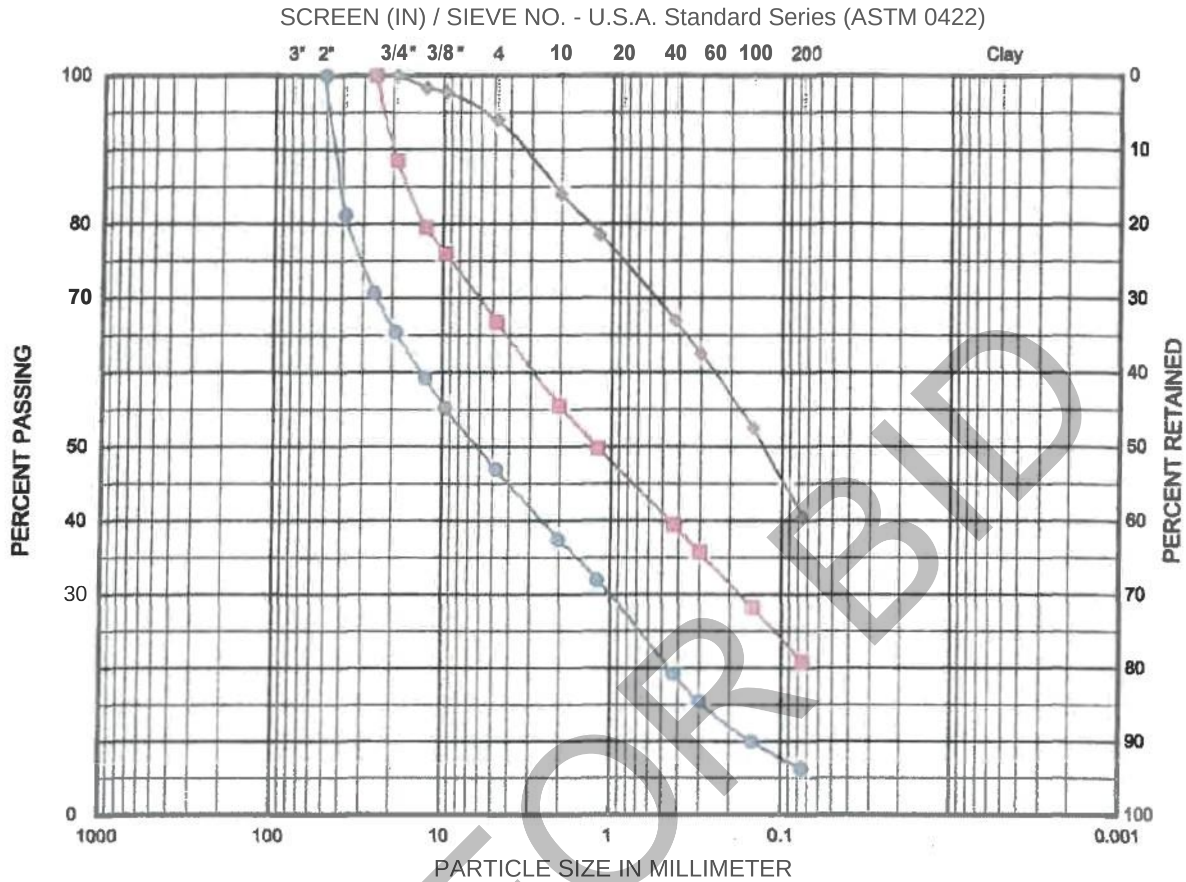
Snowdrop Road Improvement Project
Snowdrop Road, 0.4 Miles West of Haven Avenue
City of Rancho Cucamonga, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project No. **18-81-316-01**

Drawing No. **B-2**

# Appendix B-2

Laboratory Testing, CHJ Consultants (3/17/14)




| Cobbles & Boulders | Gravel | | Sand | | | Silt | Clay |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | P-inc | | |

| | Boring No. | Depth | Gravel | Sand | Fines | Clay | $D_{10}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $C_u$ | $C_c$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | 2 | 0.0 | 53.0 | 40.7 | 6.3 | | 0.1489 | 0.994 | 6.198 | 13.300 | 89.3 | 0.5 |
| | (GP-GM) Poorly-graded gravel with silt and sod | | | | | | | | | | | |
| ■ | 4 | 6.0 | 33.2 | 46.0 | 20.8 | | | 0.175 | 1.214 | 2.8M | | |
| | (SM) Silly sand with gravel | | | | | | | | | | | |
| ◆ | 5 | 1.0 | 6.1 | 53.5 | 40.4 | | | | 0.128 | 0.250 | | |
| | (SM) Silty sand, fine to coarse | | | | | | | | | | | |

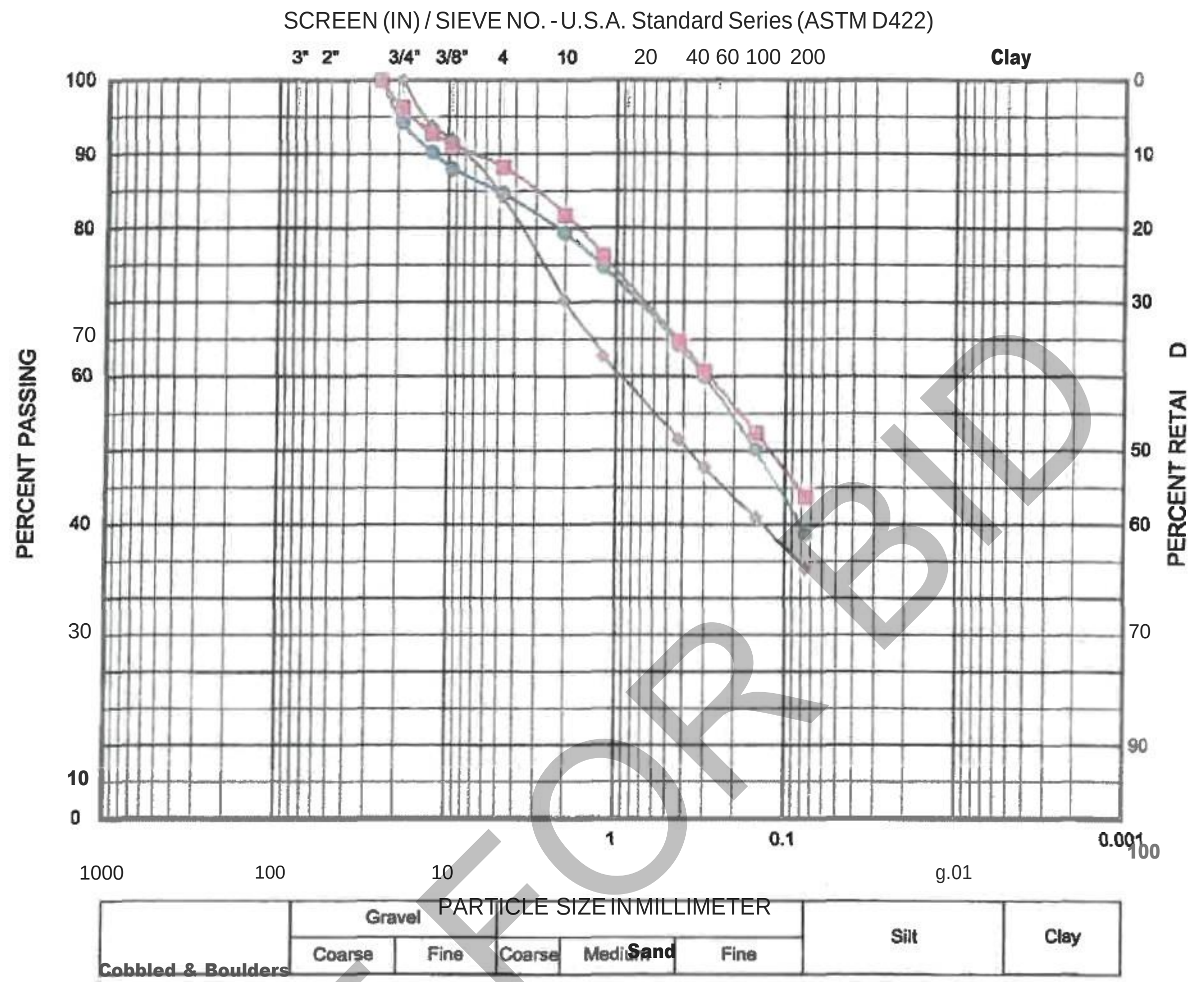
**PARTICLE SIZE DISTRIBUTION (ASTM D422)**

Job Number: 14095-3 Engineer: JFC C-1

Project: Snow Drop Road Roadway and Drainage ImprovenwnB

Location: Alta Loma Area of County of San Bemardlno. California

Endosure:




| | | Depth | Gravel | Sand | Fines | Clay | $D_{10}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $C_u$ | $C_c$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | 6 | 0.0 | 15.4 | 45.8 | 38.8 | | | 0.044 | 0.149 | 0.301 | | |
| | (SC) Clayey sand with gravel | | | | | | | | | | | |
| ■ | 12 | 21.0 | 11.8 | 44.5 | 43.7 | | | | 0.122 | 0.283 | | |
| | (SC) Clayey sand, fine to coarse | | | | | | | | | | | |
| ◆ | 14 | 0.0 | 15.8 | 50.2 | 34.0 | | | 0.051 | 0.371 | 0.932 | | |
| | (SC) Clayey sand with gravel, fine to coarse | | | | | | | | | | | |



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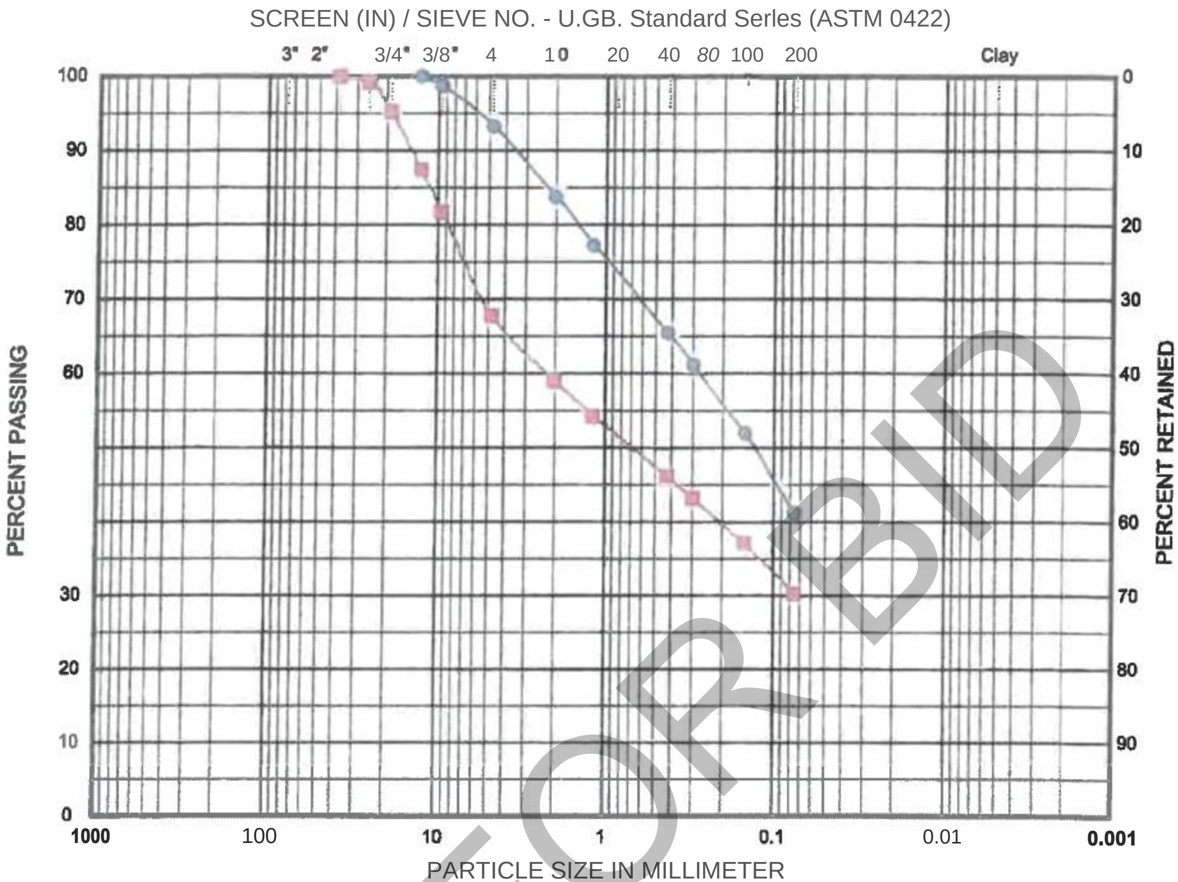
| | |
|---|---|
| Project: | PARTICLE SIZE DISTRIBUTION (ASTB D422) |
| | Snow Drop Road Roadway and Drainage Improvements |
| Location: | Alfa Loma Area of County of san Bamardino, California |
| Job Number: 14095-3 | Engineer: JFC |
| Enclosure: | C-2 |




| Cobbles & Boulders | Gravel | | Sand | | | Silt | Clay |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | | |

| | | | Gravel | Sand | Fines | Clay | $D_{10}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $C_u$ | $C_c$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | 16 | 0.0 | 6.7 | 51.9 | 41.3 | | | | 0.130 | 0.273 | | |
| | (SC) Clayey 6and, line to coarse | | | | | | | | | | | |
| ■ | 20 | 0.0 | 32.3 | 37.3 | 30.3 | | | 0.072 | 0.685 | 2.235 | | |
| | **(SM) Silty sand with gravel** | | | | | | | | | | | |



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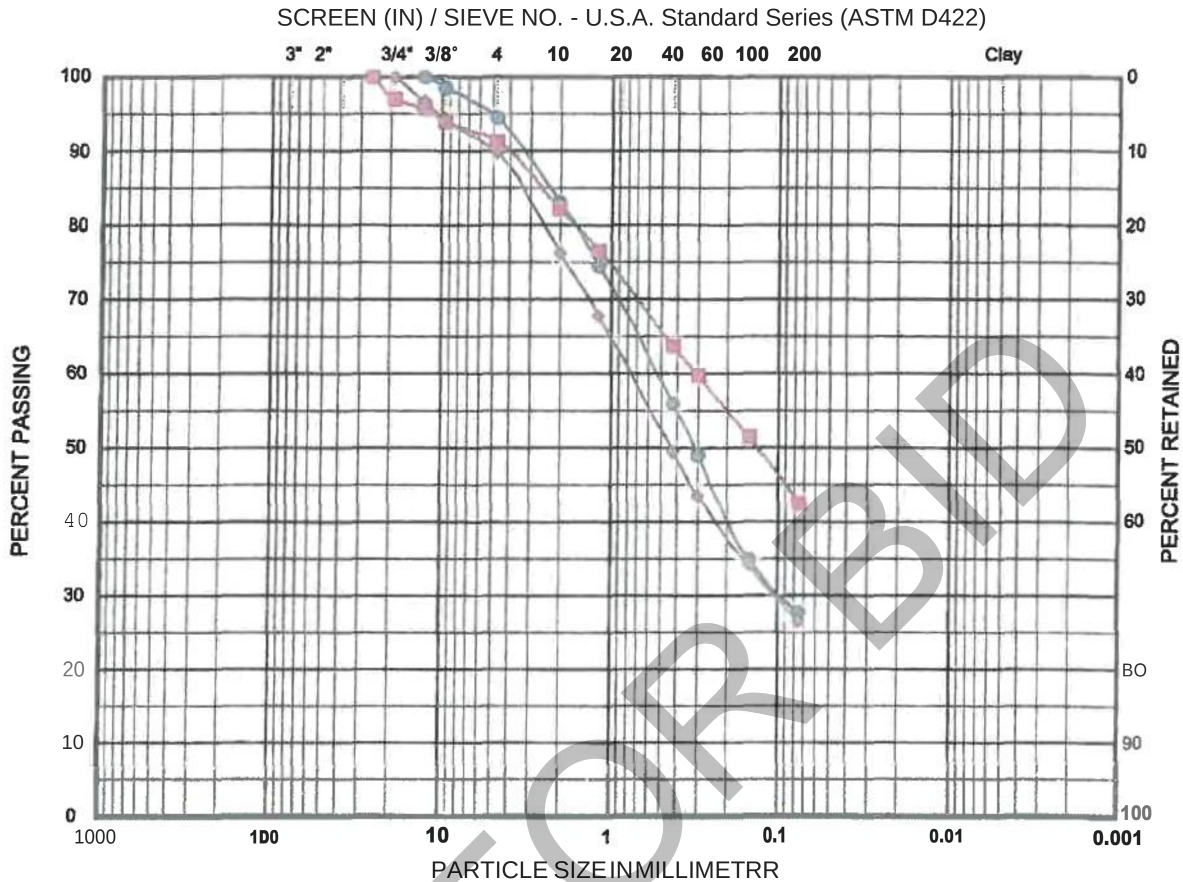
| | |
|---|---|
| | PARTICLE SIZE DISTRIBUTION (ASTI\3 0422) |
| Project: | Snow Drop Road Roadway and Drainage Improvemena |
| Location: | Alia Lorna Area of County of San Bemardino, Cagfomia |
| Job Number: 14095-3 | Engineer: JFC · Enclosure: C-3 |




| Cobbles & Boulders | Gravel | | Sand | | | Silt | Clay |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | | |

| | Boring No. | Depth | Gravel | Sand | Fines | Clay | $D_{10}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $C_u$ | $C_c$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | | 0.0 | 5.6 | 66.8 | 27.7 | | | | | | | |
| | Rv-21 | | | | | | | 0.097 | 0.314 | 0.524 | | |
| | (St4) Silty sand. gne to e›ame | | | | | | | | | | | |
| ■ | | 0.0 | 8.7 | 48.7 | 42.6 | | | | | 0.309 | | |
| | Rv-22 | | | | | | | | 0.133 | | | |
| | (SM) Silty sand, fine to coarae | | | | | | | | | | | |
| ◆ | | 0.0 | 10.2 | 63.3 | 26.5 | | | | | | | |
| | Rv-23 | | | | | | | 0.104 | 0.459 | 0.766 | | |
| | (SM) Silty sand. Gne to azarse | | | | | | | | | | | |

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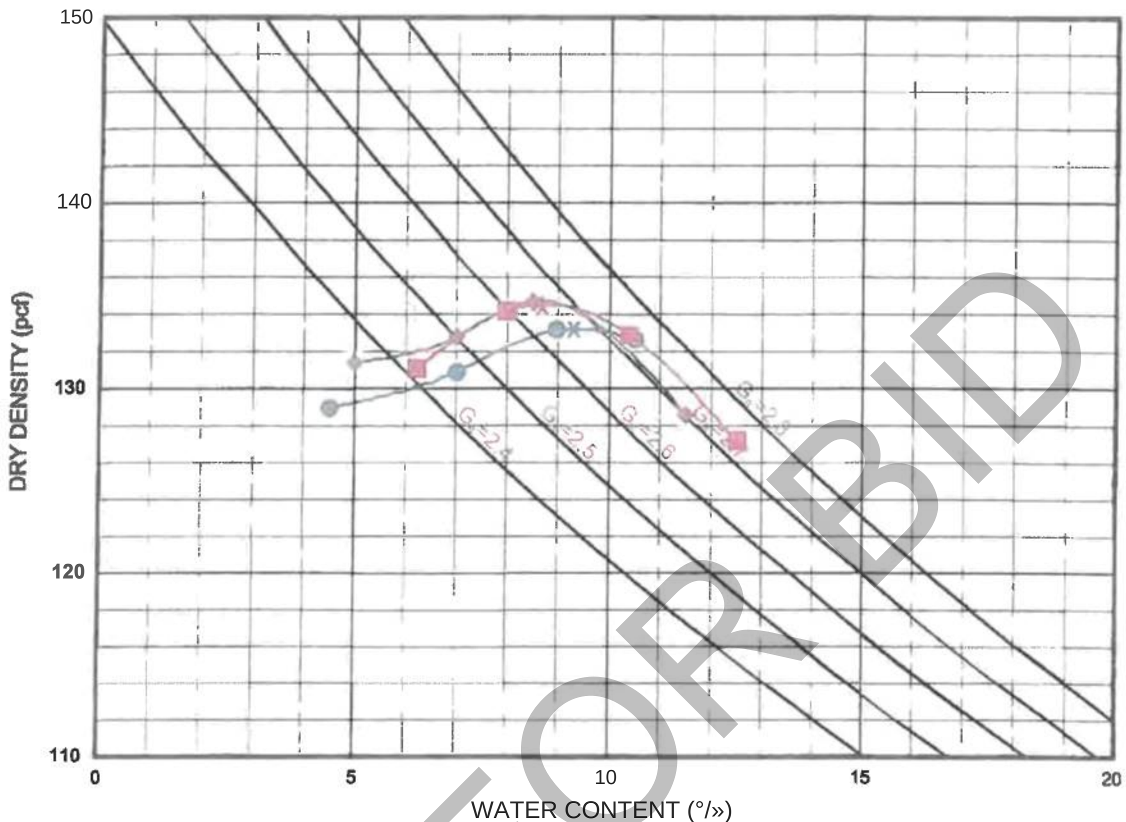
| | |
|---|---|
| Project: | PARTICLE SIZE DISTRIBUTION (ASTM D422) |
| | Snow Drop Road Roadway and Drainage Improvemana |
| Location: | Alta Loma Area of County of San Bemardino, California |
| Job Number: | 14095-3 |
| Engineer: | JFC |
| Enclosure: | C-4 |




| | | | USCS Ctassl8catioa | | | |
|---|---|---|---|---|---|---|
| | 12 | 0.0 | (SC) Clayey sand wi¥1 gravel | 17 | 23 | 6 |
| | | 21.0 | (SC) Ckyay sand, One to coarse | 20 | 27 | 7 |
| | 14 | 0.0 | (SC) Clayay sand yfiJh gzavel, Eye to coarsa | 22 | 31 | 9 |
| | 16 | 0.0 | (SC) Ciayay sand, fine to coarsa | 19 | '2 | 7 |

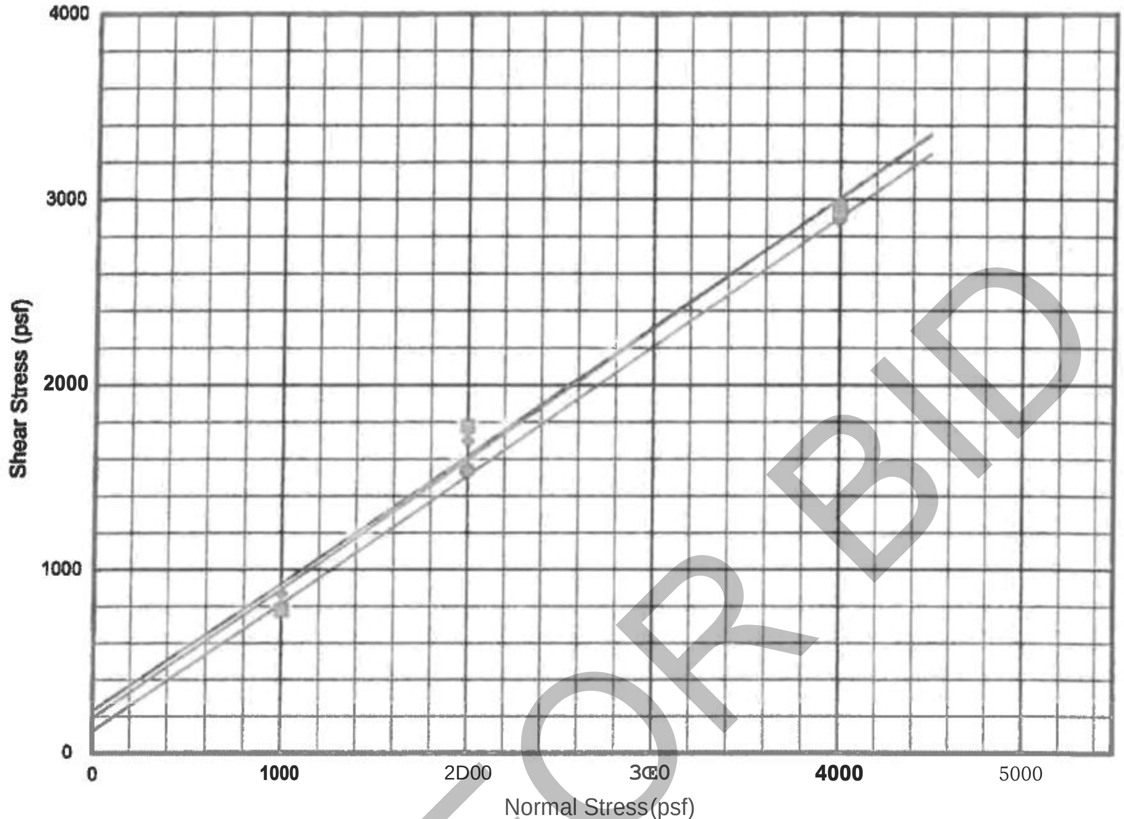



| | Boring No. | Depth (ft) | USCS Classification | $\gamma_{dmax}$ (pcf) | $w_o$ (%) |
|---|---|---|---|---|---|
| ● | 1 | 0.0 | (SM) Silty sand, fine to rzzar8e | 133.3 | 9.3 |
| ■ | 4 | 6.0 | (SM) Silty sand with grevel | 154.5 | 8.7 |
| ◆ | 9 | 0.0 | (SM) Silly sand, fine to medium | 134.7 | 8.6 |

| | |
|---|---|
| Project: | **COMPACTION CURVEG (ASTIg D1557)** |
| Location: | Snow Drop Road Roadway and Drainage Improvements |
| | Alta Lorna Area of County of San Bemardino, Calllbmia |

C-6

Job Number: 14095-3 Engineer: JFC Enclosure:




| | | Depth (ft) | $\gamma_d$ (pcf) | w (%) | $C_{pk}$ (psf) | $\phi_{pk}$ (°) | $C_{rs}$ (psf) | $\phi_{rs}$ (°) |
|---|---|---|---|---|---|---|---|---|
| ● | 1A | 1 | 120.0 | 9.0 | 308.1 | 36.8 | 115.1 | 34.9 |
| | (SM) Silly aand, fine to coaæe / Remolded (RC=90Y+) | | | | | | | |
| ■ | 5 | 1 | 121.0 | 6.8 | 109.9 | 38.6 | 190.7 | 35.2 |
| | (SM) Silty sand, fina to œarae / Undisturbed | | | | | | | |
| ◆ | 6 | 1 | 121.0 | | 323.3 | 34.1 | 230.1 | 34.7 |
| | (SC-SM) Clayey silty sand, fine to medium / Undisturbed | | | | | | | |

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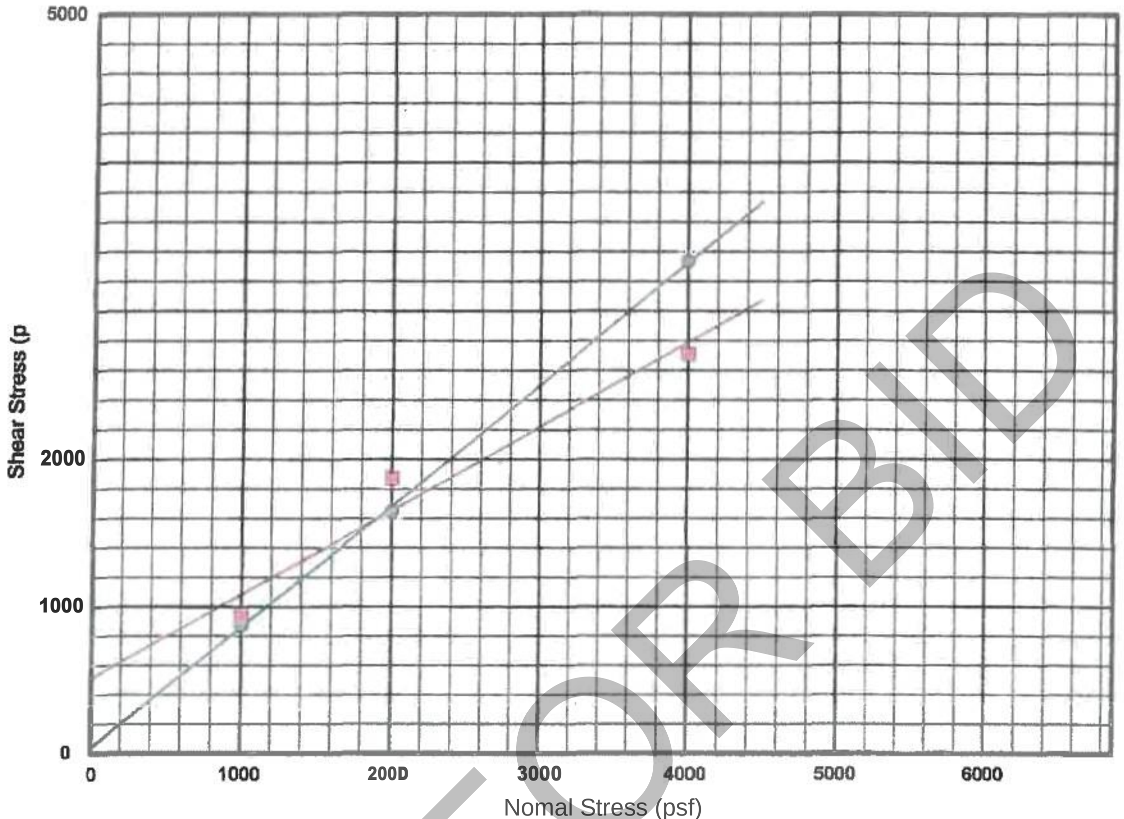
Project: **DIRECT SHEAR TESTS (ASTM D3080)**

Snow Drop Road Roadway and Dæinage Improvemena

Job Number: 14095-3 | C-7

Location: Æta Lorna Area of County of San Bemardho, California

Engineer: JFC Endosuæ:




NOT FOR BID

| | Boring No. | Depth (ft) | | | $C_{pk}$ (psf) | $\phi_{pk}$ (°) | $C_{rs}$ (psf) | $\phi_{rs}$ (°) |
|---|---|---|---|---|---|---|---|---|
| ● | 8 | 1 | 125.0 | 2.9 | 0.0 | 42.2 | 38.5 | 39.4 |
| | (SM) Silly sand, fine with madium / Undisturbed | | | | | | | |
| ■ | 12 | | 106.4 | 15.5 | 654.2 | 28.2 | | 29.6 |
| | (SC-SM) Clayey silty sand, fine to coarse / Undisturbed | | | | | | | |

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## DIRECT SHEAR TESTS (ASTM D3080)

Project: Snow Drop Road Roadway and Drainage Improvements

Location: Ala Lorna Area of County of San Bemardino, Califemia

Job Number: 14095-3 Engineer: JFC Enclosure: C-8



Prepared at 3/17/2014 3:03:49 PM

3000

1000

0 1000 2000 3000 4000 5000

Nomal Stress (psf)



| Boring No. | Depth (ft) | $\gamma_d$ (pcf) | w (%) | $C_{pk}$ (psf) | $\phi_{pk}$ (°) | $C_{rs}$ (psf) | $\phi_{rs}$ (°) |
|---|---|---|---|---|---|---|---|
| 16 | 1 | 111.0 | &4 | 142.1 | 31.0 | 168.9 | 3g.g |
| (SC-SM) Clayey silty aand, fine to coarsa / Undieturbed | | | | | | | |
| 16 | 5 | 91.0 | 6.7 | 0.0 | 39.9 | 0.0 | 39.6 |
| (SC•SM) Clayey ditty eand, fine to coaee / Undisturbed | | | | | | | |

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## DIRECT GHEAR TESTS (A9TM D3080)

Project: Snow Drop Road Roadway and Drainaga ImprovamenB

Location: Alta Lorna Area of County of san Bemardino, California

Job?umber: 14095-3 Englnes« JFC Enclosure: W9

## R•VALUE (CALTRANG 301)

| Sample No. | Rv-21 | Rv-22 | Rv-23 |
|---|---|---|---|
| **Depth (ft)** | 0.0 | 0.0 | 0.0 |
| **Classification** | SM | SC-SM | SM |
| **Sand Equivalent** | 19 | 11 | 19 |
| **R-valae** | 51 | 11 | 35 |



| TEST DATA SUMMARY | | | | | |
|---|---|---|---|---|---|
| Project: | Snow Drop Road Roadway and Drainage Improvements | | | | |
| Locafion: | Alfa Lorna Area of County of San Bemardino, California | | | | |
| Job Numbe« | 14095-3 | Engineer: | JFC | Enclosure: | C-10 |

/-/$@ <, SCHIFF

www.hd4nc.com

*Corrosion Control* and Condition Assessment /C3A] *Department*

# Table 1 - Laboratory Tests on Soil Samples

***Your #14095-3, HDR\Schiff #14-0140LAB***
***5-Mar-14***

| Sample ID | | | 6A | 14A |
|---|---|---|---|---|
| **Re8i8tivtty** | | **Units** | | |
| as-received | | OIttI1-GO1 | 17,200 | **32,000** |
| satucated | | OOO1-GO1 | 1,440 | **4,000** |
| **pH** | | | 7.3 | 5.8 |
| Electrical Conducdvlty | | **mS/cm** | **0.19** | 0.05 |
| **Cbemicet Aaatysee** | | | | |
| Cadoos | | | | |
| calcium | Cs"' | **mg/kg** | 128 | **22** |
| magnesium | Mg'° | **mg/kg** | 29 | **10** |
| sodium | Na'° | **mg/kg** | 30 | 43 |
| potassium | K" | mg/kg | 6.8 | 4.8 |
| carbonate | CO '° | mg/kg | **ND** | **ND** |
| bicarbonate | **HCO** "' | mg/leg | 159 | 34 |
| fluoride | F"' | mg/kg | **4.3** | 1.5 |
| chloride | CI'° | mg/kg | **3.6** | 23 |
| sulfate | $SO_4$"' | mg/kg | **45** | 41 |
| phosphate | POR T, | **mg/kg** | **ND** | j'jj3 |
| Otber Tesfs | | | | |
| ammonium | NJ"' | **mg/kg** | **ND** | **ND** |
| nitrate | NO " | mg/kg | **257** | **ND** |
| sulfide | S'2 | qual | **ND** | **ND** |
| **Redox** | | **mV** | **ND** | **ND** |

Electrical conductivity in milliiiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na= not analyzed



431 West Baseline Road - Claremont, CA 9I711
Phone: 909.626.0967 Fax: 909.626.3316

# Appendix C

Seismic Refraction Survey
(by Terra Geosciences)

TERRA GEOSCIENCES

# SEISMIC REFRACTION SURVEY

## SNOWDROP ROAD IMPROVEMENT PROJECT

## RANCHO CUCAMONGA, SAN BERNARDINO COUNTY, CALIFORNIA

Project No. 193289-1

October 9, 2019



**Prepared for:**

Converse Consultants
2021 Rancho Drive, Suite 1
Redlands, CA 92373

Converse Consultants
2021 Rancho Drive, Suite 1
Redlands, CA 92373

October 9, 2019
Project No. 193289-1

Attention: Mr. Robert L. Gregorek II, Senior Geologist

Regarding: Seismic Refraction Survey
Snowdrop Road Improvement Project
Rancho Cucamonga, San Bernardino County, California
Converse Project No. 18-81-316-02



## EXECUTIVE SUMMARY

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to evaluate whether high velocity bedrock materials (non-rippable) may be present. Additionally, the structure and seismic velocity distribution of the subsurface earth materials was also assessed. This report will describe in further detail the procedures used and the results of our findings, along with presentation of representative seismic models for the survey traverse.

For this study, five survey traverses were performed across the subject property, as selected by your office. The traverses were located in the field by use of Google™ Earth imagery (2019) and GPS coordinates. The approximate locations of these traverses are shown on the Seismic Line Location Map, Plate 1, of which the base map is a captured Google™ Earth image (2019).

This opportunity to be of service is sincerely appreciated. If you should have questions regarding this report or do not understand the limitations of this study or the data and results that are presented, please do not hesitate to contact our office.

Respectfully submitted,
**TERRA GEOSCIENCES**

**Donn C. Schwartzkopf**
Principal Geophysicist
PGP 1002

PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

# TABLE OF CONTENTS

## INTRODUCTION

The subject study area is located along Snowdrop and Santina Roads, in the northern portion of Rancho Cucamonga, San Bernardino County, California. The seismic traverses, as located by your firm, were placed along the northern edge of the roadway along four selected locations, where areas of proposed excavations will be performed.

Geologic mapping of the area by Morton and Matti (2001), as presented on Figure 1 below, indicates that the local bedrock is comprised of Proterozoic age (?) granulitic gneiss (map symbol Pm) that is largely retrograded to amphibolite and greenschist grade mylonite and cataclasite. Along the eastern portion of the roadway, the surficial earth materials have been mapped as being early Pleistocene age very old alluvial fan deposits (map symbol Qvof) that consist of unconsolidated to well-consolidated coarse- grained sand to bouldery alluvium. Along the bottom of the drainage courses, it is presumed that recent alluvium is present of unknown thickness and composition.

For reference, the approximate locations of the seismic traverses are indicated as the green lines in Figure 1 below.



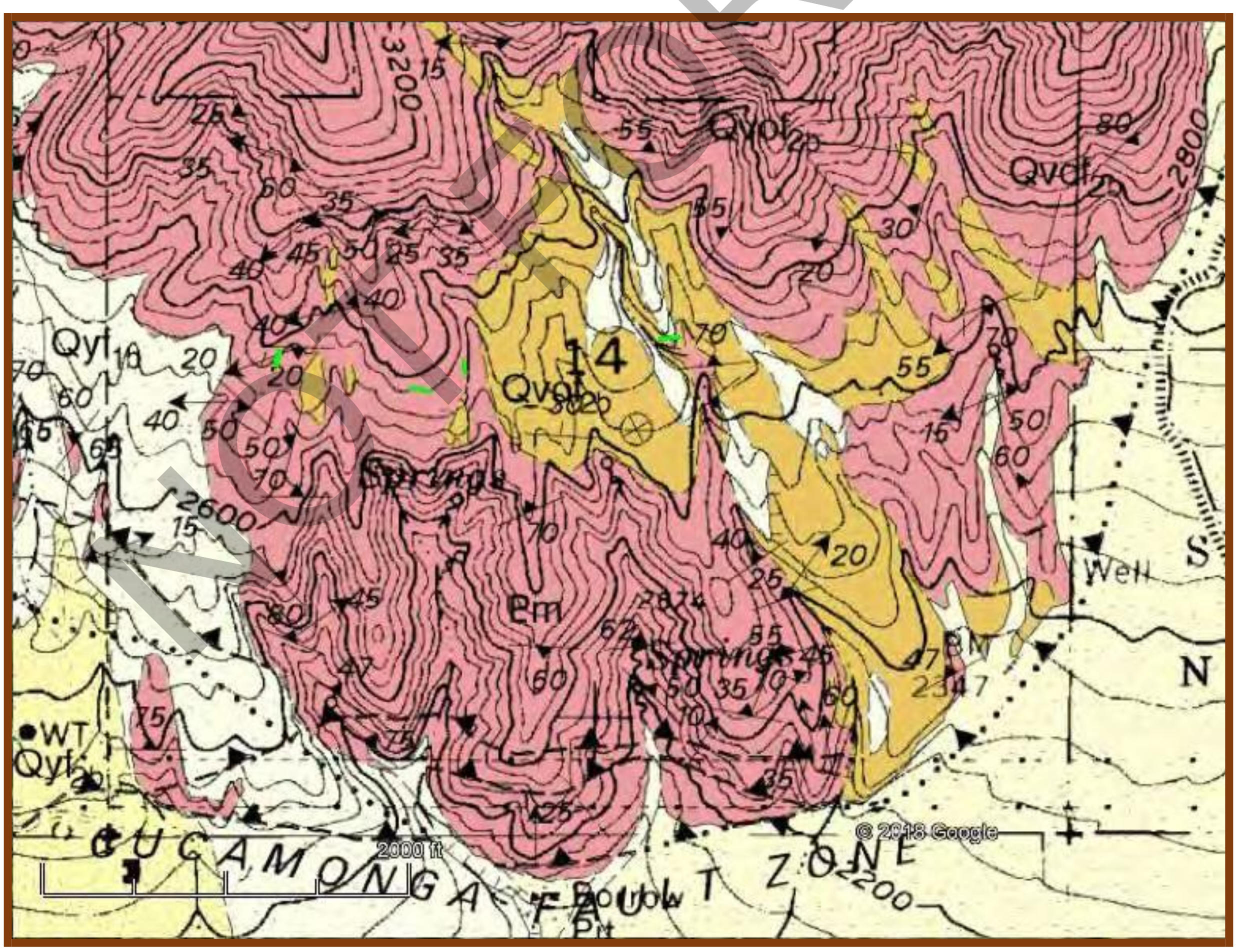



**FIGURE 1- Geologic Map (Morton and Matti, 2001), Seismic traverses shown as green lines.**

## SEISMIC REFRACTION SURVEY

### Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

### Field Procedures

Four seismic refraction survey lines (Seismic Lines S-1 through S-4) have been performed along representative areas across the subject study area as selected by you. The traverses were located in the field by use of Google™ Earth imagery (2019) and GPS coordinates and have been delineated on the Seismic Line Location Map, as presented on Plate 1. The survey traverses ranged in length from 75 to 125 feet in length, based on the available space available, where the lines need to be straight and not underlain by pavement. Each line consisted of a total of twenty-four 14-Hertz geophones, spaced at regular three- to five-foot intervals, in order to detect both the direct and refracted waves. A 16-pound sledge-hammer was used as the energy source to produce the seismic waves. Multiple hammer impacts were utilized at each shot point in order to increase the signal to noise ratio, which enhanced the primary seismic “P”-waves. The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. Seven shot points were utilized along each spread using forward, reverse, and several intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.064 to 0.080 seconds. No acquisition filters were used during data collection.

During acquisition, the seismograph displays the seismic wave arrivals on the computer screen which were used to analyze the arrival time of the primary seismic “P”-waves at each geophone station, in the form of a wiggle trace for quality control purposes in the field. If spurious “noise” was observed, the shot location was resampled during relatively quieter periods. Each geophone and seismic shot location were surveyed using a hand level and ruler for topographic correction, with “0” being the lowest point along each survey line.

## Data Processing

The recorded seismic data was subsequently transferred to our office computer for processing and analyzing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004); **Refractor** (Geogiga, 2001-2018); and **Rayfract™** (Intelligent Resources, Inc., 1996-2019). All of the computer programs perform their individual analyses using exactly the same input data, which includes the first-arrival times of the “P”-waves and the survey line geometry.

- **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the “Seismic Refraction Modeling by Computer” method (Scott, 1973). The first step in the modeling procedure is to compute layer velocities by least-squares techniques. Then the program uses the delay-time method to estimate depths to the top of layer-2. A forward modeling routine traces rays from the shot points to each geophone that received a first-arrival ray refracted along the top of layer-2. The travel time of each such ray is compared with the travel time recorded in the field by the seismic system. The program then adjusts the layer-2 depths so as to minimize discrepancies between the computed ray-trace travel times and the first arrival times picked from the seismic waveform record. The process of ray tracing and model adjustment is repeated a total of six times to improve the accuracy of depths to the top of layer-2. This first-arrival picks were then used to generate the Layer Velocity Models using the **SIPwin** computer program, which presents the subsurface velocities as individual layers and are presented within Appendix A for reference. In addition, the associated Time-Distance Plot for each survey line, which shows the individual data picks of the first “P-wave” arrival times, also appears in Appendix A.

- **Refractor** is seismic refraction software that also evaluates the subsurface using layer assignments utilizing interactive and interchangeable analytical methods that include the Delay-Time method, the ABC method, and the Generalized Reciprocal Method (GRM). These methods are used for defining irregular non-planar refractors and are briefly described below. The <u>Delay-Time</u> method will measure the delay time depth to a refractor beneath each geophone rather than at shot points. Delay- time is the time spent by a wave to travel up or down through the layer (slant path) compared to the time the wave would spend if traveling along the projection of the slant path on the refractor. The <u>ABC</u> (intercept time) method makes use of critically refracted rays converging on a common surface position. This method involves using three surface to surface travel times between three geophones and the velocity of the first layer in an equation to calculate depth under the central geophone and is applied to all other geophones on the survey line. The <u>GRM</u> method is a technique for delineating undulating refractors at any depth from in-line seismic refraction data consisting of forward and reverse travel-times and is capable of resolving dips of up to 20% and does not over-smooth or average the subsurface refracting layers. In addition, the technique provides an approach for recognizing and compensating for hidden layer conditions.

- **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves which generally indicates the relative structure and velocity distribution of the subsurface using first break energy propagation modeling. An initial 1D gradient model is created using the DeltatV method (Gebrande and Miller, 1985) which gives a good initial fit between modeled and picked first breaks. The DeltatV method is a turning-ray inversion method which delivers continuous depth vs. velocity profiles for all profile stations. These profiles consist of horizontal inline offset, depth, and velocity triples. The method handles real-life geological conditions such as velocity gradients, linear increasing of velocity with depth, velocity inversions, pinched-out layers and outcrops, and faults and local velocity anomalies. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion (Schuster and Quintus-Bosz, 1993).

  WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break. This computer program performs the analysis by using the same first-arrival P-wave times and survey line geometry that were generated during the layer velocity model analyses. The associated Refraction Tomographic Models which display the subsurface earth material velocity structure, is represented by the velocity contours (isolines displayed in feet/second), supplemented with the color-coded velocity shading for visual reference, and are presented within Appendix B.

The combined use of these computer programs provided a more thorough and comprehensive analysis of the subsurface structure and velocity characteristics. Each computer program has a specific purpose based on the objective of the analysis being performed. **SIPwin** and **Refractor** were primarily used for detecting generalized subsurface velocity layers providing “weighted average velocities.” The processed seismic data of these two programs were compared and averaged to provide a final composite layer velocity model which provided a more thorough representation of the subsurface. **Rayfract™** provided tomographic velocity and structural imaging that is very conducive to detecting strong lateral velocity characteristics such as imaging corestones, dikes, and other subsurface structural characteristics.

## SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin our discussion, it is important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (foliation, fracturing, relic bedding, etc.), creating anisotropic conditions. Anisotropy (direction-dependent properties of materials) can be caused by “micro-cracks,” jointing, foliation, layered or inter-bedded rocks with unequal layer stiffness, small-scale lithologic changes, etc. (Barton, 2007). Velocity anisotropy complicates interpretation and it should be noted that the seismic velocities obtained during this survey may have been influenced by the nature and character of any localized structural discontinuities within the bedrock underlying the subject site.

Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Such variable directions can result in velocity differentials of between 2% to 40% depending upon the degree of the structural fabric (i.e., weakly-moderately-strongly foliated, respectively). Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time.

The first computer method described below used for data analysis is the traditional layer method (**SIPwin** and **Refractor**). Using this method, it should be understood that the data obtained represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders, dikes, or other local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics. In general, the site where locally surveyed, was noted to be characterized by three major subsurface layers (Layers V1, V2, and V3) with respect to seismic velocities.

The following velocity layer summaries have been prepared using the **SIPwin** and **Refractor** analysis, with the representative Layer Velocity Model presented within Appendix A along with the respective Time-Distance Plot.

- **Velocity Layer V1**:

  This uppermost velocity layer (V1) is most likely comprised of alluvium, colluvium, topsoil, artificial fill, and/or completely-weathered and fractured bedrock materials. This layer has an average weighted velocity of 1,238 to 1,618 fps, which is typical for these types of unconsolidated surficial earth materials. Fill materials are present beneath Seismic Line S-1 where the roadway crosses a stream channel.

- **Velocity Layer V2**:

  The second layer (V2) yielded a seismic velocity range of 1,801 to 2,700 fps, which is typical for very highly-weathered metamorphic bedrock materials. This velocity range may indicate the presence of homogeneous weathered bedrock with a relatively wide spaced joint/fracture system and/or the possibility of buried relatively- fresher boulders within a completely highly-weathered bedrock matrix. Additionally, older alluvial materials are also typical of this velocity range and may be present locally.

- **Velocity Layer V3**:

  The third layer (V3) indicates the presence of highly-weathered metamorphic bedrock, having a seismic velocity range of 2,346 to 4,714 fps. This velocity range may indicate the presence of homogeneous weathered bedrock with a relatively wide spaced joint/fracture system and/or the possibility of buried relatively-fresher boulders within a very highly-weathered bedrock matrix.

The following table summarizes the results of the survey lines with respect to the "weighted average" seismic velocities for each layer, as indicated on the Layer Velocity Models, presented within Appendix A.

TABLE 1- VELOCITY SUMMARY OF SEISMIC SURVEY LINES

| Seismic Line | V1 Layer (fps) | V2 Layer (fps) | V3 Layer (fps) |
|---|---|---|---|
| **S-1** | **1,618** | **1,886** | **4,714** |
| **S-2** | **1,614** | **2,700** | **3,350** |
| **S-3** | **1,253** | **1,801** | **2,346** |
| **S-4** | **1,238** | **2,107** | **2,998** |



Using **Rayfract™**, tomographic models were also prepared for comparative purposes to better illustrate the general structure and velocity distribution of the subsurface, using velocity contour isolines, as presented within Appendix B. Although no discrete velocity layers or boundaries are created, these models generally resemble the corresponding overall average layer velocities as presented within Appendix A.

In general, the seismic velocity of the bedrock gradually increases with depth, with occasional lateral velocity differentials suggesting the local presence of buried corestones and/or dike structures. The colors representing the velocity gradients have been standardized on all of the models for comparative purposes.

## GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

A summary of the generalized rippability characteristics of bedrock based on a compilation of rippability performance charts prepared by Caterpillar, Inc. (2018; see Figure 2, Page 8), Caltrans (Stephens, 1978), and Santi (2006), has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. These seismic velocity ranges and rippability potentials have been tabulated below for reference.

TABLE 2- CATERPILLAR RIPPABILITY CHART (D9 Ripper)

| Metamorphic Rock Velocity | Rippability |
|---|---|
| < 7,200 | Rippable |
| 7,200 – 9,000 | Moderately Rippable |
| > 9,000 | Non-Rippable |

Additionally, we have provided the Caltrans Rippability Chart as presented below within Table 2 for comparison. These values are from published Caltrans studies (Stephens, 1978) that are based on their experience and which appear to be more conservative than Caterpillar's rippability chart. It should be noted that the type of bedrock was not indicated.

TABLE 3- STANDARD CALTRANS RIPPABILITY CHART

| Velocity (feet/sec ±) | Rippability |
|---|---|
| < 3,500 | Easily Ripped |
| 3,500 – 5,000 | Moderately Difficult |
| 5,000 – 6,600 | Difficult Ripping / Light Blasting |
| > 6,600 | Blasting Required |

Table 3 is partially modified from the "Engineering Behavior from Weathering Grade" as presented by Santi (2006), which also provides velocity ranges with respect to rippability potentials, along with other rock engineering properties that may be pertinent.

TABLE 4- SUMMARY OF ROCK ENGINEERING PROPERTIES

| ENGINEERING PROPERTY: | Slightly Weathered | Moderately Weathered | Highly Weathered | Completely Weathered |
|---|---|---|---|---|
| **Excavatability** | Blasting necessary | Blasting to rippable | Generally rippable | Rippable |
| **Slope Stability** | ½ :1 to 1:1 (H:V) | 1:1 (H:V) | 1:1 to 1.5:1 (H:V) | 1.5:1 to 2:1 (H:V) |
| **Schmidt Hammer Value** | 51 – 56 | 37 – 48 | 12 – 21 | 5 – 20 |
| **Seismic Velocity (fps)** | 8,200 – 13,125 | 5,000 – 10,000 | 3,300 – 6,600 | 1,650 – 3,300 |

The Caterpillar D9R Ripper Performance Chart (Caterpillar, 2018) has been provided on Figure 2 below for reference.

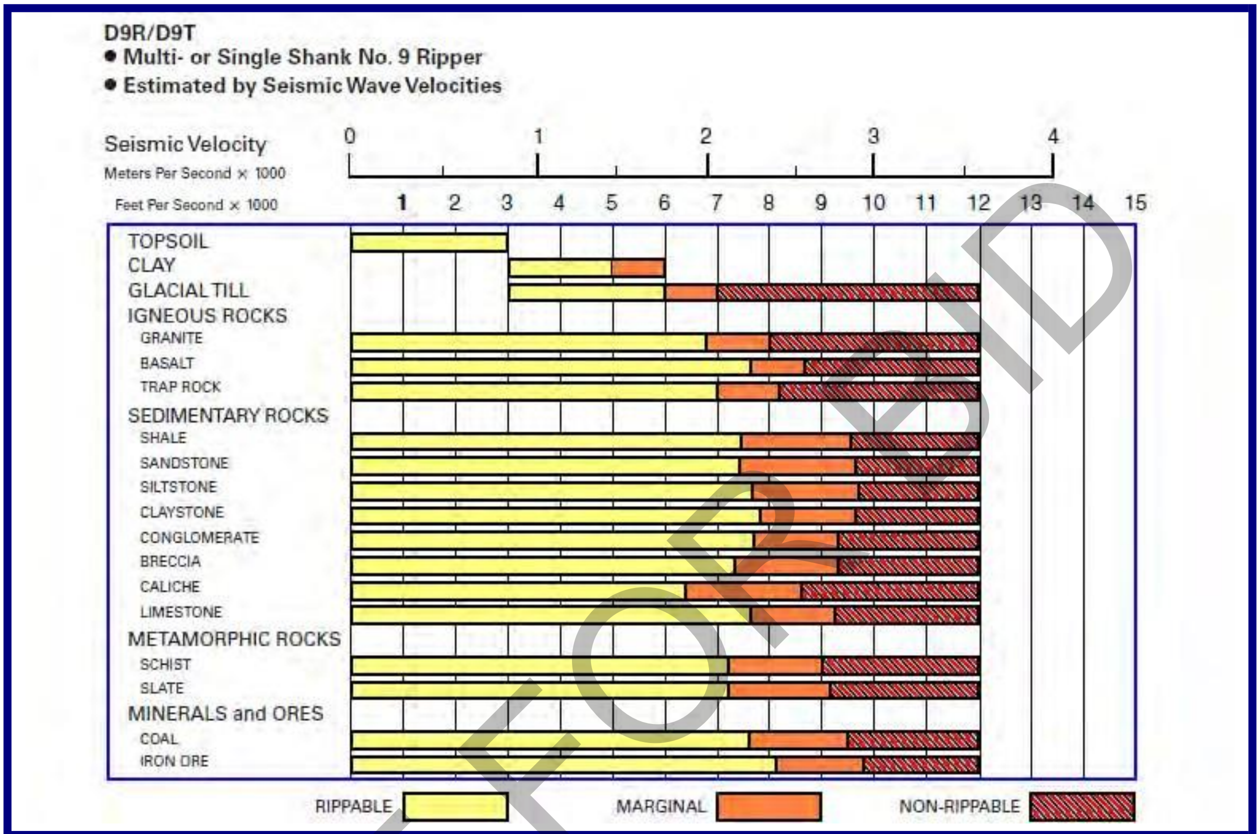



**FIGURE 2- Caterpillar D9R Ripper Performance Chart (2018).**

For purposes of the discussion in this report with respect to the expected bedrock rippability characteristics, we are assuming that a D9R/D9T dozer will be used as a minimum, such as discussed further below and as shown in Figure 2 above. Smaller excavating equipment will most likely result in slower production rates and possible refusal within relatively lower velocity bedrock materials. It should be noted that the decision for blasting of bedrock materials for facilitating the excavation process is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

A summary of the generalized rippability characteristics of bedrock (such as present within the subject study area) has been provided below to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas that were surveyed. The velocity ranges described below are general averages of Tables 2 and 3 presented in this report (see Page 7) and assume typical, good- working, heavy excavation equipment, such as D9R dozer using a single shank, as described by Caterpillar, Inc. (2000 and 2018).

However, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges as the rippability performance charts are tailored for conventional bulldozer equipment and cannot be directly correlated. Trenching operations which utilize large excavator-type equipment within granitic bedrock materials, typically encounter very difficult to non-productable conditions where seismic velocities are generally greater than 4,000± fps, and less for smaller backhoe-type equipment.

These average seismic velocity ranges are summarized below:

- **Rippable Condition (0 - 4,000 ft/sec)**:

  This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed granitic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

  **Marginally Rippable Condition (4,000 - 7,000 ft/sec)**:

  This range of seismic velocities indicates materials which may consist of moderately weathered bedrock and/or large areas of fresh bedrock materials separated by weathered fractured zones. These bedrock materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **Non-Rippable Condition (7,000 ft/sec or greater)**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

## GEOLOGIC & EARTHWORK CONSIDERATIONS

To evaluate whether a particular bedrock material can be ripped or excavated, this geophysical survey should be used in conjunction with the geologic and/or geotechnical report and/or information gathered for the subject project which may describe the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults, and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification or lamination, large grain size, moisture permeated clay, and low compressive strength. If the bedrock is foliated and/or fractured at depth, this structure could aid in excavation production.

Unfavorable bedrock conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic. Use of these physical bedrock conditions along with the subsurface velocity characteristics as presented within this report should aid in properly evaluating the type of equipment that will be necessary and the production levels that can be anticipated for this project. A summary of excavation considerations is included within Appendix C in order to provide you and your grading contractor with a better understanding of the complexities of excavation in bedrock materials, so that proper planning and excavation techniques can be employed.

## SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of good quality with minor amounts of ambient “noise” that was introduced during our survey, originating from distant vehicular and air traffic, and wind sources. Analysis of the data and picking of the primary “P”-wave arrivals was therefore performed with little difficulty, with only minor interpolation of some data points being necessary.

Based on the results of our comparative seismic analyses of the computer programs **SIPwin**, **Refractor**, and **Rayfract™**, the seismic refraction survey line models appear to generally coincide with one another, with some minor variances due to the methods that these programs process, integrate, and display the input data. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- **Velocity Layer V1**:

  No excavating difficulties are expected to be encountered within the uppermost, low-velocity V1 layer (average weighted velocity of 1,238 to 1,618 fps) and should excavate with conventional ripping. This surficial velocity layer is expected to be comprised of colluvium, alluvium, topsoil, artificial fill, and/or completely-weathered and fractured bedrock materials.

- **<u>Velocity Layer V2</u>**:

  The second V2 layer (average weighted velocity of 1,801 to 2,700 fps) is believed to consist of very highly-weathered metamorphic bedrock along with possible older alluvial fan deposits. Using the rock classifications as presented within Tables 2 through 4 and Figure 2, seismic wave velocities of less than 7,200± fps are generally noted to be within the threshold for conventional ripping. Excavation difficulties are not anticipated within this velocity layer.

- **<u>Velocity Layer V3</u>**:

  The third V3 layer is believed to consist of highly-weathered metamorphic bedrock. Moderate excavation difficulties within this velocity layer (average weighted velocity range of 2,346 to 4,714 fps) should be anticipated. This layer may consist of relatively homogeneous bedrock with wide-spaced fracturing, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. Caterpillar (2018; see Figure 2) indicates this velocity range to be "rippable" using a D9R dozer or equivalent. Placement of infrastructure within this velocity layer using excavator equipment may require some breaking and/or light blasting to obtain desired grade.

The ray sampling coverage of the subsurface seismic waves that were acquired during the processing of the tomographic models using **Rayfract™**, appeared to be of good quality which was verified by having a Root Mean Square Error (RMS) of 2.3 to 4.7 percent (see lower right-hand corner of each model). The RMS error (misfit between picked and modeled first break times) is automatically calculated during the processing routine, with a value of less than 5.0% being preferred, of which all of the models obtained.

Based on the tomographic modeling and typical excavation characteristics observed within bedrock materials of the southern California region, anticipation of gradual increasing hardness with depth should be anticipated during grading. Some lateral velocity variations should be expected to be encountered across the site generally due to the presence of buried corestones, dikes, and/or lithologic variabilities.

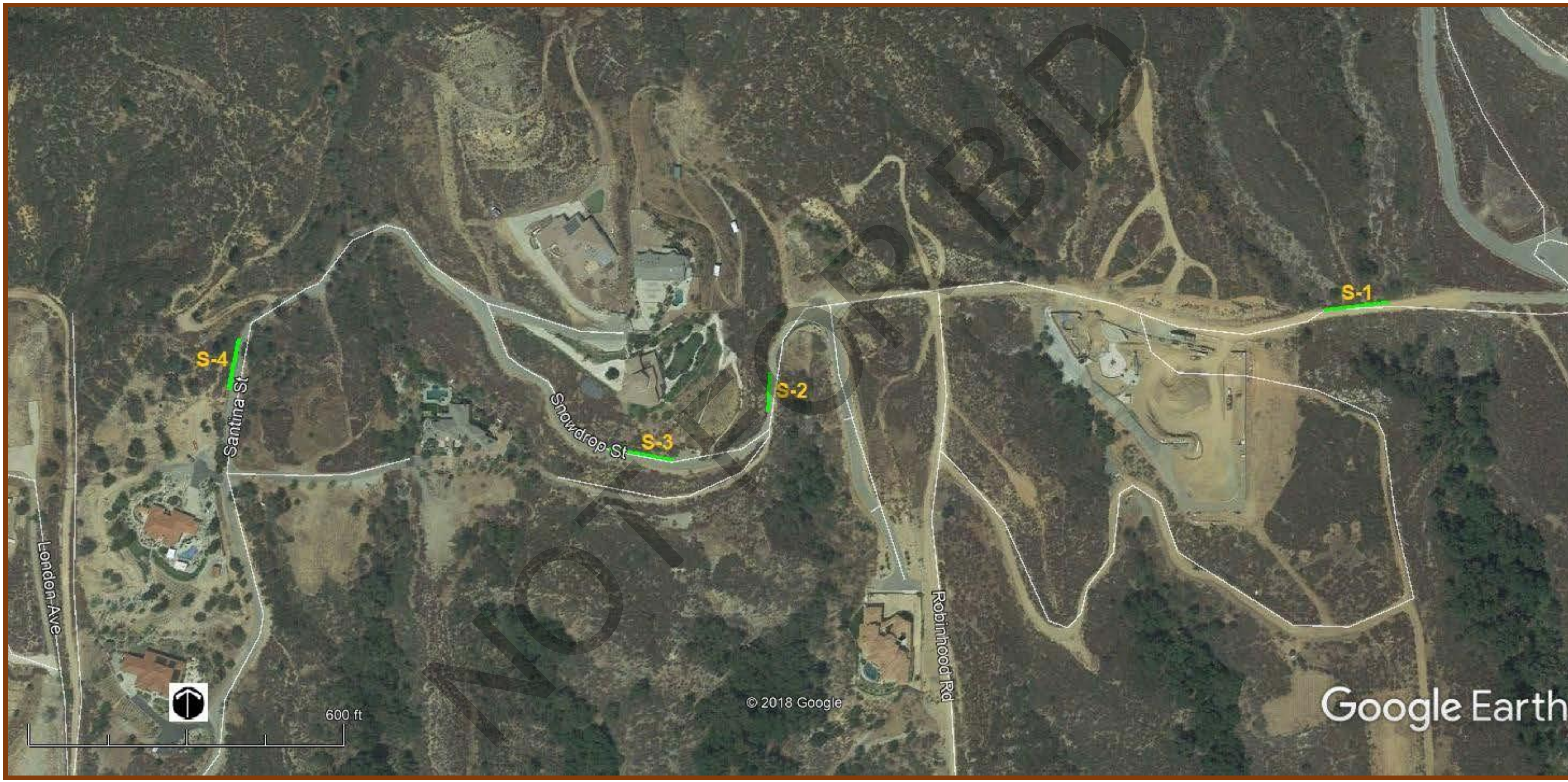
## CLOSURE

The field geophysical survey was performed on October 4, 2019 by the undersigned using "state of the art" geophysical equipment and techniques along the selected traverse location. The seismic data was further evaluated using recently developed computerized tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface velocity and structural conditions. It should be noted that our data presented within this report was obtained along five specific locations therefore other areas in the local may contain different velocity layers and depths not encountered during our field survey. Additional survey traverses may be necessary to further evaluate the excavation characteristics across other portions of the site where cut grading will be proposed, if warranted. Estimates of layer velocity boundaries as presented in this report are generally considered to be within 10± percent of the total depth of the contact.

It is important to understand that the fundamental limitation for seismic refraction surveys is known as nonuniqueness, wherein a specific seismic refraction data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained, and in the interpretation, and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied.

In summary, the results of this seismic refraction survey are to be considered as an aid to assessing the rippability and excavation potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative "test" excavations with the proposed type of excavation equipment for the proposed construction should be considered, so that they may be correlated with the data presented within this report.

# SEISMIC LINE LOCATION MAP



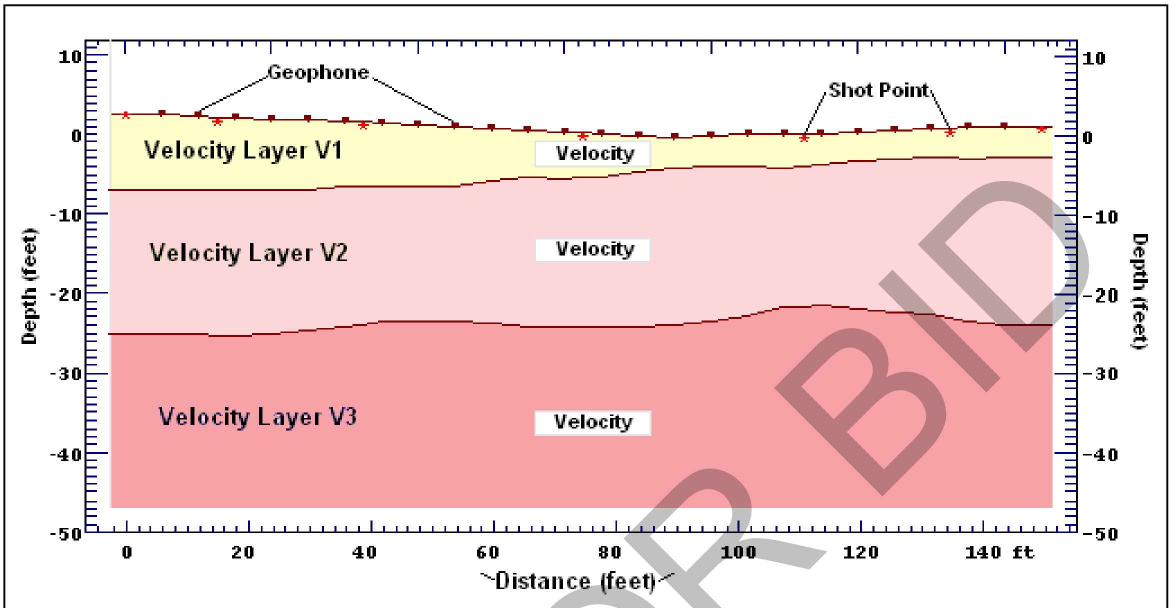

Base Map: Google™ Earth imagery (2019); Seismic traverses shown as green lines.

# APPENDIX A

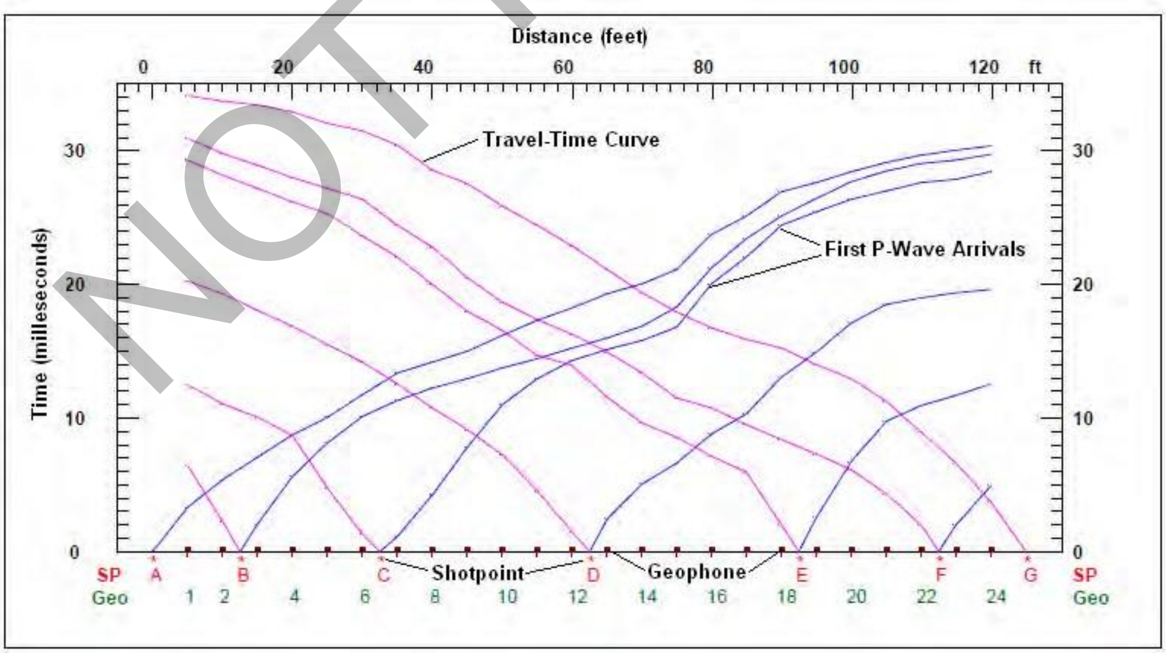
## LAYER VELOCITY MODELS

# LAYER VELOCITY MODEL LEGEND

## LAYER VELOCITY MODEL



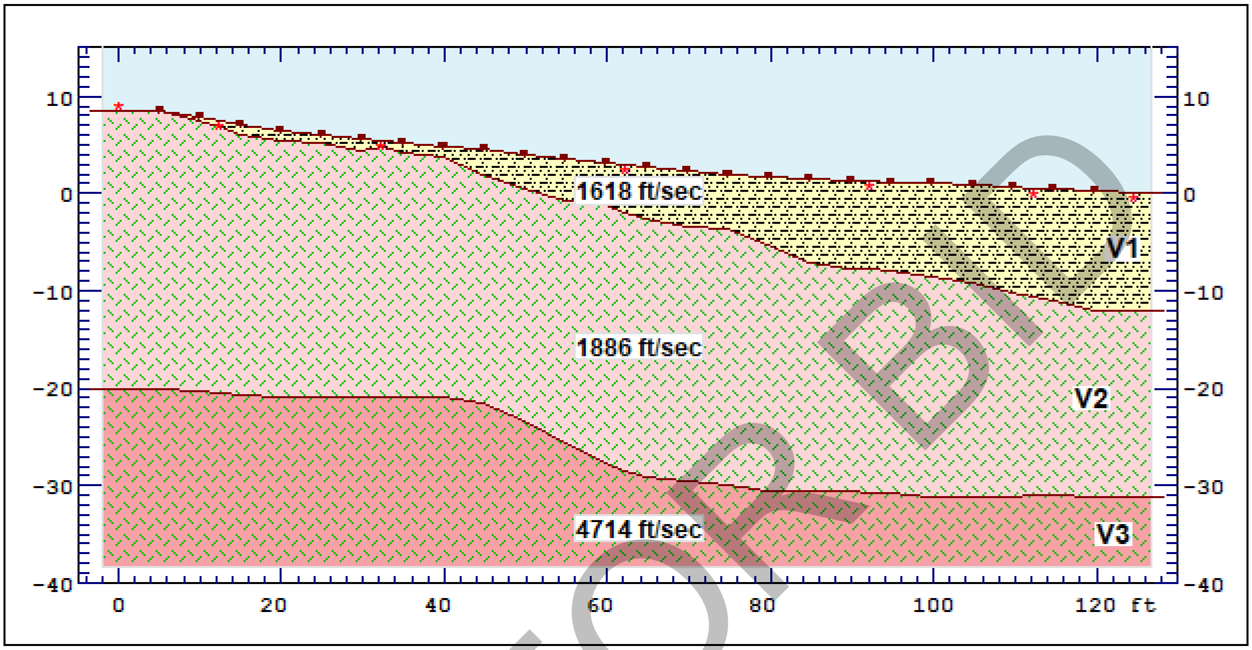

## TIME-DISTANCE PLOT



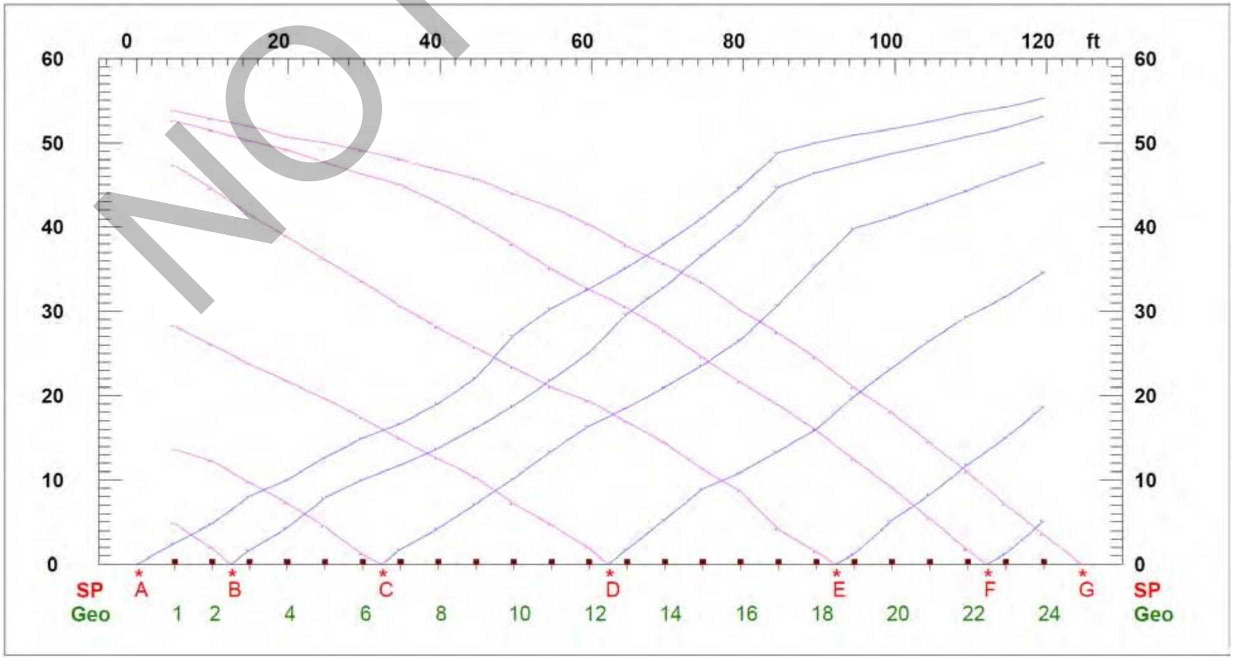
# SEISMIC LINE S-1

## North 83° East >

### LAYER VELOCITY MODEL



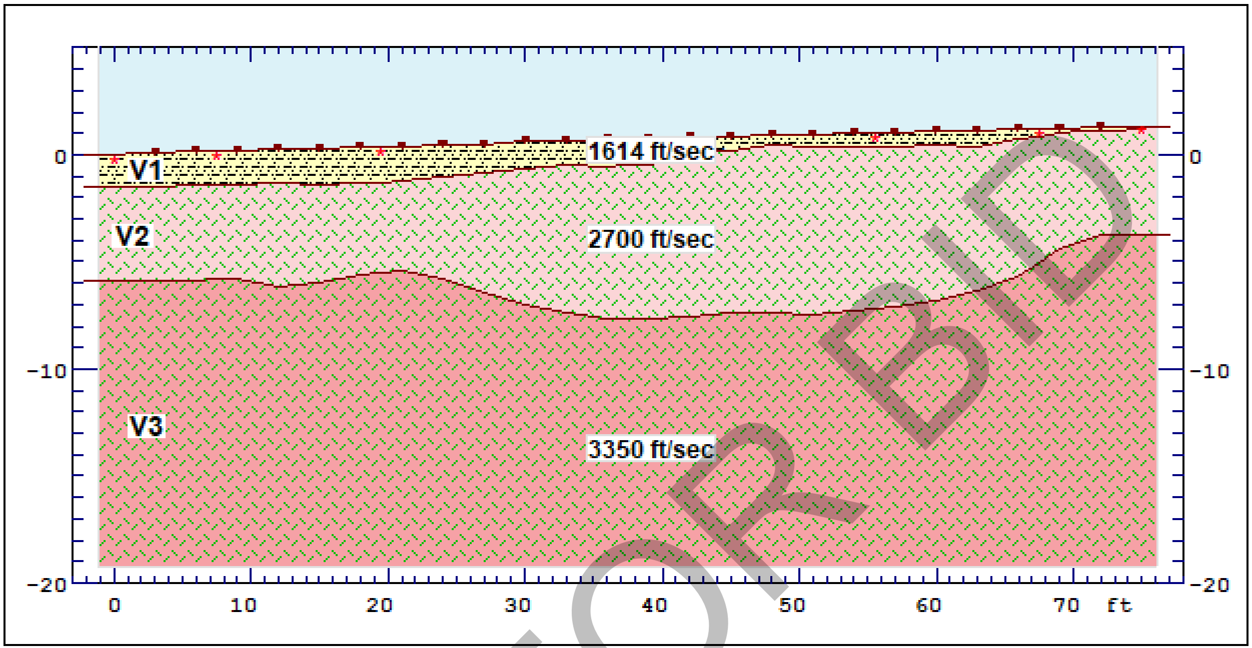

### TIME-DISTANCE PLOT



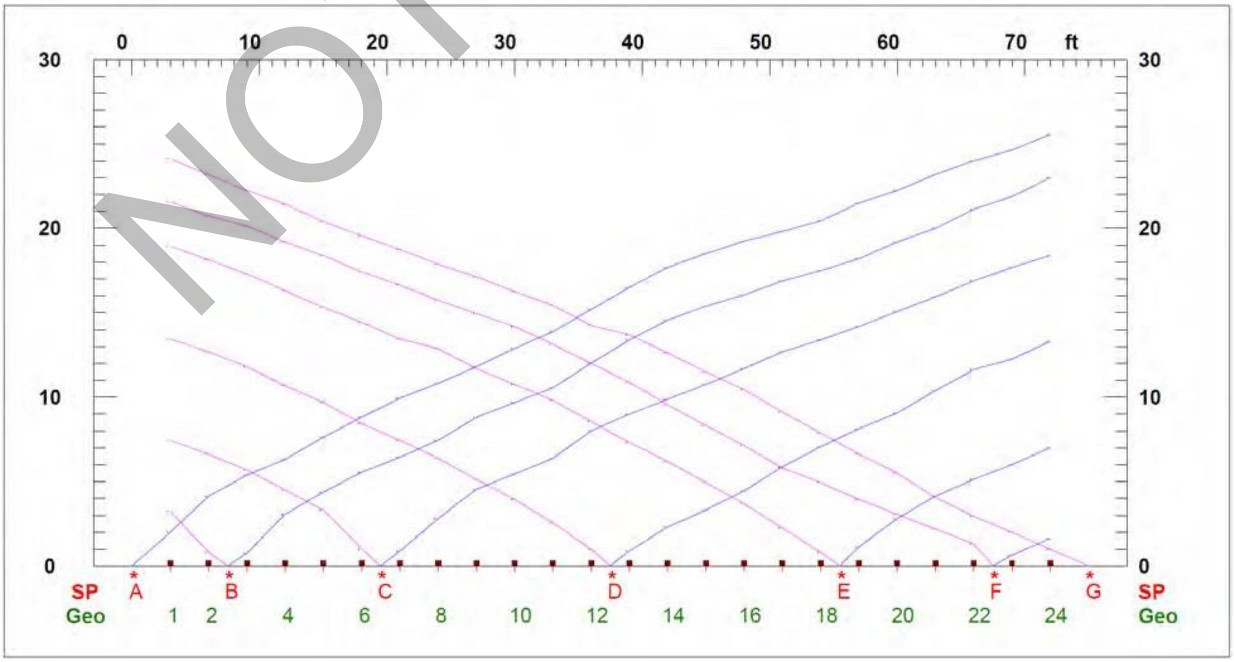
# SEISMIC LINE S-2

## North 3° East >

### LAYER VELOCITY MODEL



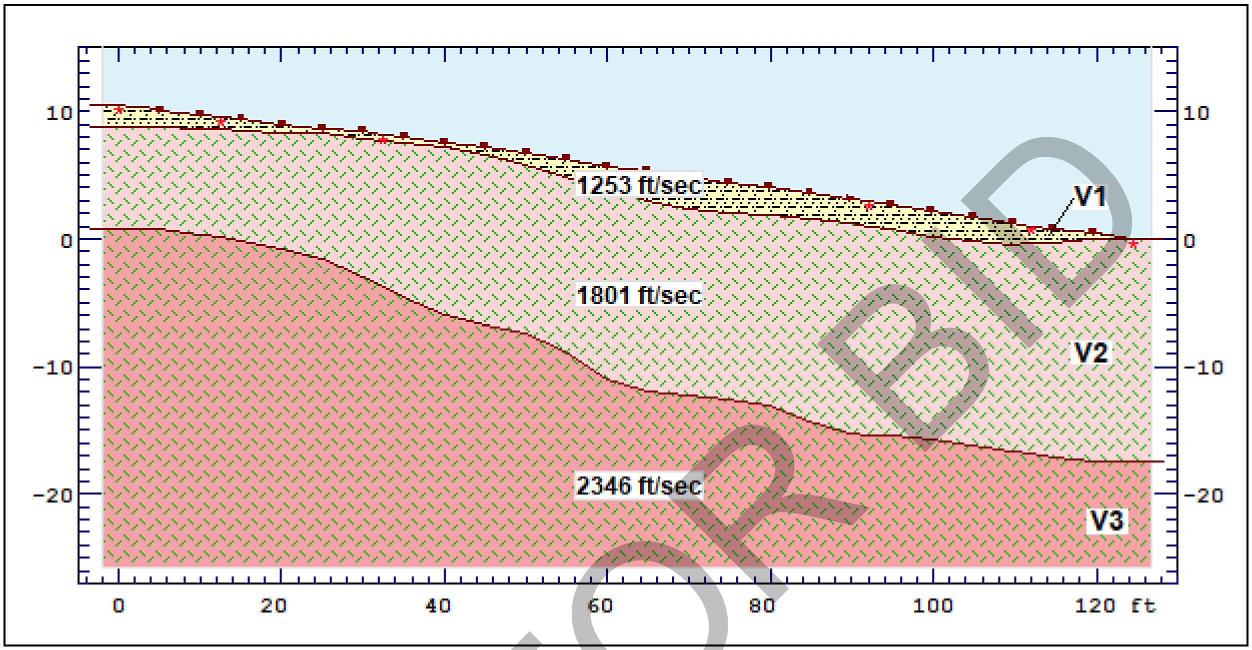

### TIME-DISTANCE PLOT



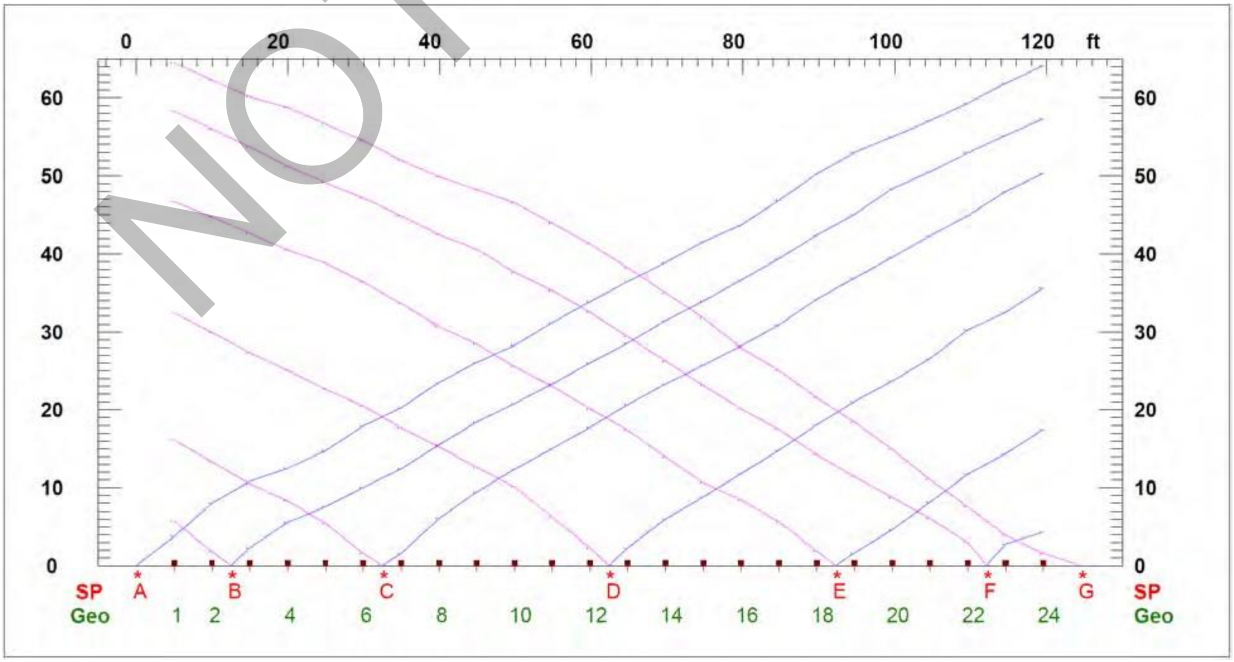
# SEISMIC LINE S-3

## South 80° East >

### LAYER VELOCITY MODEL



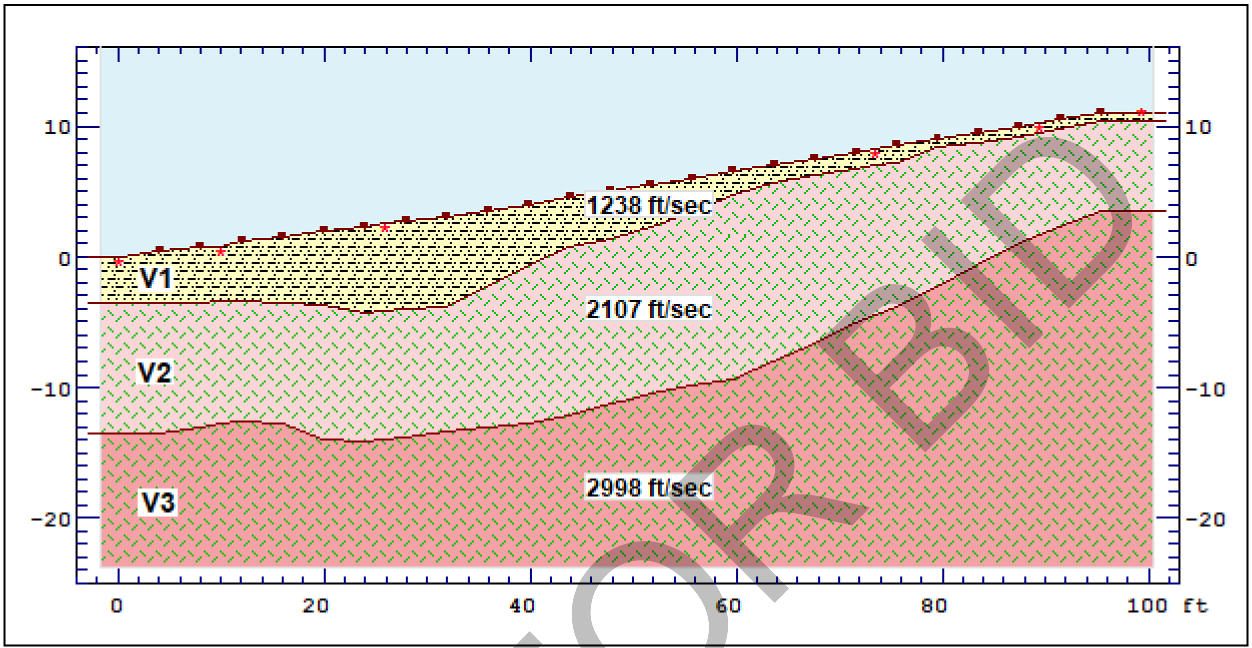

### TIME-DISTANCE PLOT



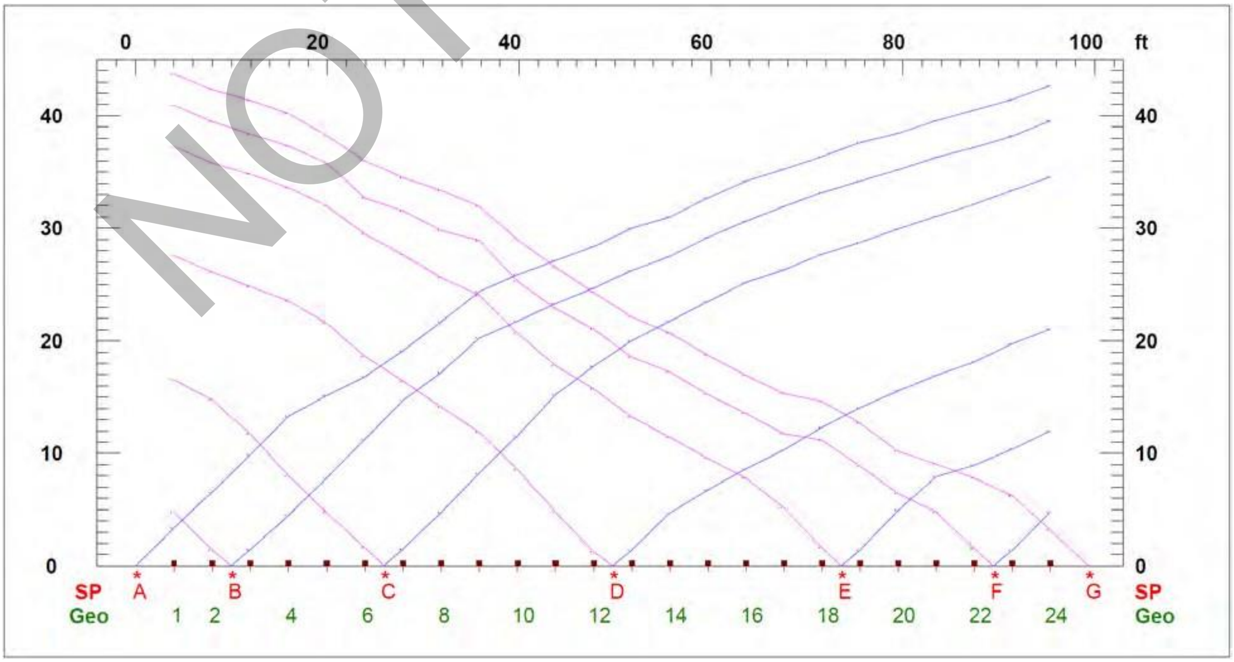
# SEISMIC LINE S-4

## North 11° East >

### LAYER VELOCITY MODEL




### TIME-DISTANCE PLOT



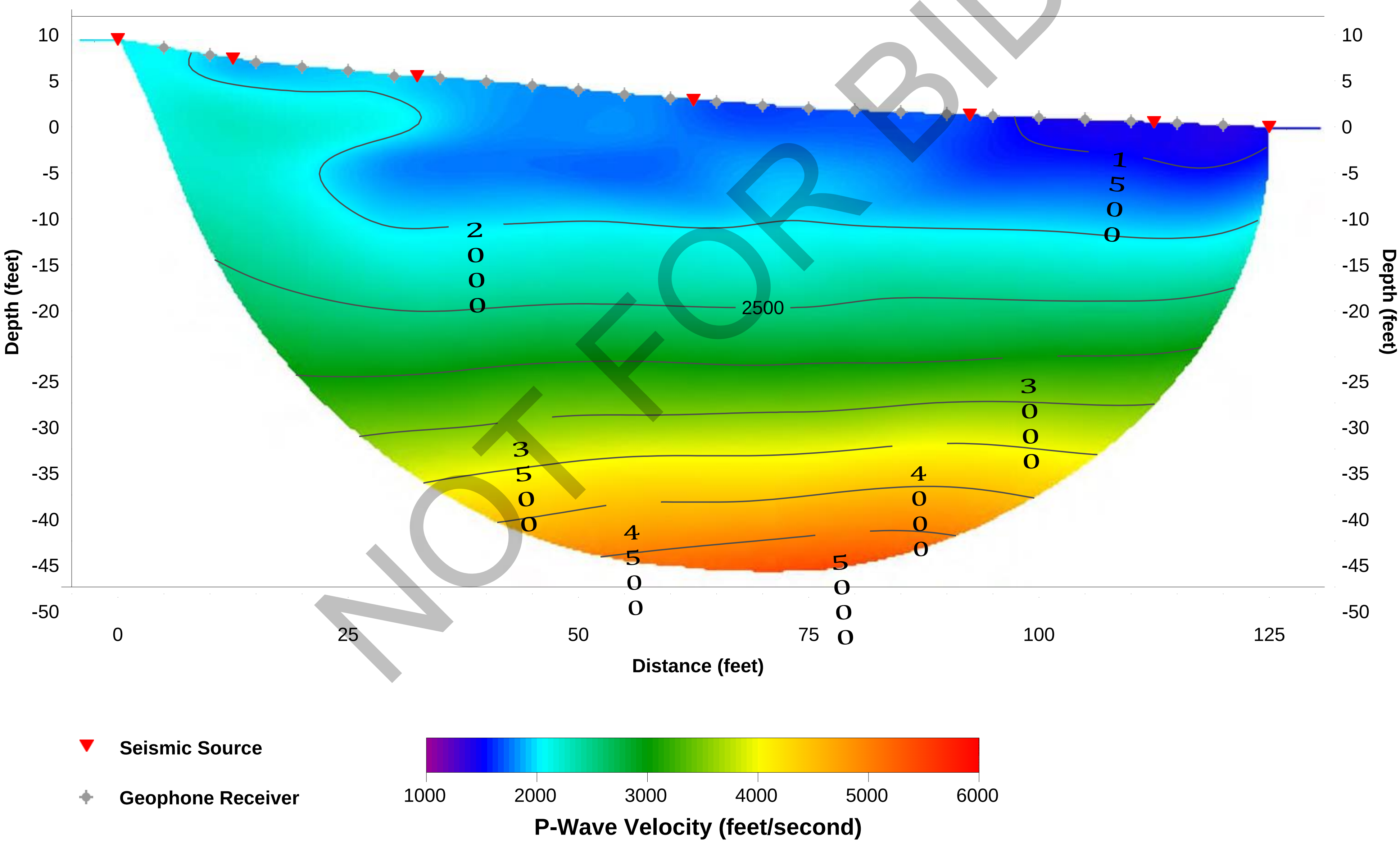
# APPENDIX B

## REFRACTION TOMOGRAPHIC MODELS

# SEISMIC LINE S-1

## North 83° East →

### REFRACTION TOMOGRAPHIC MODEL



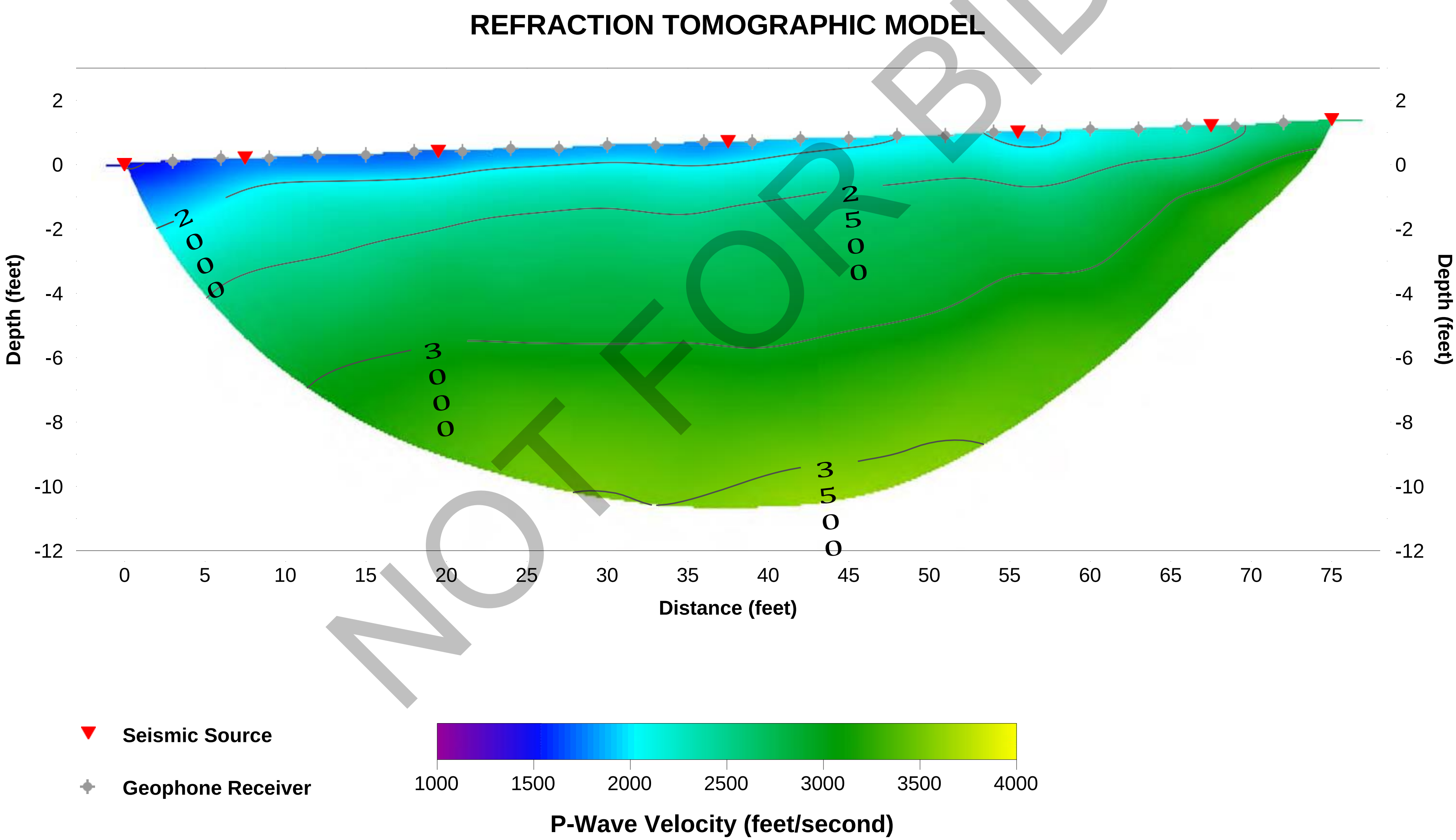
SCALE: 1:1 (Horizontal = Vertical)

# SEISMIC LINE S-1

RMS error 4.1%; Rayfract Version 3.36

# SEISMIC LINE S-2

## North 3° East →



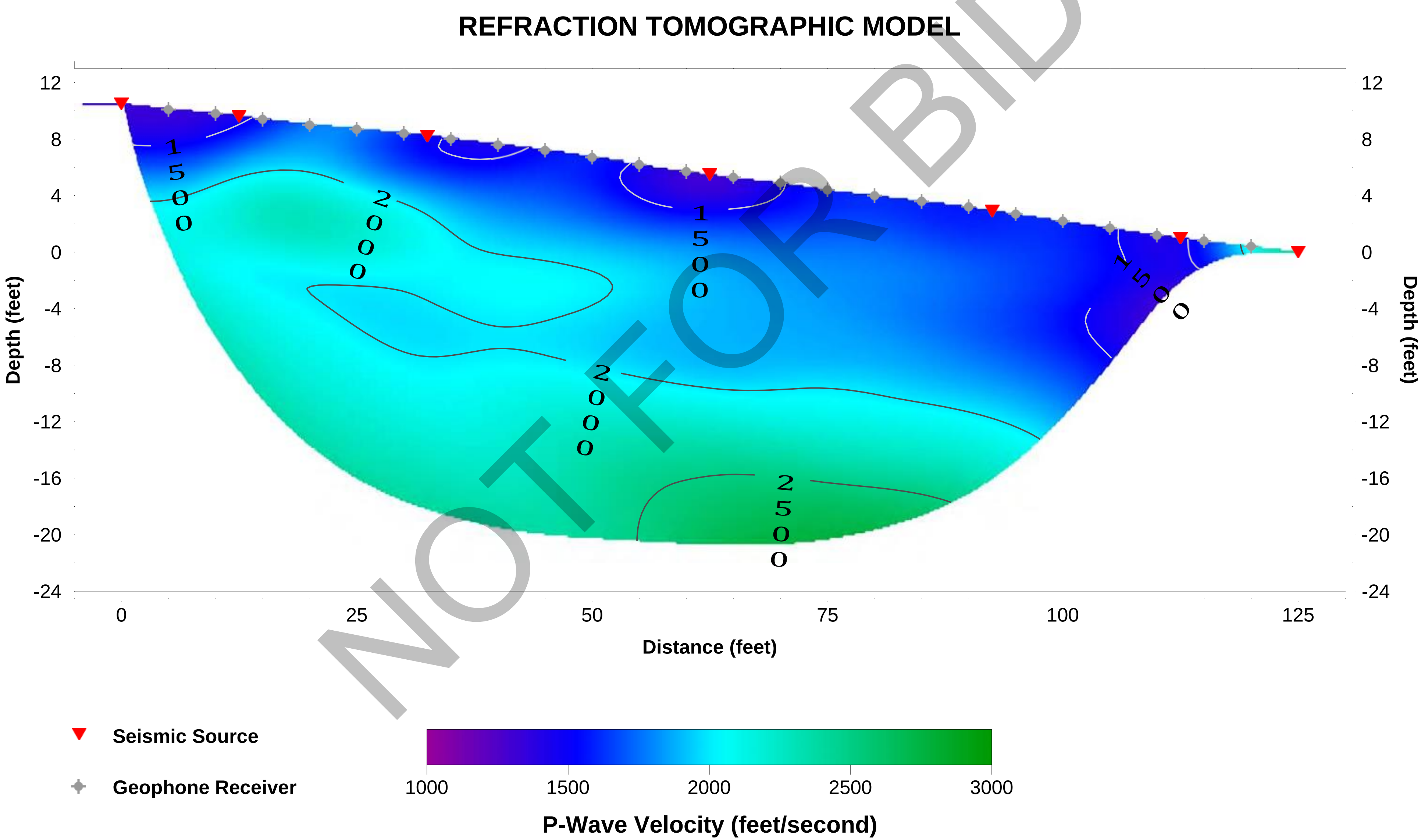
SCALE: Vertical Exaggeration 2X

# SEISMIC LINE S-2

RMS error 3.4%; Rayfract Version 3.36

# SEISMIC LINE S-3

## South 80° East →



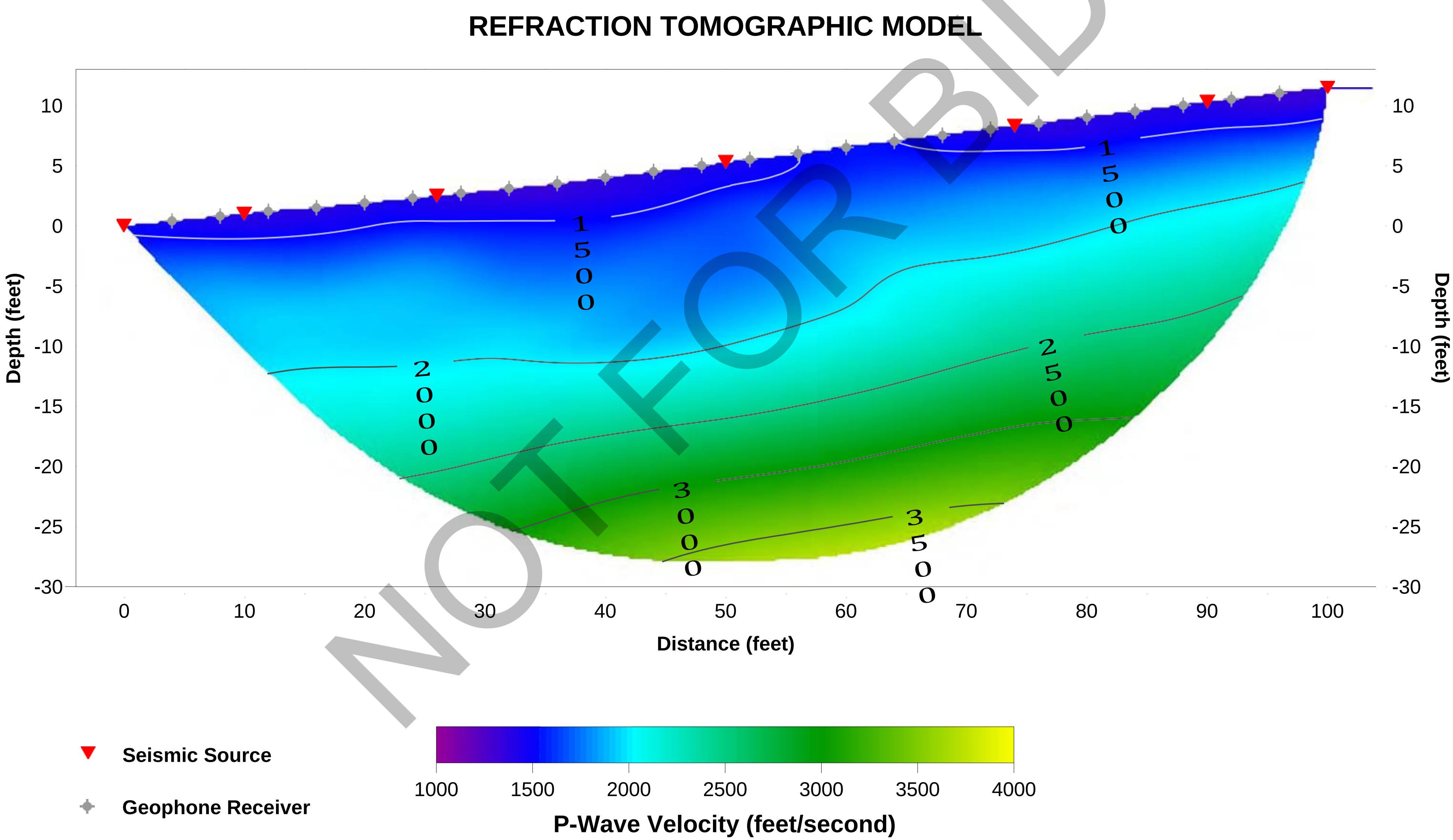
SCALE: Vertical Exaggeration 1.5X

# SEISMIC LINE S-3

RMS error 2.3%; Rayfract Version 3.36

# SEISMIC LINE S-4

## North 11° East →



SCALE: 1:1 (Horizontal = Vertical)

# SEISMIC LINE S-4

RMS error 4.7%; Rayfract Version 3.36

# APPENDIX C

## EXCAVATION CONSIDERATIONS

# EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the client's responsibility to ensure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ***Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,***
- ***Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,***
- ***Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.***

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also, important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to ensure that the proper selection of equipment and ripping techniques are used for the proposed grading.

# APPENDIX D

## REFERENCES

# REFERENCES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