Attachment F –Second Update Report of Geotechnical Investigation

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Report
Second Update Geotechnical Investigation

Proposed Kensho Housing Residential Apartments
Lado De Loma Drive, Vista, California

Submitted to:
Tideline Partners
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TIDELINE PARTNERS

NOVA Project No. 2021172
April 25, 2022

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Tideline Partners
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April 25, 2022
NOVA Project 2021172

Subject: Report
Second Update Geotechnical Investigation
Proposed Kensho Housing Residential Apartments
Lado De Loma Drive, Vista, California

Dear Mr. Gershman:

The work reported herein was completed by NOVA Services, Inc. (NOVA) for Tideline Partners in accordance with the work described by NOVA proposals dated August 2, 2021, January 7, 2022, and March 24, 2022.

Revised site development concepts were provided to NOVA in March 2022. This report provides the findings of additional site characterization and reporting to address development concepts that are current as of this date.

NOVA appreciates the opportunity to be of continued service to Tideline Partners. Should you have any questions regarding this report or other matters, please do not hesitate to call us at 858.292.7575 x 209.

Sincerely,
NOVA Services, Inc.

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Report
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Kensho Housing Residential Apartments
Lado De Loma Drive, Vista, California

Table of Contents

Plates

List of Appendices

List of Tables

List of Figures

List of Figures (continued)

1. INTRODUCTION

1.1 Terms of Reference

This report presents the findings of an update geotechnical investigation for a proposed residential apartment building in the City of Vista, California. The proposed apartment building will be located on an approximately 4.6-acre, irregularly-shaped lot corresponding to APNs 179-024-09, 179-093-05, 179-093-18, 179-093-23, 179-093-30, 179-093-32, and 179-093-34 (hereinafter, 'the site'). Figure 1-1 provides a graphic that depicts the site vicinity.

Figure 1-1. Vicinity Map

The work reported herein was completed by NOVA Services, Inc. (NOVA) for Tideline Partners in accordance with the scope of work detailed in NOVA's proposal dated August 2, 2021, and January 7, 2022.

1.2 Previous Geotechnical Reporting

1.2.1 Work by Others

The site was previously investigated by others in two events, as described below.

1. VME 2003. This work provides the findings of a 2003 geotechnical investigation (reference, *Report of Geotechnical Investigation,* Vinje & Middleton Engineering, 2003).

2. GSI 2016. Geosoils, Inc. utilized the findings of VME 2003, as well as its own subsurface exploration, to provide a 2016 geotechnical assessment of the site (reference, *Geotechnical Evaluation, Proposed Kensho Housing Development, APN's 179-093-18, 23, 30, 32, and 34, Lado De Loma Drive, Vista, CA 92083,* Geosoils, Inc., WO 7089-A-SC, July 28, 2016, hereinafter 'GSI 2016').

Records of test pits and related laboratory testing reported in GSI 2016 have been reviewed by NOVA for this report. The recommendations provided herein supersede those of GSI 2016. NOVA did not have access to VME 2003. However, records of the subsurface exploration reported by VME 2003 are included with GSI 2016 and were reviewed by NOVA.

1.2.2 Previous NOVA Reporting

NOVA provided an updated report of its geotechnical investigation of this site in a September 22, 2021 report (reference, Report, Update Geotechnical Investigation, Proposed Kensho Housing Residential Apartments, Lado De Loma Drive, Vista, California, NOVA Project 2018216, September 22, 2021, hereinafter, 'NOVA 2021').

Design concepts were updated since the submittal of NOVA 2021 to incorporate two additional parcels that have been acquired for this development. This report addresses the current design, expanding the subsurface investigation to address the additional parcels. The recommendations of this report update and supersede those provided in NOVA 2021.

1.3 Objectives, Scope, and Limitations of This Work

1.3.1 Objectives

The objectives of the work reported herein are twofold, as described below.

- Objective 1, Geotechnical. Develop subsurface information conditions to supplement the findings of NOVA 2021, thereafter providing updated recommendations for geotechnical-related development, including earthwork, foundations, retaining walls, and pavements.

- Objective 2, Stormwater. Complete percolation testing in accordance with the requirements of the City of Vista sufficient to identify requirements for development of permanent stormwater infiltration Best Management Practices ('stormwater BMPs').

1.3.2 Scope

In order to accomplish the above objectives, NOVA undertook the task-based scope of work described below.

1. Task 1, Background Review. Reviewed previous reporting and other readily available background data for the site. This documentation included geotechnical reports, topographic maps, geologic data, and development plans. Civil and architectural information was reviewed.

2. Task 2, Subsurface Exploration. A subsurface exploration supplementing that was completed for NOVA 2021 included the subtasks listed below.

   - *Subtask 2-1, Reconnaissance*. A site reconnaissance was undertaken prior to exploratory work. This work included layout of test pits and percolation test borings. Underground Service Alert and a utility location contractor were notified for underground utility mark-out services.

   - *Subtask 2-2, Coordination.* A specialty subcontractor was retained to excavate test pits and percolation test borings. NOVA coordinated with you regarding access for fieldwork.

   - *Subtask 2-3, Test Pits*. Eight (8) test pits were excavated with a backhoe to determine the depth and extent of soils overlying formational tonalite (granitic) rock. These were in addition to our previous 17 test pits.

   - *Subtask 2-4, Percolation Testing*. Conceptual locations for permanent stormwater infiltration Best Management Practices (stormwater BMPs) have not been selected. Feasibility level infiltration data was supplemented by testing an additional four (4) percolation test wells per City of Vista requirements to supplement the previous six (6) tests.

   - *Subtask 2-5, Well and Test Pit Closure*. Following sampling and logging, the test pits and percolation test wells were backfilled and lightly compacted.

   - *Subtask 2*-6, *Seismic Refraction Survey*. In order to determine relative strength and excavation characteristics of the bedrock, a seismic refraction survey comprised of seven (7) seismic refraction lines was completed by a licensed geophysicist.

3. Task 3, Laboratory Testing. Index soil laboratory testing was used to support soil classification and estimates of soil behavior. Chemical testing addresses the potential that soils may be corrosive to embedded concrete or metals.

4. Task 4, Engineering Evaluations. The findings of Tasks 1 through 3 were directed toward assessment of requirements for geotechnical-related development, including earthwork, foundations, retaining walls, and pavements.

5. Task 5, Reporting. Submittal of this report concludes NOVA's scope of services. The report provides a record of all work and geotechnical-related recommendations for earthwork, foundations, retaining walls, and pavements.

1.3.3 Limitations

The construction recommendations included in this report are not final. Geotechnical and geologic studies are characterized by uncertainty. These recommendations are developed by NOVA using judgment and opinion, based upon the subsurface exploration. NOVA can finalize its recommendations only by observing actual subsurface conditions revealed during construction. NOVA cannot assume responsibility or liability for the report's recommendations if NOVA does not perform construction observation.

This report does not address any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, soil gas, groundwater, or surface water within or beyond the site.

Appendix A to this report provides important additional guidance regarding the use and limitations of this report. This information should be reviewed by all users of the report

1.3.4 Understood Use

NOVA expects that the findings and recommendations provided herein will be utilized by Tideline Partners and its design team in decision-making regarding foundation-related design and construction.

1.4 Organization of this Report

The remainder of this report is organized as abstracted below.

- Section 2 reviews available project information.
- Section 3 describes the field investigation and laboratory testing.
- Section 4 describes the surface and subsurface conditions.
- Section 5 reviews geologic, soil, and siting-related hazards common to this area of California, considering each for its potential to affect the planned development.
- Section 6 provides recommendations for earthwork and foundation design.
- Section 7 provides recommendations for pavements.
- Section 8 addresses design for stormwater infiltration.
- Section 9 provides recommendations for geotechnical observation during construction
- Section 10 lists the principal references utilized in the development of the report.

Figures and tables are embedded in the text of the report at the point which they are referenced. Plate 1, provided immediately following the text of this report, shows the location of fieldwork in larger scale. Plate 2 provides geologic cross-sections. The report is supported by five appendices, as described below.

- Appendix A presents guidance regarding the use and limitations of this report.
- Appendix B presents logs of NOVA’s test pits.
- Appendix C provides records of laboratory testing.
- Appendix D provides records of the geophysical survey.
- Appendix E provides Worksheet C.4-1: Categorization of Infiltration Feasibility.

2. PROJECT INFORMATION

2.1 Site Location

The proposed Kensho Housing site (hereinafter, also referenced as 'the site') is an approximately 4.6-acre irregularly-shaped lot corresponding to the following seven APNs: 179-024-09 (212 Guajome Street), 179-093-05 (446 Lado De Loma), 179-093-18, -23, -32, and -34 (420 Lado De Loma Drive) in the City of Vista, CA. The site is bounded on the northwest by Guajome Street, the Sprinter rail line to the northeast, Lado De Loma Drive, and residential properties to the southwest.

The location and limits of the property are depicted on Figure 2-1.

Figure 2-1. Site Location and Limits

2.2 Current and Historic Site Use

2.2.1 Current

As may be seen by review of Figure 2-1, the site is largely vacant, with the ground surface vegetated by grasses and weeds, and several trees. Single-family residential structures occupy the northern and southern parcels of the site.

The site consists of sloping terrain. Site elevations range from about +347 feet mean sea level (msl) in the north to about +430 feet msl in the south near Lado De Loma Drive.

2.2.2 Historic

Review of historic aerial photos dating to 1938 indicates that the site has not been developed since that time. The area of the site was largely used for agriculture (orchards) until about 1960 when residential development first emerged on properties surrounding the site. Figure 2-2 reproduces a 1938 aerial view of the site area.

Figure 2-2. Aerial View, 1938
(source: USDA (1938-05-11 - 1938-08-07))

2.3 Planned Development

2.3.1 Design Basis

NOVA's understanding of the planned development is based upon review of conceptual development plans contained in *Kensho Housing, EDR Submittal* by Safdie Rabines Architects, March 11, 2022 (hereinafter, 'SRA 2022').

2.3.2 Proposed Apartments

SRA 2022 depicts planning for a four-level residential apartment building with two levels of below- and above-grade parking. Additionally, a three-level apartment building comprised of micro units is proposed on the northern parcel (179-024-09-00, 212 Guajome Street). Townhomes are still being considered for the site on APN 179-093-05-00 (446 Lado De Loma).

Figures 2-3 below and 2-4 on the following page provide a site elevation and a 3-D view, respectively, of the planned development.

Figure 2-3. Site Section of Planned Development
(source: SRA 2022)

Figure 2-4. Planned Development
(source: SRA 2022)

2.3.3 Structural

Structural design has not been initiated for the planned apartment building. As is evident by review of Figure 2-4, current planning envisions the planned apartment building will include four levels of residences above two levels of parking.

As employed at this site, the two levels of the parking garage would be constructed of reinforced concrete. The residential units above the parking garage would be wood framed. The lowest level of the parking garage would include a ground-supported concrete slab.

Designed as described above, the structure might rise to about 55 feet above surrounding ground. Column loads to foundations (DL +LL) will vary with column spacing and other factors. Based upon experience with analogous structures, NOVA expects that maximum interior column loads may range to about 600 kips. Exterior and corner column loads may be in the range of 150 to 400 kips. Interior shear walls developed around elevator shafts may be used to collect seismic and wind loads.

2.3.4 Planned Site Grading

Plate 2 – Geologic Cross Sections provides four cross-sections through the planned apartment building. These sections were developed with approximate elevations of finished floor of the garage level provided by the project civil engineer. This design has not been finalized to this point, such that the garage floor elevations should be understood to be for the purposes of illustration only.

As seen by review of the cross-sections, a significant amount of grading will be required to complete the project, including cuts up to 28 feet in depth to reach the garage floor elevation.

Fills up to 6 feet are required to reach finish floor in the building, and up to 10 feet of fill will be required in the paved areas east of the building. These fills will be deeper once remedial grading of the loose fill and colluvial soils are performed.

The building pads will have ‘transition’ conditions wherein the west side of the building will be set in deep bedrock cuts, while the east side will be set in shallow cuts to shallow fill. Because of the existing sloping topography at the site, large retaining walls up to 28 feet in height will be constructed along the west side of the building to accommodate the lowest building levels.

3. SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Overview

A subsurface exploration was completed under the direction of a NOVA geologist in three events during August 2021 and April 2022, as described below.

1. Geophysical. Seven (7) geophysical seismic lines were completed on August 7, 2021 and April 4, 2022.
2. Test Pits. Twenty-five (25) test pits were excavated by a backhoe. Seventeen (17) test pits were completed on August 10, 2021, and eight (8) additional test pits were completed on January 12, 2022.
3. Percolation Testing. Percolation testing was completed at six (6) locations on August 11, 2021, and an additional four (4) locations on January 13, 2022.

In addition to the above, GSI 2016 reports the findings of seven (7) test pits (TP-1 through TP-7) completed for a June 2016 geotechnical investigation. Figure 3-1 presents a plan view of the site indicating the location of the subsurface explorations described above. Plate 1, provided immediately following the text of this report, shows the location of this work in larger scale.

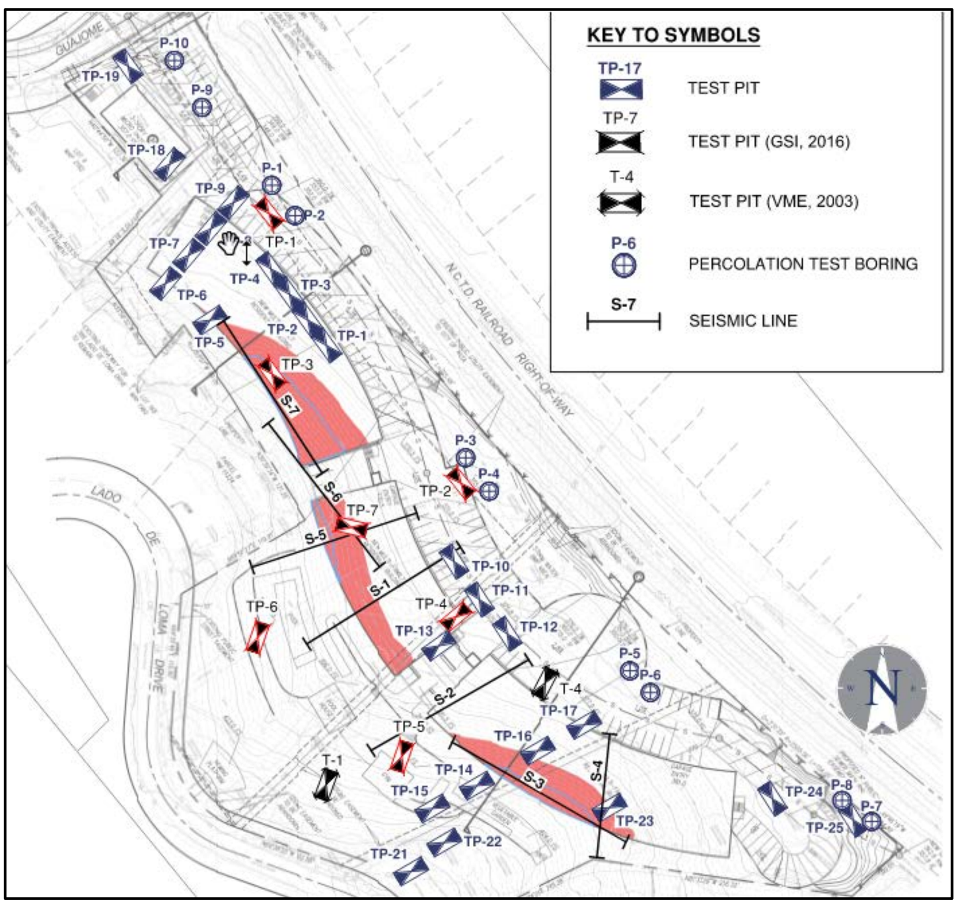


Figure 3-1. Locations of Subsurface Explorations

The remainder of this section describes the work by NOVA, the test pits reported in GSI 2016, and laboratory testing by NOVA and GSI 2016.

3.2 Test Pits

3.2.1 Test Pits by NOVA

A NOVA geologist directed excavation and sampling of 25 test pits (TP-1 through TP-25) to depths between 2 feet and 8 feet below ground surface (bgs) on August 10, 2021, and on January 12, 2022. Prior to beginning fieldwork, excavation locations were determined by a geologist based on the proposed building configuration at the site. The test pits were excavated using a track-mounted mini-excavator. Table 3-1 provides a summary of the test pits.

Table 3-1. Summary of the Test Pits by NOVA

Test Pit Reference	Approx. Ground Surface Elev. (feet, msl)	Total Depth Below Ground Surface (feet)	Thickness of Fill (feet)	Thickness Colluvium (feet)	Depth to Tonalite (feet)
TP-1	+367	4.5	1.5	2.5	4
TP-2	+366	5.5	2	3	5
TP-3	+365	5	2	2.5	4.5
TP-4	+363	5	2	2.5	4.5
TP-5	+377	3	0 to 1	0 to 1	1
TP-6	+374	2	0	1	1
TP-7	+367	2.5	1	1	2
TP-8	+363	4.5	1.5	2.5	4
TP-9	+358	4	1.5	2	3.5
TP-10	+373	6	0	5	5
TP-11	+373	6	1	4	5
TP-12	+375	8	1	6.5	7.5
TP-13	+380	6-7	2 to 3	2 to 3	6
TP-14	+405	2	0	1	1
TP-15	+411	3	0	2	2
TP-16	+393	2	0	2	2
TP-17	+383	4	0	3	3
TP-18	+358	6	2.5	2.5	5
TP-19	+352	6	3	2.5	5.5
TP-20	+407	4	0	1.5	1.5
TP-21	+427	3	0	0	0
TP-22	+420	8	8	0	8
TP-23	+395	4	0	1	1
TP-24	+368	5	0	2	2
TP-25	+363	4	0	1	1

Following excavation and logging, the test pits were backfilled with the excavated soil and lightly compacted. Test pit logs by NOVA are presented in Appendix B. Figure 3-2 depicts test pit excavation operations.

Figure 3-2. Test Pit Excavation (TP-5, August 2021)

3.2.2 Test Pits by Others

<u>GSI 2016</u>

GSI 2016 reports the findings of seven (7) test pits (TP-1 through TP-7) excavated by a backhoe in June 2016 at the locations depicted on Figure 3-1. Table 3-2 (following page) provides a summary of the test pits by GSI.

No groundwater was encountered during excavation of the test pits.

<u>VME 2003</u>

GSI 2016 also provides the records of two (2) test pits ('T-101' and 'T-102') completed by Vinje & Middleton Engineering (VME) in 2003. The report associated with that work is not available to NOVA, such that the locations of these test pits are uncertain and not shown on Figure 3-1.

Table 3-3 summarizes the material encountered in the test pits. No groundwater was encountered during excavation of the test pits by VME.

Table 3-2. Summary of the Test Pits Reported by GSI 2016

Test Pit Reference	Approx. Ground Surface Elev. (feet, msl)	Total Depth Below Ground Surface (feet)	Thickness of Fill (feet)	Thickness Colluvium (feet)	Depth to Weathered Tonalite (feet)
TP-1	+355	8	3	1.5	4.5
TP-2	+363	5	0	2.5	2.5
TP-3	+378	8	0	2	2
TP-4	+376	8	3	3	6
TP-5	+395	5	0	3	3
TP-6	+405	5	0	2.5	2.5
TP-7	+385	4	0	2.5	2.5

Table 3-3. Summary of the Trenches Reported by VME 2003

Trench Reference	Approx. Ground Surface Elev. (feet, msl)	Total Depth Below Ground Surface (feet)	Thickness of Fill (feet)	Thickness Colluvium (feet)	Depth to Weathered Tonalite (feet)
T-101	Not reported	7	3.5	3	6.5
T-102	Not reported	7	7	0	7

3.3 Percolation Testing

3.3.1 General

NOVA directed the excavation and construction of ten (10) percolation test borings, following the recommendations for percolation testing presented in the City of Vista BMP Design Manual. The locations of these borings are shown on Figure 3-1.

3.3.2 Drilling

Borings were drilled with a backhoe-mounted 8-inch solid stem auger to depths of 5 to 6 feet bgs. Field measurements were taken to confirm that the borings were excavated to approximately 8 inches in diameter. The borings were logged by a NOVA geologist, who observed and recorded exposed soil cuttings and the boring conditions. Logs of the exploratory percolation test borings are provided in Appendix B.

3.3.3 Conversion to Percolation Wells

Once the test borings were drilled to the design depth, the borings were converted to percolation wells by placing an approximately 2-inch layer of ¾-inch gravel on the bottom, then extending 3-inch diameter Schedule 40 perforated PVC pipe to the ground surface. The ¾-inch gravel was used to fill the annular space around the perforated pipe to at least 12 inches below existing finish grade to minimize the potential of soil caving.

Figure 3-3 (following page) depicts the percolation test well setup.

3.3.4 Percolation Testing

In percolation boring P-1 through P-4 and P-7 through P-10, the pre-soak water did not percolate over 6 inches into the soil unit within 25 minutes. In percolation borings P-5 and P-6, the pre-soak water did percolate over 6 inches into the soil unit within 25 minutes. Based on the results of the trials, water levels were recorded every 30 minutes for 6 hours in borings P-1 through P-4, and every 10 minutes for 1 hour in boring P-5 and P-6. At the beginning of each test interval, the water level was raised to approximately the same level as the previous tests, in order to maintain a near-constant head during all test periods.

Table 3-4 summarizes the percolation test conditions and infiltration rates calculated using the Porchet method.

Table 3-4. Percolation Testing by NOVA

Boring Reference	Approximate Elevation (feet, msl)	Total Depth (feet)	Approximate Percolation Test Elevation (feet, msl)	Subsurface Unit Tested[1]	Infiltration Rate (in/hr)	Infiltration Rate (in/hr, FS=2)[2]
P-1	+354	5	+349	Qcol	0.06	0.03
P-2	+355	6	+349	Qcol	0.10	0.05
P-3	+364	6	+358	Kt	0.12	0.06
P-4	+365	5	+360	Kt	0.06	0.03
P-5	+369	6	+363	Qcol	2.99	1.49
P-6	+370	5.5	+364.5	Qcol	2.32	1.16
P-7	+362	5	+357	Kt	0.12	0.06
P-8	+362	5	+357	Kt	0.69	0.34
P-9	+371	5	+365.5	Qcol	0.16	0.08
P-10	+371	5	+365.5	Qcol	0.38	0.19

Note 1: The referenced subsurface units tested Colluvium (Qcol), and Cretaceous Tonalite (Kt)
Note 2: 'FS' indicates 'Factor of Safety'.

As may be seen by review of Table 3-4, a factor of safety (FS) is applied to the infiltration rate (I) determined by the percolation testing. This factor of safety, at least FS = 2 in local practice, considers the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time. The calculated infiltration rates after applying FS = 2 range from 0.03 to 1.49 inches per hour.

The infiltration feasibility of the site is discussed in Section 8 of this report and infiltration worksheets are presented in Appendix E.

3.3.5 Closure

At the conclusion of the percolation testing, the upper sections of the PVC pipe were removed and the resulting holes backfilled with soil cuttings to match the existing surface.

Figure 3-3. Percolation Testing, August 2021

3.4 Seismic Refraction Survey

3.4.1 General

As currently designed, proposed cuts to reach the finish floor elevation of the parking structures at the site are up to 28 feet deep. Accordingly, a seismic refraction survey was undertaken to estimate the quality of rock that occurs within this depth limit. As employed in this instance, the objective of the testing was to identify the seismic wave velocities of the subsurface materials to both (i) identify the occurrence of soil and rock, and (ii) characterize the requirements for loosening/excavation of the bedrock. Figure 3-4 (following page) depicts the principles of seismic refraction testing as employed in this instance.

Seven (7) seismic refraction lines, aligned as shown on Figure 3-1 and Plate 1, were performed for a seismic refraction survey on August 7, 2021 and April 4, 2022. The seismic refraction survey results are provided in Appendix D.

Seismic refraction relies on measurements of the travel times of seismic waves traveling through and refracting from subsurface layers with contrasting densities. Seismic waves are introduced to the subsurface by striking a steel plate at the surface with a heavy hammer and measuring seismic wave reception from each geophone. 'Geophones' spaced at regular intervals intercept the seismic waves traveling through and reflecting off of the subsurface media. The data is then processed and interpreted using seismic interpretation software.

The seismic refraction surveys conducted for this project are used to measure the seismic compression velocity ('P-wave') and estimate the rippability of the subsurface stratigraphy. The measurements of seismic velocity can be used to estimate the relative difficulty of excavating rock and rock-like materials.

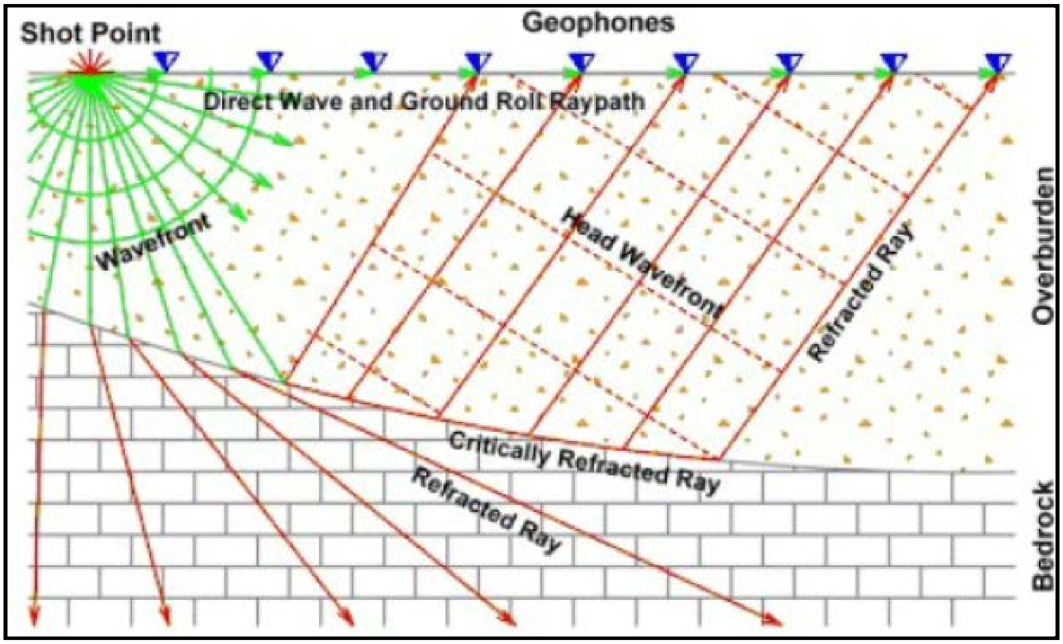


Figure 3-4. Seismic Refraction Geometry

This data provides an estimate of the degree of 'rippability', a semi-quantitative assessment of the potential for mechanical loosening of rock so that the rock may be excavated. Figure 3-5 (following page) provides an example of the tomographic model determined by the refraction testing at line S-2.

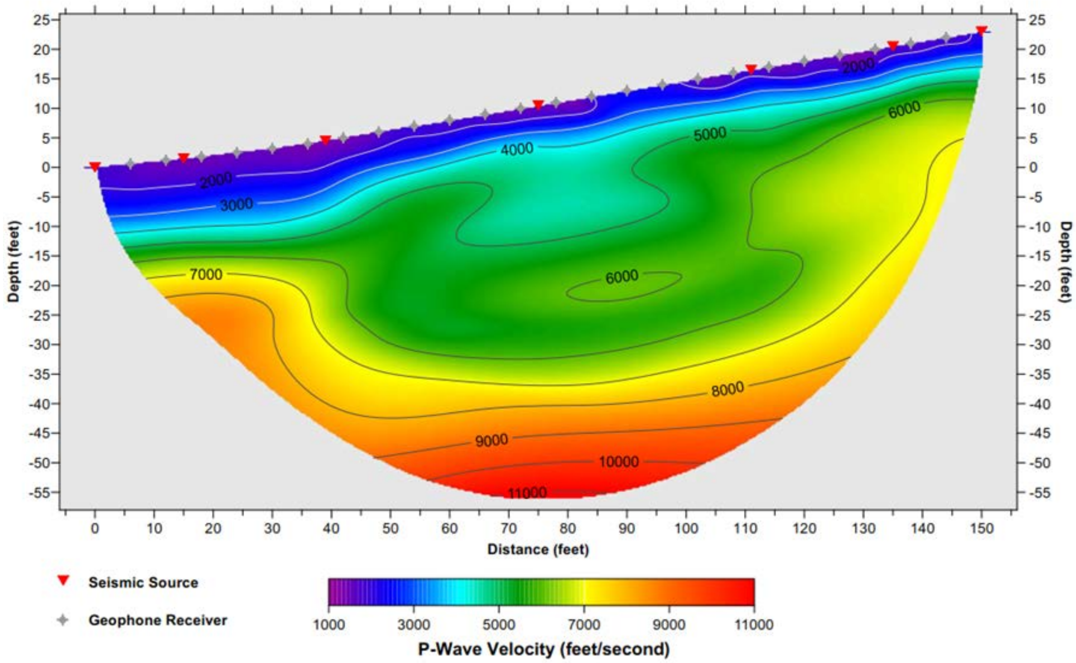


Figure 3-5. Subsurface Profile along Seismic Line S-2

3.4.2 Indications of the Seismic Lines

The seismic traverses indicate the following sequence of subsurface materials. It is important to note that the degree of weathering is highly variable across the length of the lines, as may be seen in Figure 3-5 at the horizontal distance of 10 feet, where the 8,000 feet/second tomographic line is encountered approximately 20 feet higher than below the rest of the profile.

1. <u>Seismic Unit V1, Colluvium and Fill</u>. A near-surface layer of colluvium and fill mantles the site to depths of approximately 5 feet. The average velocity of this unit is about 1,360 feet/second (fps), typical for such soils.

2. <u>Seismic Unit V2, Weathered Tonalite</u>. The near-surface layer of colluvium and fill is underlain by weathered tonalite rock, with seismic velocities of about 3,600 fps to 6,000 fps. These velocities reflect weathered, fractured bedrock. This unit extends to a depth of typically 10 to 12 feet below existing grade.

3. <u>Seismic Unit V3, Less Weathered Tonalite</u>. Below about 10 to 15 feet depth, the site is underlain by relatively sound, less weathered igneous bedrock. The seismic velocity anticipated for the deep removals at the site are between 6,000 to 7,000 feet/second, bordering on marginally rippable.

It is important to understand that seismic velocity is influenced by many factors, such as the presence of moisture and rock fractures. Similarly, the rippability of a rock mass is controlled by numerous parameters including strength, weathering and the spacing of discontinuities/fractures.

Measured seismic velocity models may be compared to empirically derived rippability charts such as those published by Caterpillar Inc. Figure 3-6 (following page) is taken from the Caterpillar Performance Handbook. This table provides a rough correlation of rippability for a D10 bulldozer as a function of the seismic velocity of the subsurface material.

Typically, weathered rocks, highly stratified rocks, and rocks with extensive fracturing are rippable. Conversely, massive or crystalline rocks (such as the tonalite that underlies this site) and rocks with few planes of weakness are typically less rippable. The tonalite that underlies the site is a granitic-type igneous rock. Therefore, the "granite" correlation in the chart of Figure 3-6 can be used to estimate the rippability of the rock beneath the site.

This chart indicates that up to a velocity of 7,100 feet per second (fps) the material is rippable, between approximately 7,100 and 8,500 fps the rock is marginally rippable, requiring special breaking techniques, and more than 8,500 fps, the rock will likely require blasting.

Despite the correlations presented in this chart that addresses rippability, it is important to understand that these correlations are empirical. Actual rippability may vary from that forecast by the chart.

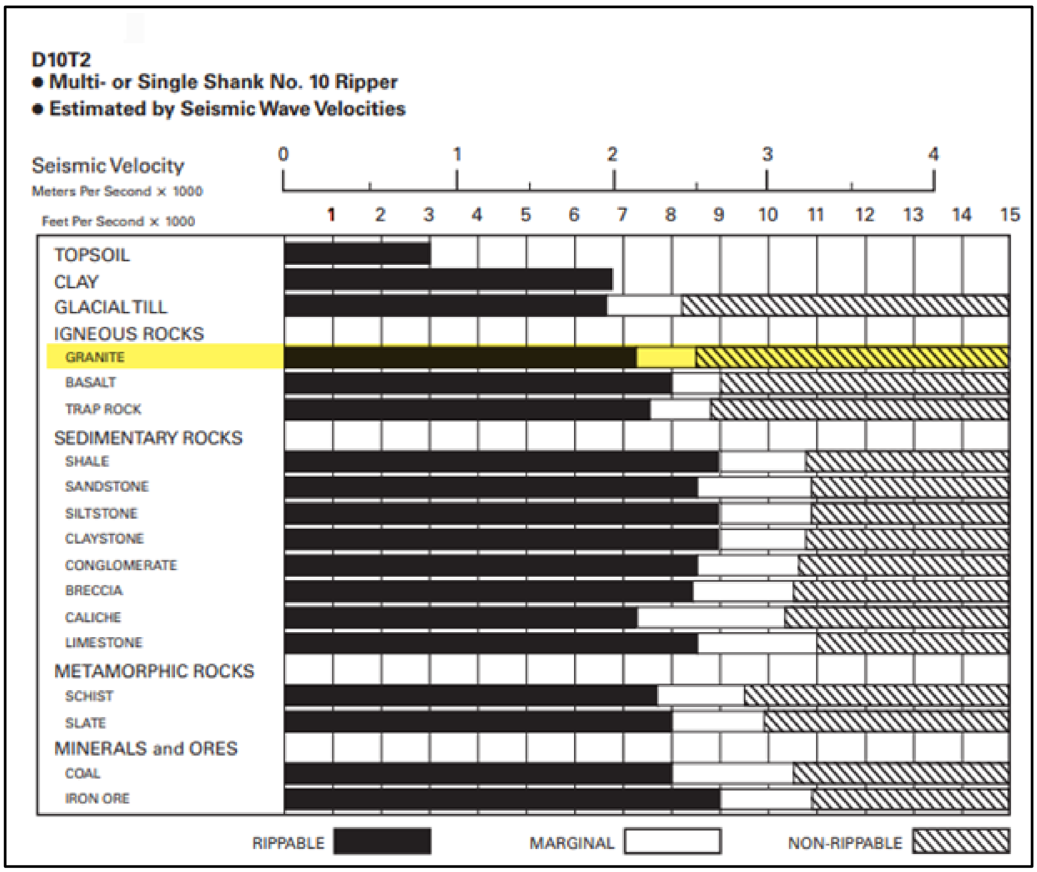


Figure 3-6. D10 Seismic Velocity Rippability Chart
(source: Caterpillar Performance Handbook, 2019)

3.5 Laboratory Testing

3.5.1 General

Soil samples recovered from the engineering borings were transferred to NOVA's geotechnical laboratory where a geotechnical engineer reviewed the soil samples and the field logs. Representative soil samples were selected and tested in NOVA's materials laboratory to check visual classifications and to determine pertinent engineering properties.

The laboratory program included visual classifications of all soil samples, as supported by index testing in general accordance with ASTM standards. The visual classifications were further evaluated by performing grain size, plasticity, and expansivity tests. The index testing may be also used to estimate a variety of soil characteristics and physical properties. The remainder of this section provides the results of the laboratory by NOVA and the laboratory testing reported in GSI 2016. Records of the geotechnical laboratory testing are provided in Appendix C.

3.5.2 Gradation

Mechanical gradation analyses were completed on samples of the fill and the colluvium. Table 3-5 provides a summary of the gradation testing by NOVA and the testing reported in GSI 2016.

Table 3-5. Soil Gradation Testing

Source	Boring	Depth (feet)	Soil Description	Passing #200[1]	Classification After ASTM D2487
GSI	TP-2	3	Silty sand	32	SM
GSI	TP-7	3	Silty sand	7	SM
NOVA	TP-9	3.5-4	Clayey sand	48	SC
NOVA	TP-10	1-3	Sandy clay	60	CL
NOVA	TP-12	3-7	Silty sand	30	SM

Note: 'Passing #200' percent by weight passing the U.S. # 200 sieve (0.074 mm), after ASTM D6913.

3.5.3 Maximum Density and Optimum Moisture

Samples of near-surface soils to determine moisture-density relationship after ASTM D1557 (the 'modified Proctor') were tested by NOVA and reported in GSI 2016. Table 3-6 summarizes the results of this testing.

Table 3-6. Maximum Density and Optimum Moisture Testing

Source	Test Pit	Depth (feet)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
GSI	TP-2	2 – 3	Brown silty sand	130	10
GSI	TP-7	3	Yellow-brown silty sand	132	9
NOVA	TP-5	2 – 3	Orange-brown silty sand	131.7	9
NOVA	TP-12	3 – 7	Orange-brown silty sand	132.2	8
NOVA	TP-13	0 – 3	Dark brown clayey sand	133.9	6.6

3.5.4 Plasticity and Expansion Potential

Localized portions of the fill and colluvium in the near subsurface are clayey. Plasticity and expansion potential testing were undertaken to characterize these soils.

Atterberg Limit testing after ASTM D4318 was completed on representative samples of colluvium to assess its plasticity. Table 3-7 tabulates the results of this testing.

Table 3-7. Atterberg Limits

Test Pit	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Natural Moisture (%)	USGS Soil Type
TP- 9	3.5 - 4	36	17	19	5.1	CL
TP-10	1 - 3	43	16	27	6.8	CL

Testing to address the expansion potential of this same soil is reported in GSI 2016. Testing to determine Expansion Index (EI) after ASTM D4829 showed EI < 20, indicating a soil of Very Low expansion potential.

3.5.5 Direct Shear

Direct shear testing was completed by both GSI 2016 and NOVA. GSI 2016 reports the results of two remolded samples that were tested in direct shear after ASTM D3080. Table 3-8 summarizes the testing reported in GSI 2016.

Table 3-8. Direct Shear Testing, GSI 2016

Source	Test Pit	Depth (feet)	Soil Description	Peak		Residual	
				Friction Angle (°)	Cohesion (psf)	Friction Angle (°)	Cohesion (psf)
GSI	TP-2	2-3	Brown silty sand	32	218	32	166
GSI	TP-7	3	Brown silty sand	40	210	34	189

3.5.6 R-Value Testing

The R-Value for near-surface soils in the paved area was evaluated in general accordance with California Test (CT) 301 and ASTM D2844. Table 3-9 presents the result.

Table 3-9. R-Value

Test Pit	Depth (feet)	R-Value	Soil Description
TP-13	0-3	44	Dark brown silty sand

3.5.7 Chemical Testing

Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals. Water-soluble sulfates are used as an index of the potential for sulfate attack to concrete. These chemical tests were completed by both GSI 2016 and NOVA, as summarized on Table 3-10. This testing is discussed in more detail in Section 6.3.

Table 3-10. Chemical Testing

Source	Sample Ref		pH	Resistivity (Ohm-cm)	Sulfates		Chlorides	
	Test Pit	Depth (feet)			ppm	%	ppm	%
GSI	TP-2	2 - 3	8.8	660	170	0.017	180	0.018
NOVA	TP-10	1 - 3	8.3	730	<30	<0.00	21	0.002
NOVA	TP-15	2 - 3	8.2	17000	<30	<0.00	43	0.004

4. SITE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

The site is located in the Peninsular Range geomorphic province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California. The province varies in width from approximately 30 to 100 miles. The project area is located on the boundary of the Coastal Plains and Foothills physiographic provinces.

The western portion of this area of the province has undergone several episodes of marine inundation and subsequent marine regression (coastline changes) throughout the last 54 million years. These events have resulted in the deposition of a thick sequence of marine and nonmarine sedimentary rocks on the basement igneous rocks of the Peninsular Ranges batholith and the metamorphic rocks into which the batholith was intruded.

Gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and nonmarine terrace deposits, formed as the sea receded from the land. Accelerated fluvial erosion during periods of heavy rainfall, along with the lowering of base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons that characterize the landforms in western San Diego County.

The site lies within the Foothills physiographic province of San Diego County. The Foothills physiographic province can be generalized as granitic-type rocks of the Southern California batholith, which were intruded to the pre-existing Mesozoic volcanic and sedimentary rocks, often displaying low-grade metamorphism near the contacts.

4.1.2 Site Specific

Generally, the site is underlain (from ground surface downward) by a sequence of artificial fill, colluvium, and Cretaceous-aged tonalite (Kt), a granitic-type bedrock.

Where encountered, the artificial fill and colluvium range in thicknesses from less than 1 foot up to 8 feet bgs.

Figure 4-1 (following page) reproduces mapping of the surface geologic units in the site vicinity.

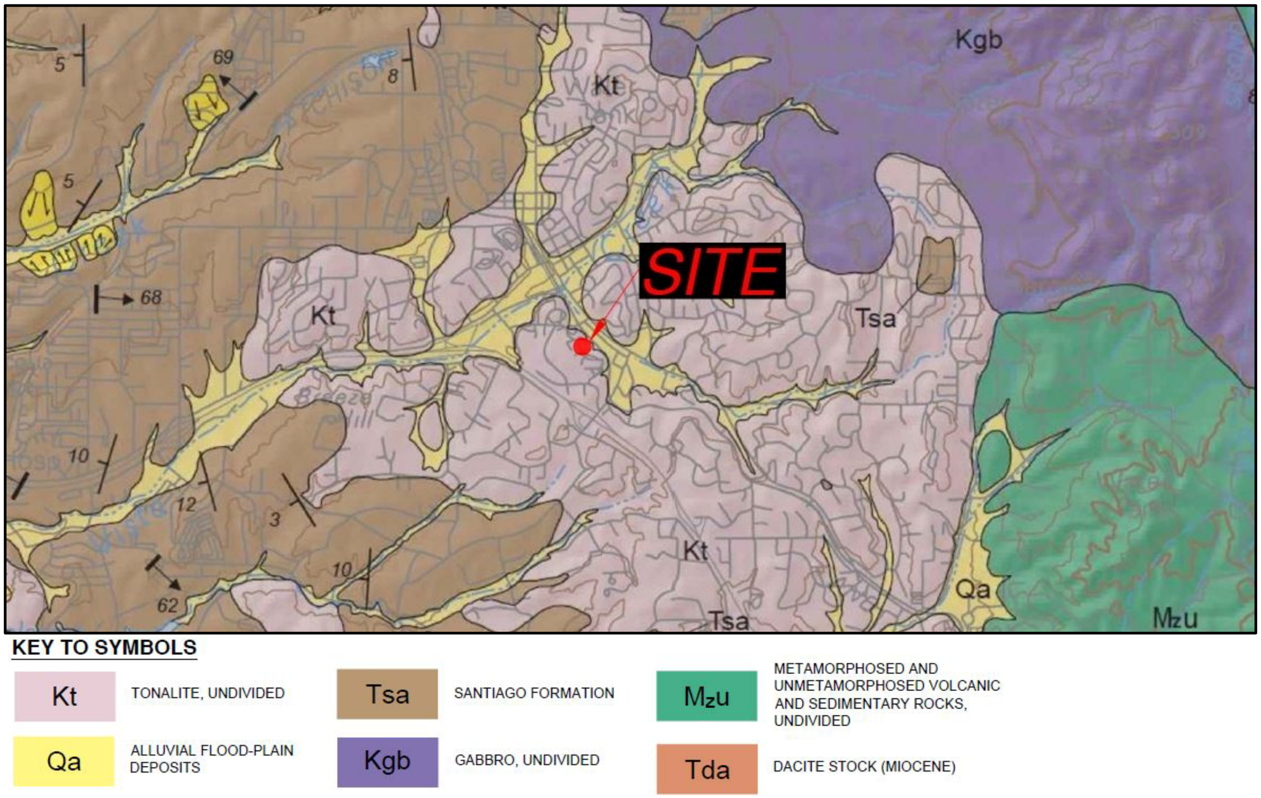


Figure 4-1. Geologic Mapping of the Site Vicinity

4.2 Surface, Subsurface, and Groundwater

4.2.1 Surface

The project site is largely vacant, with surface vegetated by grasses and weeds, and several trees are located across the site. Single-family residential structures occupy the northern and southern parcels of the site. Smaller erosional features are evident in areas of a steeper topographic gradient.

The site is characterized by a relatively sloping ground surface. The steepest topographic gradients range from about +360 feet mean sea level (msl) at the eastern periphery of the site to about +430 feet msl at Lado De Loma Drive, a surface gradient of about 20%.

Relatively unweathered outcrops of tonalite are visible at the surface in some areas across the site. Figures 4-2 and 4-3 (following page) depict surface conditions.

Figure 4-2. Surface Conditions, Looking East from Lado De Loma Drive

Figure 4-3. Surface Conditions, Looking South from Guajome Street

4.2.2 Subsurface

Subsurface conditions indicated by this investigation, as well as by GSI 2016 may be generalized to occur as the sequence of soil units in the near subsurface as described below.

Artificial Fill (Afu). The undocumented artificial fill encountered at the site is generally found in three localized zones, which are presented on Plate 1 – Subsurface Investigation Map. Fill in the two eastern areas are less than 3 feet in thickness, observed to be grayish-brown silty sand with gravel of loose consistency. Some construction debris was observed within the fill.

In the southern portion of the site, the fill encountered was 8 feet deep. This fill was used to create an area for the residence located along Lado De Loma Drive. In this area, the fill was observed to be dark brown to dark red-brown silty fine to medium-grained sand with abundant refuse and construction debris. This undocumented soil is at risk for wide variations in quality and consistency.

Quaternary Colluvium (Qcol). The colluvium was encountered either below the fill or from the surface in all borings except TP-21. This unit was generally observed to be olive to reddish-brown silty sand and brown to dark brown clayey sand and sandy clay of loose/soft to medium dense in consistency/relative density. The unit was generally porous with some gravel and cobbles encountered. The colluvium is depicted on Figure 4-4.

Figure 4-4. Colluvium

Cretaceous Tonalite (Kt). This unit underlies the entire site and was encountered in all test pits. The upper portion of this unit is generally highly weathered and excavates to medium dense to dense medium to coarse-grained, light yellow-brown to grayish silty sand. This soil is derived from in-place weathering of the parent tonalite (Kt). The degree of weathering diminishes with depth. The seismic refraction lines performed by NOVA indicate the bottoms of the deep excavations will be in marginally rippable material with velocities between 6,000 to 7,000 feet per second. Appendix D presents the findings from the seismic lines performed in the locations of the deepest cuts.

Based on NOVA's extensive experience with grading in this tonalite unit, localized highly unweathered zones (core stones, or floaters) are commonly encountered in the tonalite that require extra effort and time to remove from the surrounding more weathered rock.

Figure 4-5 depicts the highly weathered tonalite at the bottom of Test Pit TP-5 and Figure 4-6 (following page) presents a sample of the less weathered tonalite encountered during the investigation.

Figure 4-5. Highly Weathered Tonalite

Figure 4-6. Less Weathered Tonalite

4.2.3 Groundwater

Groundwater was not encountered in any of the test pits by NOVA or in the test pits reported in GSI 2016. Groundwater is not anticipated to be a constraint to development. Although not encountered within the subsurface explorations, seepage from perched groundwater may be encountered locally in excavations especially in landscaping and sloping areas of the site. If dewatering is necessary, the dewatering method should be evaluated and implemented by an experienced dewatering subcontractor.

Infiltrating stormwater can ‘perch’ atop localized zones of lower permeability soil that exist above the static groundwater level. Localized perched groundwater conditions may also develop once site development is complete and landscape irrigation commences. These perched water issues are difficult to predict but may be mitigated if the need arises.

4.2.4 Surface Water

NOVA did not observe any evidence of seeps, springs, or surface staining that would suggest the recent problems with surface water on the site. As is common for the area, smaller erosional features are evident in areas of a steeper topographic gradient.

5. REVIEW OF GEOLOGIC, SOIL, AND SITING HAZARDS

5.1 Overview

This section provides review of geologic, soils, and siting-related hazards common to this region of California, considering each for its potential to affect the planned construction.

The review identified two primary hazards, as abstracted below.

1. Hazard 1, Strong Ground Motion. This site is at risk for moderate-to-severe ground shaking in response to either a local moderate or more distant large-magnitude earthquake. While there is no risk of liquefaction or related seismic phenomena, strong ground motion could affect the site.

2. Hazard 2, Undocumented Fill. The area of the planned building addition is locally underlain by undocumented fill that locally extends to depths of about 8 feet depth but maybe thicker in unexplored areas of the site. No records exist regarding placement of this fill, such that the fill is considered 'undocumented', at risk for wide variations in quality and consistency. Unmanaged by design, the fill colluvium may be compressible under loads from the new structure.

3. Difficult Excavation. Based on the seismic refraction lines, the majority of material to be excavated is considered rippable near the surface, to marginally rippable at the bottoms of the excavations. Sound tonalite rock underlies the entire site. Locally, this rock occurs within the near subsurface, and may be encountered within depths planned for excavation for building pads and infrastructure. The sound rock may locally be difficult to loosen for removal, adding cost and time to the planned earthwork.

The following subsections detail the geologic, soils, and siting-related hazards that may affect the planned improvements to the site.

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structure. Major known active faults in the region consist generally of en echelon, northwest striking, right-lateral, strike-slip faults. These include the San Andreas, Elsinore, and San Jacinto faults located northeast of the site, and the San Clemente, San Diego Trough, and Agua Blanca-Coronado Bank faults located to the west of the site.

The tectonic setting of western San Diego County includes major north and northwest striking fault zones, the most prominent and active of which is the Newport-Inglewood-Rose Canyon Fault Zone (NIRC). The NIRC includes offshore faulting from Newport Beach southeastward past North San Diego County until it returns back onshore at La Jolla Cove.

The seismicity of the site was evaluated utilizing the Seismic Design Maps web-based analytical tool provided by Structural Engineers Association of California (SEA) and California's Office of Statewide Health Planning and Development (OSHPD). The USGS Unified Hazard Tool indicates the site may be subjected to a Magnitude 6.9 seismic event, with a corresponding site-adjusted Peak Ground Acceleration (PGAM) of 0.474g.

5.2.2 Fault Rupture

Earthquake Fault Zones (formerly known as special study zones) have been established along known active faults in California in accordance with the Alquist-Priolo Earthquake Fault Zoning Act. The site is not located in an Alquist-Priolo Earthquake Fault Zone. No active surface faults are mapped across the site. The nearest active fault is the Oceanside section of the NIRC fault zone, located approximately 10.9 miles to the southwest. Evidence of active faulting was not observed at the site during the time of the field evaluation. No active faults are known to underlie or project toward the site. The probability of fault rupture is considered very low.

Figure 5-1 shows the locations of known faults in the general site area.

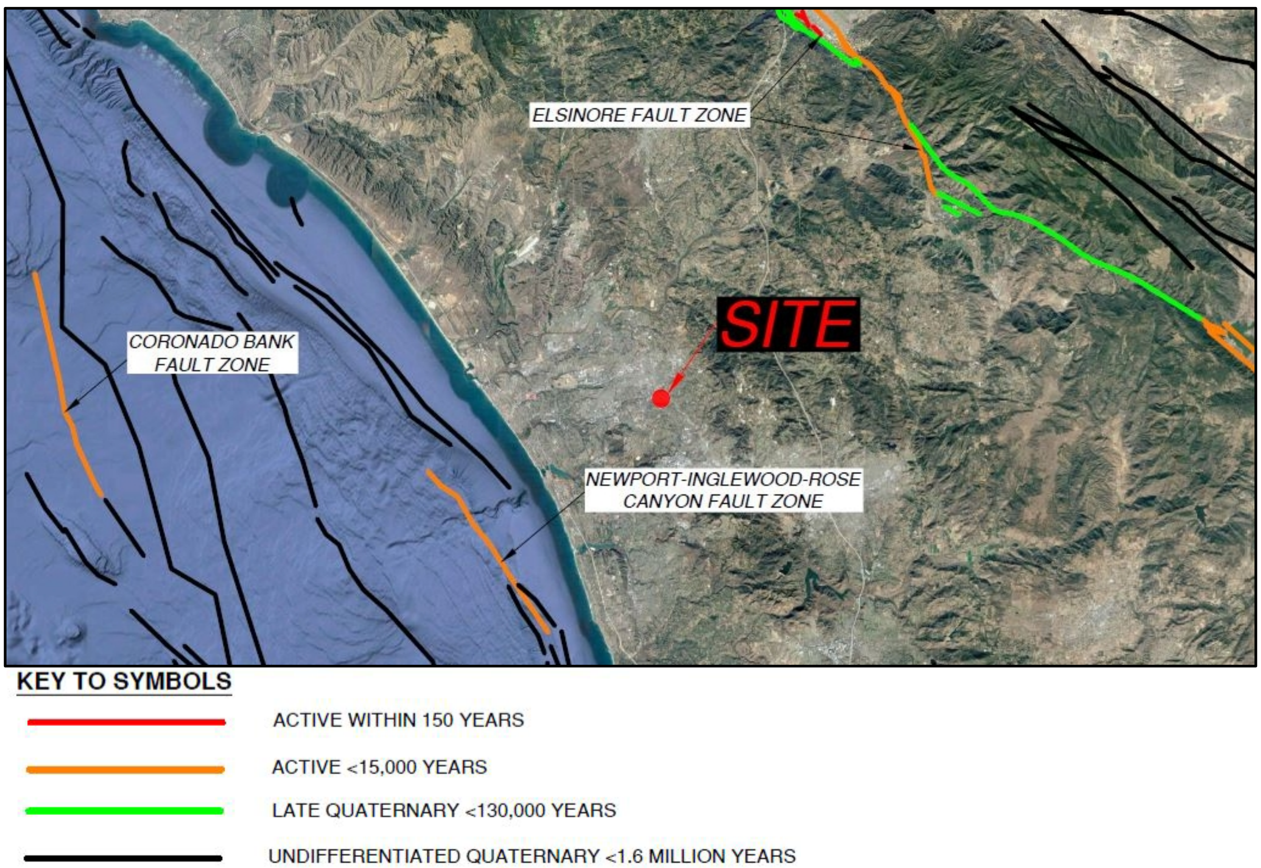


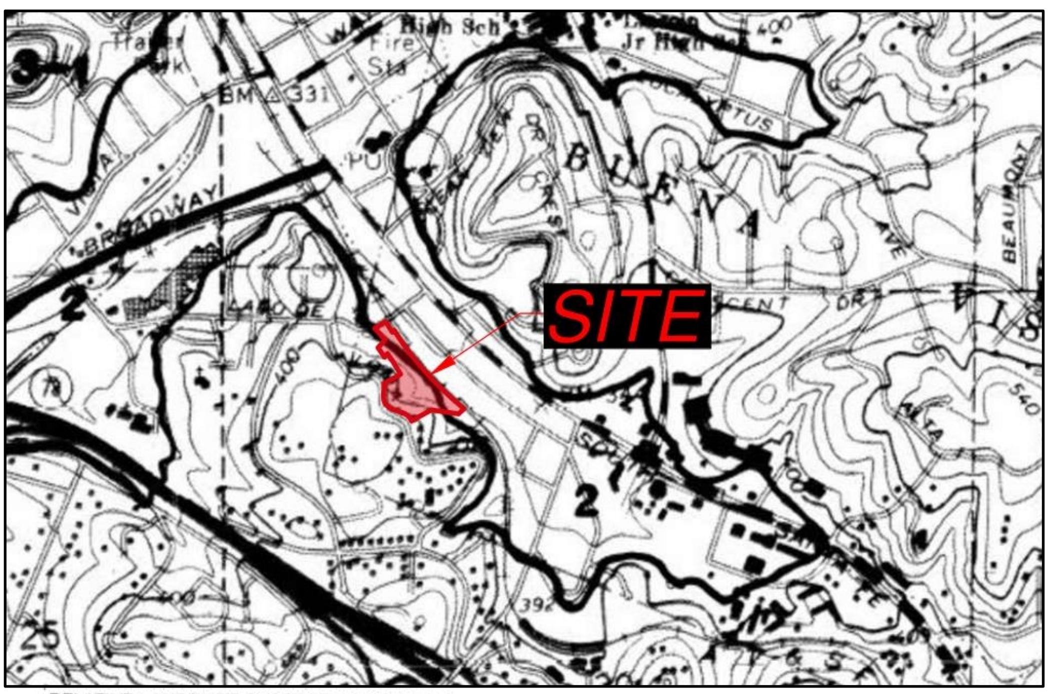
Figure 5-1. Faulting in the Site Vicinity
(Source: USGS US Quaternary Fault Database)

5.2.1 Landslide

As used herein, 'landslide' describes downslope displacement of a mass of rock, soil, and/or debris by sliding, flowing, or falling. Such mass earth movements are greater than about 10 feet thick and larger than 300 feet across. Landslides typically include cohesive block glides and disrupted slumps that are formed by translation or rotation of the slope materials along one or more slip surfaces. These mass displacements can also include similarly larger scale, but more narrowly confined modes of mass wasting such as 'mud flows' and 'debris flows'.

The causes of classic landslides start with a preexisting condition - characteristically, a plane of weak soil or rock inherent within the rock or soil mass. Thereafter, movement may be precipitated by earthquakes, wet weather, and changes to the structure or loading conditions on a slope (e.g., by erosion, cutting, filling, release of water from broken pipes, etc.).

Geologic reconnaissance, geologic mapping, and review of aerial photography indicated no evidence of active or dormant landsliding. Further clues to landslide hazards can also be obtained by review of mapping that depicts both historic landslides and landslide-prone topography. Figure 5-2 reproduces such mapping for the site area. The mapping indicates that the site is in an area judged to be 'marginally susceptible' to landsliding.



RELATIVE LANDSLIDE SUSCEPTIBILITY AREAS

1	2	3-1	3-2	4-1	4-2
Least Susceptible	Marginally Susceptible	Generally Susceptible		Most Susceptible	

Increasing Landslide Susceptibility →

Figure 5-2. Landslide Risk in the Site Area
(source: adapted from Tan 1995)

In consideration of the inherent stability of the formational tonalite, geologic reconnaissance, and review of published information, NOVA considers the site to be at low risk for landsliding in its current configuration.

5.2.2 Ground Lurching

Seismically induced ground lurching occurs when weaker soil masses move at right angles to a cliff or steep slope in response to seismic waves. Structures built on these masses can experience significant lateral and vertical deformations if ground lurching occurs.

The phenomenon is usually associated with soft, unconsolidated soils with low cohesion adjacent to a slope. The occurrence of weathered and unweathered granitic rock in the subsurface suggests the site is not at risk for ground lurching.

5.3 Soil Hazards

5.3.1 Embankment Stability

As used herein, ‘embankment stability’ is intended to mean the safety of localized natural or man-made embankments against failure. Unlike landslides described above, embankment stability can include smaller scale slope failures such as erosion-related washouts and more subtle, less evident processes such as soil creep.

No new slopes are planned as part of the site improvements.

5.3.2 Seismic

<u>Liquefaction</u>

‘Liquefaction’ refers to the loss of soil strength during a seismic event. The phenomenon is observed in areas that include geologically ‘younger’ soils (i.e., soils of Holocene age), shallow water table (less than about 60 feet in depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. The seismic ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, which causes the soils to lose strength.

Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

The dense to very dense tonalite rock at this site has no potential for liquefaction.

<u>Seismic Compression</u>

Apart from liquefaction, a strong seismic event can induce settlement within loose to moderately dense, unsaturated granular soils (‘seismic compression’). The tonalite will not be prone to settlement driven by seismic compression.

5.3.3 Expansive Soil

Expansive soils are characterized by their ability to undergo significant volume changes (shrinking or swelling) due to variations in moisture content. These volume changes can be damaging to structures. Nationally, the value of property damage caused by expansive soils is exceeded only by that caused by termites.

As is discussed in Section 3, the fill and colluvial soils have been characterized by testing to determine Expansion Index ('EI', after ASTM D4829). EI has been adopted by the California Building Code ('CBC', Section 1803.5.3) for characterization of expansive soils. Table 5-1 tabulates the qualitative descriptors of expansion potential based upon EI.

Testing to address the expansion potential of the colluvium is reported in GSI 2016. Testing to determine Expansion Index (EI) after ASTM D4829 showed EI < 20, indicating a soil of Very Low expansion potential. Inspections of the samples recovered from the twenty-five (25) test pits completed by NOVA show the fill and colluvium are of low expansion potential.

Table 5-1. Descriptors of Expansive Soil after ASTM D4829

Expansion Index (EI)	Expansion Potential, ASTM D4829	Expansion Classification, 2016 CBC
0 to 20	Very Low	Non-Expansive
21 to 50	Low	Expansive
51 to 90	Medium	
91 to 130	High	
>130	Very high	

5.3.4 Hydro-Collapsible Soils

Hydro-collapsible soils are common in the arid climates of the western United States in specific depositional environments - principally in areas of young alluvial fans, debris flow sediments, dune sands, and loess (wind-blown sediment) deposits. These soils are characterized by low *in situ* density, low moisture contents, and relatively high unwetted strength.

The soil grains of hydro-collapsible soils were initially deposited in a loose state (i.e., high initial 'void ratio') and thereafter lightly bonded by water-sensitive binding agents (e.g., clay particles, low-grade cementation, etc.). While relatively strong in a dry state, the introduction of water into these soils causes the binding agents to fail. Destruction of the bonds/binding causes relatively rapid densification and volume loss (collapse) of the soil. This change is manifested at the ground surface as subsidence or settlement. Ground settlements from the wetting can be damaging to structures and civil works.

Tonalite is not potentially hydro-collapsible, however, the existing fill and colluvial materials may be. Remedial grading recommendations for these materials are presented in the next section of the report.

5.3.5 Near Surface Rock

Tonalite, a granitic-type rock, underlies the entire site. This rock occurs within the near subsurface, within depths planned for excavation of building pads and infrastructure. The majority of this rock is weakened by weathering and will be excavatable by larger earthwork equipment in good working condition with experienced operators. However, localized zones of sound rock resistant to weathering will be encountered. These core stones and floaters should be expected to be difficult to loosen for removal, adding cost and time to the planned earthwork.

In addition, the seismic refraction surveys indicate that marginally rippable to unrippable zones of tonalite are as shallow as 20 feet below ground surface. At this time cuts up to 35 feet are planned. Given the variability in the density of the underlying rock, difficult excavation (with the possibility of blasting) should be anticipated with cuts of this depth.

5.4 Siting Hazards

5.4.1 Effect on Adjacent Properties

The proposed development will not affect the structural integrity of adjacent properties or existing public improvements and street rights-of-way located adjacent to the site if the recommendations of this report are incorporated into design.

5.4.2 Flood

The site is not mapped within a FEMA-designated flood zone and is designated as Flood “Zone X” (FEMA, 2006). Zone X is an “Area of 500-year flood: areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood”.

Figure 5-3 (following page) reproduces flood mapping of the site vicinity.

5.4.3 Tsunami

Tsunami (‘tidal wave’) describes a series of fast-moving, long-period ocean waves caused by earthquakes or volcanic eruptions. The elevation and distance of the site from the ocean preclude this threat.

5.4.4 Seiche

Seiches are standing waves that develop in an enclosed or partially enclosed body of water such as lakes or reservoirs. Harbors or inlets can also develop seiches. They are most commonly caused by wind and atmospheric pressure changes. Seiches can also result from seismic events and tsunamis.

The site is not located near a body of water that could generate a seiche.

Figure 5-3. Flood Mapping of the Site Area

6. EARTHWORK AND FOUNDATIONS

6.1 Overview

6.1.1 Review of Site Hazards

Section 5 provides review of geologic, soils, and siting-related hazards common to this region of California, considering each for its potential to affect the planned construction. That review identifies two primary site hazards, as abstracted below.

1. Hazard 1, Strong Ground Motion. This site is at risk for moderate-to-severe ground shaking in response to either a local moderate or more distant large-magnitude earthquake. Section 6.2 provides parameters for seismic-resistant structural design.

2. Hazard 2, Undocumented Fill. The area of the planned building addition is locally underlain by undocumented fill that locally extends to depths of about 3 to 5 feet but may be thicker in unexplored areas of the site. Section 6.3 guides earthwork, including planning to mitigate the risk of undocumented fill.

3. Hazard 3, Difficult Excavation. Sound tonalite rock underlies the entire site. Locally, this rock occurs within the near subsurface, within depths planned for excavation for building pads and infrastructure. Sound rock may locally be difficult to loosen for removal, adding cost and time to the planned earthwork. Section 6.3 provides guidance for managing the near-surface occurrence of rock.

The remainder of this section provides design guidance for development of the site, including guidance for design and construction of foundations, retaining walls, roadways, and utilities.

6.1.2 Site Suitability

Based upon the indications of the field and laboratory data developed for this investigation, as well as review of previously developed subsurface information, it is the judgment of NOVA that the site is suitable for proposed development, provided the geotechnical recommendations described herein are followed.

Development as presently envisioned will not affect the structural integrity of adjacent properties or existing public improvements and street rights-of-way located adjacent to the site if the recommendations of this report are incorporated into project design; however, special care should be taken along the corner of the central portions of the six-story apartment structure, that as currently designed, is 6 feet from the neighboring property line. At this point, it has 22 feet of cut proposed to reach the garage floor level. If soil nails are proposed for shoring this excavation, the soil nails may not extend below the neighboring property without consent from the property owner.

6.1.3 Review and Surveillance

It is intended that the recommendations provided herein be sufficient to develop the project in general accordance with the 2019 California Building Code (CBC) requirements.

NOVA should review the grading plans, foundation plans, and geotechnical-related specifications as they become available to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project. All earthwork related to site and foundation preparation should be completed under the observation of NOVA.

Section 9 design addresses review and construction surveillance in more detail.

6.2 Seismic Design Parameters

6.2.1 Site Class

The Site Class has been determined from ASCE 7, Table 20.3-1. The depth of soil information available for this site is limited. However, as is well-established by the geologic literature and by experience at numerous nearby sites, the granitic rock unit that occurs in the near subsurface extends to depths greater than 100 feet, such that the site may be classified as Site Class C per ASCE 7-16 (Table 20.3-1).

6.2.2 Seismic Design Parameters

Table 6-1 provides seismic design parameters in accordance with ASCE 7-16.

Table 6-1. Site-Specific Seismic Design Parameters, ASCE 7-16

Parameter	Site Class C
Site Latitude, degrees	33.1955
Site Longitude, degrees	-117.2407
Mapped Short Period Spectral Acceleration, S	0.914g
Mapped One-Second Period Spectral Acceleration, S_1	0.338g
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.097g
One-Second Period Spectral Acceleration Adjusted For Site, S_{M1} Class, S_{M1}	0.506g
Design Short Period Spectral Acceleration, S_{DS}	0.731g
Design One-Second Period Spectral Acceleration, S_{D1}	0.338g
Site Modified Peak Ground Acceleration, PGAM	0.474 g

source: *ASCE 7-16 Hazard Tool,* found at https://asce7hazardtool.online/

6.3 Corrosivity and Sulfates

6.3.1 General

Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals. Levels of water-soluble sulfates are correlated with the potential for sulfate attack to concrete. These chemical tests were performed on representative samples of the near-surface soils.

The results of the testing are tabulated in Table 6-2.

Table 6-2. Abstract of Chemical Testing

Source	Sample Ref		pH	Resistivity (Ohm-cm)	Sulfates		Chlorides	
	Boring	Depth (feet)			ppm	%	ppm	%
GSI	TP-2	2 - 3	8.8	660	170	0.017	180	0.018
NOVA	TP-10	1 - 3	8.3	730	<30	<0.003	21	0.002
NOVA	TP-15	2 - 3	8.2	17000	<30	<0.003	43	0.004

6.3.2 Metals

Caltrans considers a soil to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:

- chloride concentration is 500 parts per million (ppm) or greater,
- sulfate concentration is 2,000 ppm (0.2%) or greater, or
- the pH is 5.5 or less.

Based on the Caltrans criteria, the on-site soils would not be considered 'corrosive' to buried metals. Appendix D provides records of the chemical testing that include estimates of the life expectancy of buried metal culverts of varying gauge.

In addition to the above parameters, the risk of soil corrosivity affecting buried metals is considered by determination of electrical resistivity (ρ). Soil resistivity may be used to express the corrosivity of soil only in unsaturated soils. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of DC electrical current from the metal into the soil. As the resistivity of the soil decreases, the corrosivity generally increases. A common qualitative correlation (cited in Romanoff 1989, NACE 2007) between soil resistivity and corrosivity to ferrous metals is tabulated in Table 6-3 (following page).

Table 6-3. Soil Resistivity and Corrosion Potential

Minimum Soil Resistivity (Ω-cm)	**Qualitative Corrosion Potential**
0 to 2,000	Severe
2,000 to 10,000	Moderate
10,000 to 30,000	Mild
Over 30,000	Not Likely

Despite the relatively benign environment for corrosivity indicated by pH and water-soluble chlorides, the resistivity testing suggests that design should consider that the soils may be corrosive to embedded ferrous metals.

Typical recommendations for mitigation of such corrosion potential in embedded ferrous metals include:

- a high-quality protective coating such as an 18-mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar;
- electrical isolation from above grade ferrous metals and other dissimilar metals by means of dielectric fittings in utilities and exposed metal structures breaking grade; and,
- steel and wire reinforcement within concrete having contact with the site soils should have at least 2-inches of concrete cover.

If extremely sensitive ferrous metals are expected to be placed in contact with the site soils, it may be desirable to consult a corrosion specialist regarding choosing the construction materials and/or protection design for the objects of concern.

6.3.3 Sulfate Attack

As shown in Table 6-2, the soil samples tested indicate water-soluble sulfate (SO_4) content of less than 170 parts per million (‘ppm,’ less than 0.017% by weight) for the fill and weathered tonalite. With SO4 < 0.10% by weight, the American Concrete Institute (ACI) 318-08 considers the soil to have negligible potential (S0) potential for sulfate attack to embedded concrete.

Table 6-4 reproduces the Exposure Categories considered by ACI.

Table 6-4. Exposure Categories and Requirements for Water-Soluble Sulfates

Exposure Category	**Class**	**Water-Soluble Sulfate (SO_4) In Soil**	**Cement Type (ASTM C150)**	**Max Water-Cement Ratio**	**Min. f'_c (psi)**
Not Applicable	S0	$SO_4 < 0.10$	-	-	-
Moderate	S1	$0.10 \le SO_4 < 0.20$	II	0.50	4,000
Severe	S2	$0.20 \le SO_4 \le 2.00$	V	0.45	4,500
Very severe	S3	$SO_4 > 2.0$	V + pozzolan	0.45	4,500

Adapted from: ACI 318-08, Building Code Requirements for Structural Concrete

6.3.4 Limitations

Testing to determine several of the chemical parameters that indicate a potential for soils to be corrosive to construction materials is traditionally completed by the geotechnical engineer, comparing testing results with a variety of indices regarding corrosion potential.

Like most geotechnical consultants, NOVA does not practice in the field of corrosion protection, since this is not specifically a geotechnical issue. Should you require more information, a specialty corrosion consultant should be retained to address these issues.

6.3.5 Review and Surveillance

NOVA should review the grading plan, foundation plan, and geotechnical-related specifications as they become available to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project.

All earthwork related to site and foundation preparation should be completed under the observation of NOVA, the Geotechnical Engineer-of-Record (GEOR) for this work.

6.4 Earthwork

6.4.1 General

Based upon the design concept that is currently understood, NOVA expects that earthwork will include (i) grading to create the new building pads, (ii) excavations for foundations for walls and structures, and (iii) excavation/backfill for utilities.

Earthwork should be performed in accordance with Section 300 of the most recent approved edition of the "*Standard Specifications for Public Works Construction*" and "*Regional Supplement Amendments*."

6.4.2 Site Preparation

Construction-related erosion and sedimentation must be controlled in accordance with Best Management Practices, as well as current City and State requirements. These controls should be established at the outset of site disturbance.

The site will be required to be cleared prior to construction, including removal of all vegetation and organics.

6.4.3 Excavation Characteristics, Planning and Contracting

<u>Excavation Characteristics</u>

As previously discussed in Section 3, Section 4, and Section 5, tonalite rock shallowly underlies the entire site. There will be significant cuts (up to 28 feet) into this rock. Utility installation will also likely require excavation into this material. Seismic lines indicated the majority of the material is rippable using appropriate construction equipment in good working order by an experienced operator; however, where this rock is unweathered, sound rock may be difficult to mechanically loosen for removal, adding cost and time to the planned earthwork.

This subsection addresses planning for difficult excavation below the upper 5 feet of the subsurface. The discussion in this section is intended to address potential difficulties with this earthwork.

- o <u>Unit 1, Fill/Colluvium</u>. This soil will be readily excavated by conventional equipment. However, cobbles and construction debris may be encountered within this layer.
- o <u>Unit 2, Weathered Tonalite</u>. This interval extends to a depth of about 15 feet and is comprised of tonalite weathered to a dense to very dense silty sand soil. This unit has seismic velocities of about 3,600 fps to 5,500 fps. This interval will likely include some zones of sound rock (floaters). NOVA expects that the majority of this interval will be able to be excavated using heavy earthmoving equipment in good working order. However, floaters may be encountered that would require special handling by the contractor.
- o <u>Zone 3, Less Weathered to Sound Tonalite</u>. Below about 15 feet depth, the site is underlain by sound, continuous rock with discontinuities (joints and fractures) that will diminish in frequency with increasing depth, making it more difficult to excavate. The compressive strength of sound, unweathered, unfractured tonalite can be similar to that of a low-grade concrete. The ability of ripping to mechanically loosen this interval will be dependent upon the frequency of fractures and joints.

  The seismic velocity of this unit is 6,000 to 7,000 fps. For perspective, the seismic velocity of weathered concrete is in the range of 5,500 fps to about 8,000 fps. It is expected that the upper perhaps 15 to 20 feet of this unit will generally be rippable with a D10 Bulldozer, though localized areas of sound, unfractured rock may require special effort. Floaters may be encountered that would require special handling by the contractor.

Care should be taken in use of these data. While measurements of seismic velocity provide a semi-quantitative determination of the potential for rippability, it should be understood that numerous variables are associated with actual rippability, and these velocities are intended to serve only as an indicator. Potential contractors for this work should be screened for experience with rock of this nature in the San Diego area.

<u>Contracting for Earthwork</u>

The complexity of the soil profile - in particular, the dramatic differential in strength between fill/colluvium and tonalite rock - complicates planning for earthwork. It is NOVA's experience that questions regarding '*What is rock*?' often drive cost overruns and related post-construction litigation for earthwork in soil profiles such as occur at this site.

As is noted above (and as is also discussed in Section 3.4), despite a well-developed base of subsurface information, pre-determination of actual rippability is difficult. Potential contractors for this work should be screened to limit bidding to those with experience excavating rock and subsurface conditions similar to this site.

Planning for earthwork for this project should consider establishing contractual criteria and pay items for ‘rock’ prior to initiating excavation. For example, “rock” may be defined based upon the nature of the work and the excavation equipment, as are driven by subsurface conditions. An example of a performance-based specification is cited below.

1. General Excavations. Rock is any material that cannot be loosened with (i) a single tooth ripper drawn by a crawler tractor having a drawbar pull rated at not less than 56,000 pounds (for example, a Caterpillar D8, or equivalent); or, (ii) a front-end loader with a minimum bucket breakout force of 25,600 pounds (for example, a Caterpillar 977 or equivalent).

2. Trench Excavation. Rock is any material that cannot be excavated with a backhoe having a bucket curling force rated at not less than 33,000 pounds and a bucket equipped with excavation teeth (for example, a Caterpillar 225B or equivalent).

6.4.4 Select Fill

Materials

All engineered fill and backfill should be Select Fill, a mineral soil free of organics or regulated constituents, with the characteristics listed below:

- at least 40% by weight finer than ¼ inches in size;
- soils classified as GW, GM, SW, or SM after ASTM D2487;
- maximum particle size of 6 inches; and,
- expansion index (EI) less than 30 (i.e., EI < 30, after ASTM D4829).

Most of the fill/colluvium that is now in place will conform to the above criteria. However, it should be noted that large cobbles and construction debris were encountered within the fill/colluvium layer that are not suitable and will require removal based on the specifications above.

Much of the weathered tonalite will meet the above criteria. However, this unit may include zones of sound, continuous rock (‘floaters’).

Oversized Material

Excavations will generate oversized material, defined as rocks or cemented clasts greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in fill, used as landscape material, or disposed of off-site.

Moisture Conditioning

All fill and backfill should be moisture conditioned to at least 2% above the optimum moisture content prior to densification.

Placement

Fill should be placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift. For most self-propelled construction equipment, this will limit loose lifts to on the order of 10 inches or less. Lift thickness for hand-operated equipment used in constrained spaces (e.g., walk-behind compactors used in utility trenches) will be limited to about 4 inches or less.

Compaction

Select Fill must be densified to at least 90% relative compaction after ASTM D1557. The fill must be densified using specialty compaction equipment and methods appropriate to the soil type. Vibratory compaction should be used for these dominantly sandy soils.

6.4.5 Remedial Grading

Building

The proposed buildings should not be underlain by a cut/fill transition or a transition from shallow fill to deep fill. To mitigate such transitions and reduce the potential for differential settlement, the buildings can be supported on shallow spread footings with bottom levels bearing entirely on a relatively uniform thickness of compacted fill. Alternatively, the buildings can be supported on shallow spread footings with bottom levels bearing entirely on competent tonalite. Recommendations for both options are provided below.

1. Footings Bearing on Compacted Fill

   Beneath the proposed building pads, the fill/colluvium not removed by planned cuts should be removed to contact with the weathered tonalite. Additionally, tonalite should be excavated a minimum of 2 feet below the bottom of the deepest footing. Horizontally, the excavations should extend at least 5 feet outside the planned perimeter building foundations or up to existing improvements, whichever is less. NOVA should observe the conditions exposed at the bottom of excavations to evaluate whether additional excavation is recommended. The resulting excavation should then be filled to the finished pad grade with compacted fill in conformance with Select Fill requirements in Section 6.4.4.

2. Footings Bearing on Competent Tonalite

   These footings may be supported directly on excavations in the competent tonalite. NOVA should observe conditions exposed in the bottom of the excavation to determine if additional excavation is required.

   Current grading plans indicate that there are portions of the building where foundations will either be in proposed fill, or in existing fill/colluvium. In these areas, the existing fill and colluvium should be removed to competent tonalite bedrock, and a 2-sack sand/cement slurry can be placed between the competent tonalite and the proposed bottom of footing elevation.

Site Walls and Retaining Walls

Beneath proposed site walls and retaining walls not connected to buildings, the existing fill and colluvium should be excavated down to competent tonalite. Horizontally, the excavations should extend at least 2 feet outside the wall footing.

If competent tonalite is exposed, additional excavation need not be performed. NOVA should observe the conditions exposed at the bottom of excavations to evaluate whether additional excavation is recommended. Any required fill should conform to Select Fill requirements in Section 6.4.4.

Slopes

Permanent slopes should be constructed no steeper than 2:1 (horizontal:vertical). Keyways should be constructed at the toe of fill slopes. Keys should extend at least 2 feet into competent tonalite. Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment or by overfilling and cutting back to design grade. Fills should be benched into sloping ground inclined steeper than 5:1 (h:v). In our opinion, slopes constructed no steeper than 2:1 (h:v) will possess an adequate factor of safety. An engineering geologist should observe cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. Slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

Pavements

Beneath proposed vehicular pavement areas, to include fills for the proposed parking lot, the existing fill and colluvial soils should be excavated to competent tonalite. Horizontally, excavations should extend at least 2 feet outside the planned pavement or up to existing improvements, whichever is less. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended.

Flatwork

Beneath proposed hardscape areas, the on-site soils should be excavated to a depth of at least 2 feet below planned subgrade elevation. Horizontally, excavations should extend at least 2 feet outside the planned hardscape or up to existing improvements, whichever is less. If competent formational materials are exposed, excavation need not be performed. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. The resulting surface should then be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. If competent formational materials are exposed, scarification and recompaction need not be performed. The excavation should be filled with Select Fill in conformance with Section 6.4.4.

Exterior concrete slabs should be at least 4 inches thick and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project architect should select the final joint patterns. A 1-inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the “Greenbook” Standard Specifications for Public Works Construction.

6.4.6 Subgrade Stabilization

Excavation bottoms should be firm and unyielding prior to placing fill.

Areas of saturated or yielding subgrade may be stabilized by placement of a reinforcing geogrid (for example, Tensar® Triax® TX-5 or equivalent) on the excavation bottom, and then at least 12 inches of compacted aggregate base. Once the surface of the aggregate base is firm enough to achieve compaction, the remaining excavation may be filled to finished pad grade with suitable material.

6.4.7 Temporary Excavations

Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations in fill should be laid back no steeper than 1:1 (horizontal:vertical). Deeper temporary excavations in competent tonalite should be laid back no steeper than ¾:1 (h:v). The faces of temporary slopes should be inspected daily by the contractor’s Competent Person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing, or raveling should be brought to the attention of the engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. NOVA should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (h:v) downward from the outside bottom edge of existing structures or improvements will require shoring. Soldier piles and lagging, internally braced shoring, or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations immediately adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring.

Development of temporary slopes should be evaluated on a case by case bases, for example, temporary slopes less than 15 feet, developed in sound tonalite bedrock may be inclined as steep as ½:1. These slopes should be individually evaluated by the geotechnical engineer as planning for the construction of these walls moves forward.

6.4.8 Groundwater Seepage

Although not encountered within the subsurface explorations, seepage from perched groundwater may be encountered locally in excavations especially in landscaping and sloping areas of the site. If dewatering is necessary, the dewatering method should be evaluated and implemented by an experienced dewatering subcontractor.

6.4.9 Trenching and Backfilling for Utilities

Given the variability in hardness within the tonalite, NOVA recommends consideration be given to over excavating the proposed utilities to one foot below the deepest utility for ease of installation.

Excavation for utility trenches must be performed in conformance with OSHA regulations contained in 29 CFR Part 1926. Utility trench excavations have the potential to degrade the properties of the adjacent soils. Utility trench walls that are allowed to move laterally will reduce the bearing capacity and increase settlement of adjacent footings and overlying slabs.

Backfill for utility trenches is as important as the original subgrade preparation or engineered fill placed to support either a foundation or slab. No utility should be aligned beneath footings within a projected 2H:1V limit from the edge of the footing to the base of the utility trench.

Backfill for utility trenches must be placed to meet the project specifications for the engineered fill of this project. Unless otherwise specified, the backfill for the utility trenches should be placed in 4- to 6-inch loose lifts and compacted to a minimum of 90% relative compaction after ASTM D1557 (the ‘modified Proctor’) at soil moisture +2% of the optimum moisture content. Up to 4 inches of bedding material placed directly under the pipes or conduits placed in the utility trench can be compacted to 90% relative compaction with respect to the Modified Proctor.

Compaction testing should be performed for every 20 cubic yards of backfill placed or each lift within 30 linear feet of trench, whichever is less.

Backfill of utility trenches should not be placed with water standing in the trench. If granular material is used for the backfill, the material should have a gradation that will filter protect the backfill material from the adjacent soils. If this gradation is not available, a geosynthetic non-woven filter fabric should be used to reduce the potential for the migration of fines into the backfill material.

6.5 Shallow Foundations

6.5.1 General

As previously mentioned, the proposed buildings can be supported on shallow spread footings with bottom levels bearing entirely on compacted fill. Alternatively, the proposed buildings can be supported on shallow spread footings with bottom levels bearing entirely on tonalite. In areas of deep fill/colluvium soils, a 2-sack sand/cement slurry can be placed between the proposed bottom of footing elevation and tonalite. Site walls and retaining walls not connected to buildings can be supported on shallow spread footings with bottom levels bearing on compacted fill or tonalite. Shade structures, covered walkways, and other pole-type structures can be supported on cast-in-drilled hole (CIDH) concrete piles.

The following subsections provide recommendations for shallow foundations. It is recommended that all footings, including any grade beams, be reinforced top and bottom. The actual reinforcement should be designed by the structural engineer.

6.5.2 Spread Footings

Footings should extend at least 24 inches below lowest adjacent finished grade. A minimum width of 24 inches is recommended for continuous footings and 36 inches for isolated or retaining wall footings. Footings proposed adjacent to the top of any slopes will need a minimum horizontal setback of 7 feet between the outer lower edge of the footing to the adjacent slope face.

An allowable bearing capacity of 2,500 psf can be used for spread footings supported on compacted fill. An allowable bearing capacity of 5,000 psf can be used for footings supported on tonalite. In areas of deep fill/colluvium soils, a 2-sack sand/cement slurry can be placed between the proposed bottom of footing elevation and competent tonalite. The allowable bearing capacity can be increased by 500 psf for each foot of depth below the minimum and 250 psf for each foot of width beyond the minimum up to a maximum of 5,000 psf on compacted fill or 7,500 psf on tonalite/slurry. The bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 10 feet exists between the lower outside footing edge and the face of the slope.

6.5.3 Lateral Resistance

Resistance to lateral loads will be provided by a combination of (i) friction between the soils and foundation interface, and (ii) passive pressure acting against the vertical portion of the footings.

Passive pressure may be calculated at 350 psf per foot of depth. An interface frictional coefficient of 0.35 may be used. No reduction is necessary when combining frictional and passive resistance. The passive pressure can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

6.5.4 Settlement

Structures supported on shallow foundations as recommended above will settle on the order of 0.5 inch or less, with about 70% of this settlement occurring during the construction period.

Angular distortion (Δ/L) due to differential settlement of adjacent, unevenly loaded footings will be less than 1 inch in 40 feet (i.e., Δ/L less than 1:480).

6.5.5 Ground Supported Slabs

The ground level of garage structures may employ conventional on-grade (ground-supported) slabs designed using a modulus of subgrade reaction (k) of 110 pounds per cubic inch (i.e., k = 110 pci).

The actual slab thickness and reinforcement should be designed by the structural engineer. NOVA recommends that slabs be a minimum 5 inches thick, reinforced by at least #4 bars placed at 18 inches on center each way and set within the middle third of the slabs by supporting the steel on chairs or concrete blocks ("dobies").

Minor cracking of concrete after curing due to drying and shrinkage is normal. Cracking is aggravated by a variety of factors, including high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due during curing. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or ‘weakened plane’ joints at frequent intervals. Joints should be laid out to form approximately square panels and never exceed a length-to-width ratio of 1.5 to 1. Proper joint spacing and depth are essential to effective control of random cracking. Joints are commonly spaced at distances equal to 24 to 30 times the slab thickness. Joint spacing that is greater than 15 feet should include the use of load transfer devices (dowels or diamond plates).

Contraction/control joints should be established to a depth of at least ¼ the slab thickness, as depicted in Figure 6-1 (following page).

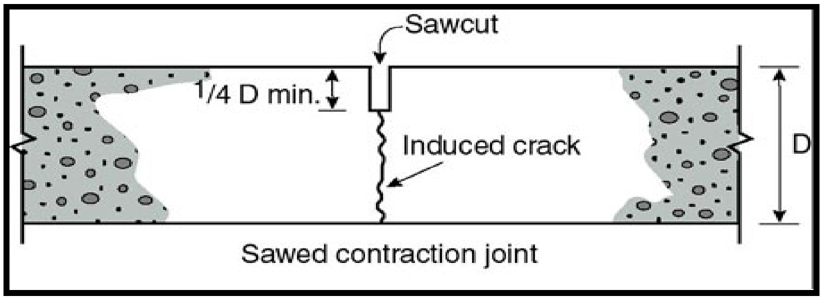


Figure 6-1. Sawed Contraction Joint

6.6 Moisture Barrier Beneath Slabs

6.6.1 Capillary Break

NOVA recommends that ground-supported slabs include a capillary break. The requirements for a capillary break ('sand layer') be determined in accordance with ACI Publication 302 *Guide for Concrete Floor and Slab Construction*.

A "capillary break" may consist of a 4-inch-thick layer of compacted, well-graded sand should be placed below the floor slab. This porous fill should be clean coarse sand or sound, durable gravel with not more than 5% coarser than the 1-inch sieve or more than 10% finer than the No. 4 sieve, such as AASHTO Coarse Aggregate No. 57.

6.6.2 Vapor Barrier

Ground-supported slabs beneath moisture-sensitive equipment or enclosures should include a vapor membrane. Membranes set below floor slabs should be rugged enough to withstand construction. If a vapor barrier is desired, a minimum 15-mil polyethylene membrane should be placed over the porous fill to preclude floor dampness.

NOVA recommends that a minimum 15-mil low permeance vapor membrane be used. For example, Carlisle-CCW produces the Blackline 400® underslab, vapor and air barrier, a 15-mil low-density polyethylene (LDPE) rated at 0.012 perms after ASTM E96.

6.6.3 Limitations of This Recommendation

Recommendations for moisture barriers are traditionally included with geotechnical foundation recommendations, though these requirements are primarily the responsibility of the structural engineer or architect. If there is particular concern regarding moisture-sensitive materials or equipment to be placed above the slab-on-grade, a qualified person (for example, such as the flooring subcontractor and/or structural engineer) should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. NOVA does not practice in the field of moisture vapor transmission since this is not specifically a geotechnical issue.

6.7 Conventional Retaining Walls

6.7.1 Shallow Foundations

Conventionally designed and reinforced concrete retaining walls should be developed on ground prepared in accordance with criteria provided in Section 6.4. Continuous shallow foundations may be designed in accordance with the criteria provided in Section 6.5. Retaining wall foundations should not be permitted to span cut and fill transitions. Foundations should bear entirely on formational soils or entirely on engineered fill.

6.7.2 Lateral Earth Pressures

General

Lateral earth pressures to walls are related to the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. The groundwater level will be well below wall levels.

Table 6-5 provides recommendations for lateral soil and groundwater wall loading to below-grade walls with level backfill for varying conditions of wall yield.

If footings or other surcharge loads are located a short distance outside the wall, these influences should be added to the lateral stress considered in the design of the wall.

Table 6-5. Wall Lateral Loads from Soil

Condition	Equivalent Fluid Pressure (psf/foot)	
	Level Backfill	2:1 Backfill Sloping Upwards
Active	35	60
At Rest	55	75
Passive	350	350

Seismic Increment

Walls shorter than 6 feet in height need not consider seismic loads.

For walls 6 feet or taller, the seismic earth pressure can be taken as equivalent to the pressure of a fluid pressure weighing 18 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, active earth pressure.

The total equivalent fluid pressure can be modeled as a triangular pressure distribution with the resultant acting at a height of H/3 up from the base of the wall, where H is the retained height of the wall. The passive pressure and bearing capacity can be increased by ⅓ in determining the seismic stability of the wall.

<u>Foundation Uplift</u>

A soil unit weight of 125 pcf may be assumed for calculating the weight of soil over the wall footing in design of cantilevered retaining walls.

<u>Wall Drainage</u>

The recommended equivalent fluid pressures provided in the preceding subsection assume that constantly functioning drainage systems are installed between walls and soil backfill to prevent the uncontrolled buildup of hydrostatic pressures and lateral stresses in excess of those stated. Figure 6-2 (following page) depicts a conceptual wall design in this regard.

Design for wall drainage may include the use of pre-engineered wall drainage panels or a properly compacted granular free-draining backfill material (EI <30). The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the ground adjacent to the base of the wall.

Numerous alternatives are available for collection of water behind retaining walls. The intent of Figure 6-2 is to depict the concepts described in this subsection.

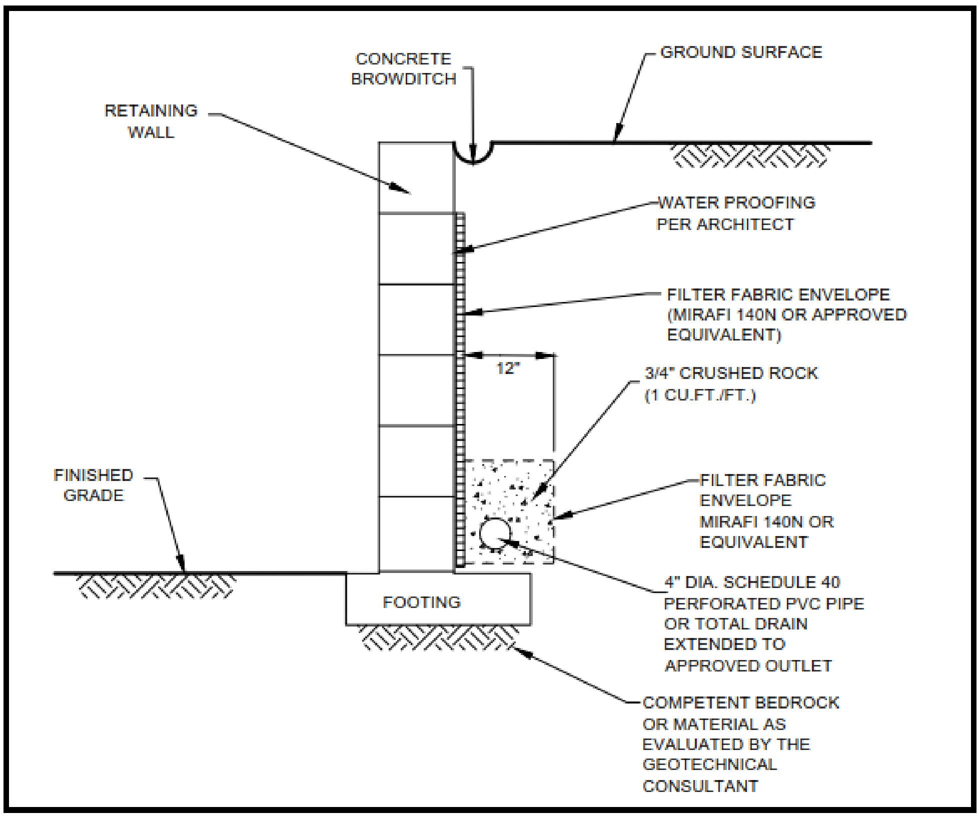


Figure 6-2. Conceptual Design for Wall Drainage

6.8 MSE Walls

6.8.1 General

There is currently no planning for use of mechanically stabilized earth (MSE) walls. However, such walls may be efficient for use in development of roadways and building access in areas where the ground is steeply sloping. This subsection provides guidance for design of such walls.

6.8.2 Backfill

Backfill should be comprised of a select granular soil that meets the parameters listed below:

- at least 40% of the material is less than ¼-inch in size,
- maximum particle size of 4 inches, and
- expansion index (EI) less than 30 (EI < 30, after ASTM D4829).

All fill/backfill placed as part of the MSE retaining wall system should be compacted to at least 90% relative compaction determined in accordance with ASTM D1557.

Table 6-6 provides geotechnical parameters for the design of the MSE retaining walls. NOVA expects that a variety of select granular soils will meet these parameters.

Table 6-6. Soil Strength Parameters for MSE Retaining Walls

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Internal Friction Angle, ϕ'	34	34	34
Cohesion, psf	0	0	0
Wet Unit Weight, pcf	115	118	118

6.8.3 Limits of Backfill

Imported select materials should be utilized for the construction of the MSE retaining wall for the foundation area, areas to be reinforced, and for areas to be retained as indicated on Figure 6-3 (following page).

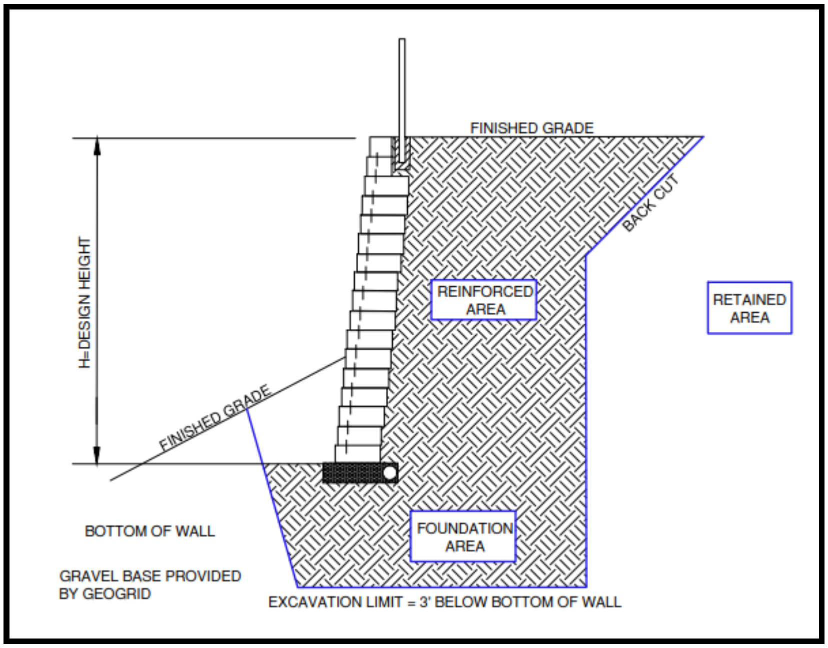


Figure 6-3. Areas of Select Granular Backfill for MSE Walls

As may be seen by review of Figure 6-3, select granular backfill should extend below the foundation of the planned wall a minimum of two feet below the foundation and areas to be reinforced (except where the wall is founded on sound tonalite). The retained area extends backward from the top of wall an equivalent distance to the height of the wall.

6.8.4 Design Review

The plans for the MSE retaining walls should be submitted to NOVA to verify that the design parameters included herein are incorporated and reflected on the project plans.

It should be noted that such review is not intended as participation in the wall design. That design will remain the responsibility of the specialty engineer/constructor who designs the wall. The intent of NOVA's review will be to verify that the wall designer has adequately utilized the design parameters provided herein.

6.9 Permanent Soil Nail Wall

6.9.1 General

As is evident by review of sections through the building that are provided on Plate 2, retained excavations could be up to 35 feet in height. Among alternatives for design to adapt to this condition may be use of a permanent soil nail wall. Such a wall could be used to both isolate the structure seismically and retain the cuts created by excavation to the garage floor slab grade.

6.9.2 Responsibility

If employed, a soil nail wall should be designed by a qualified shoring engineer. The shoring engineer should be solely responsible for the design, utilizing the indications of subsurface conditions provided in the geotechnical reporting.

The following subsection provides geotechnical design parameters for use in development of a soil nail wall.

6.9.3 Geotechnical Design Parameters

Soil nails will be embedded in the Unit 2 weathered tonalite. As is discussed in Section 3.4 and Section 4.2, this unit increases in strength and consistency with increasing depth. Design for soil nails embedded in weathered tonalite should consider two conditions for soil nail resistance, as described below.

- the upper approximately 2 to 10 feet of this unit is locally weathered to the consistency of the very dense silty sand; and
- the tonalite below this level is less weathered, with zones of sound rock.

With the above perspective, Table 6-7 summarizes the strength parameters and expected ultimate bond stresses for this unit.

Table 6-7. Soil Nail Wall Geotechnical Design Parameters

Element of the Unit 2 Tonalite	Soil Design Parameters			Expected Ultimate Bond Stress (psi)*
	Unit Weight (lb/ft3)	Friction Angle (degrees)	Cohesion (psf)	
Upper 10 Feet	125	34	500	20
10 Feet – 25 Feet	130	38	2,500	40

* Expected bond stress is an estimate, assuming ‘post-grouted’ construction. The soil nail designer should make an independent evaluation in order to verify the preliminary bond stress cited in this table. It is the contractor’s sole responsibility to obtain the required pullout capacity. Bond stress capacity is affected by soil/rock condition, method of construction, and grouting techniques.

7. PAVEMENTS

7.1 General

The structural design of pavement sections depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. For the purposes of the evaluation provided in this section, NOVA has assumed a Traffic Index (TI) of 5.0 for passenger car parking, 6.0 for the driveways, and 7.0 for fire lanes. These traffic indices should be confirmed by the civil engineer prior to final design.

7.2 Drainage

Control of surface drainage is important to the design and construction of pavements. Standing water that develops either on the pavement surface or within the base course can soften the subgrade and create other problems related to the deterioration of the pavement. Good drainage should minimize the risk of the subgrade materials becoming saturated and weakened over a long period of time.

The following recommendations should be considered to limit the amount of excess moisture, which can reach the subgrade soils:

- maintain surface gradients at a minimum 2% grade away from the pavements;
- compact utility trenches for landscaped areas to the same criteria as the pavement subgrade;
- seal all landscaped areas in or adjacent to pavements to minimize or prevent moisture migration to subgrade soils;
- planters should not be located next to pavements (otherwise, subdrains should be used to drain the planter to appropriate outlets);
- place compacted backfill against the exterior side of curb and gutter; and,
- concrete curbs bordering landscaped areas should have a deepened edge to provide a cutoff for moisture flow beneath pavements (generally, the edge of the curb can be extended an additional 12 inches below the base of the curb).

7.3 Preventive Maintenance

Preventative maintenance should be planned and provided for in the ownership of all pavements. Preventative maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Preventative maintenance consists of both localized maintenance (e.g. crack sealing and patching) and global maintenance (e.g. surface sealing). Preventative maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements.

7.4 Subgrade Preparation

7.4.1 General

Subgrades for new pavements, including areas extending to 2 feet outside the limits of new pavements, should be prepared as described below.

1. *Step 1, Excavate and Stage.* The upper 12 inches of soil below the base course level as well as any undocumented fill and colluvium should be removed and replaced with material that meets the requirements for Select Fill (Section 6.4.4).
2. *Step 2, Scarify/Moisture Condition.* The surface exposed by Step 1 and disturbed by excavation should be scarified to a depth of 6 inches, moisture conditioned to above the optimum moisture, then densified to at least 95% relative compaction after ASTM D1557.
3. *Step 3, Replacement.* Soils excavated by Step 1 should be replaced as Select Fill (Section 6.4.3).
4. *Step 4, Proof Rolling.* After the completion of compaction/densification, areas to receive pavements should be proof-rolled. A loaded dump truck or similar should be used to aid in identifying localized soft or unsuitable material. Any soft or unsuitable materials encountered during this proof-rolling should be removed, replaced with an approved backfill, and compacted.

7.4.2 Timely Pavement Construction

Construction should be managed such that preparation of the subgrade immediately precedes placement of the base course. Proper drainage of the paved areas should be provided to reduce moisture infiltration to the subgrade.

7.4.3 Surveillance

The preparation of roadway and parking area subgrades should be observed on a full-time basis by a representative of NOVA to confirm that any unsuitable materials have been removed and that the subgrade is suitable for support of the proposed driveways and parking areas after ASTM D1557.

7.5 Flexible Pavements

A laboratory-verified R-value of 44 was used for preliminary design of pavement sections. The actual R-value of the subgrade soils should be determined after grading, and the final pavement sections should be provided. Utilizing an R-value of 44, the preliminary pavement structural sections were determined for the assumed Traffic Indexes on Table 7-1 (following page).

Table 7-1. Preliminary Recommendations for Flexible Pavements

Area	Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Parking Stalls	44	5.0	3.0	5.0
Auto Driveways	44	6.0	4.0	5.0
Fire Lanes	44	7.0	4.0	6.0

The above sections assume a subgrade prepared as described above. The aggregate base materials should also be placed at a minimum relative compaction of 95%. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction (Green Book).

Note that the recommended pavement sections are for planning purposes only. Additional R-value testing should be performed on actual soils at the design subgrade levels to confirm the pavement design.

7.6 Rigid Pavements

The flexible pavement specifications used in driveways and parking stalls may not be adequate for truck loading and turnaround areas. In this event, NOVA recommends that a rigid concrete pavement section be provided.

The rigid pavement section should consist of 7 inches of concrete over a 6-inch base course. The aggregate base materials should also be placed at a minimum relative compaction of 95%. The concrete should be obtained from a mix design that conforms with the minimum properties shown on Table 7-2.

Table 7-2. Recommendations for Concrete Pavements

Property	Recommended Requirement
Compressive Strength @ 28 days	3,250 psi minimum
Strength Requirements	ASTM C94
Minimum Cement Content	5.5 sacks/cu. yd.
Cement Type	Type V Portland
Concrete Aggregate	ASTM C33
Aggregate Size	1-inch maximum
Maximum Water Content	0.5 lb/lb of cement
Maximum Allowable Slump	4 inches

Longitudinal and transverse joints should be provided as needed in concrete pavements for expansion/ contraction and isolation. Sawed joints should be cut within 24-hours of concrete placement and should be a minimum of 25% of slab thickness plus ¼-inch. All joints should be sealed to prevent entry of foreign material and doweled where necessary for load transfer. Where dowels cannot be used at joints accessible to wheel loads, pavement thickness should be increased by 25% at the joints and tapered to regular thickness in 5 feet.

8. STORMWATER INFILTRATION FEASIBILITY

8.1 Overview

Based upon the indications of the field exploration and laboratory testing reported herein, NOVA has evaluated the site as abstracted below after guidance contained in the City of Vista BMP Design Manual (hereafter, 'the BMP Manual').

Section 3.3 provides a description of the fieldwork undertaken to complete the testing. Figure 3-1 depicts the location of the testing. This section provides the results of that testing and related recommendations for management of stormwater in conformance with the BMP Manual.

As is well-established in the BMP Manual, the feasibility of stormwater infiltration is principally dependent on geotechnical conditions at the project site. In consideration of geotechnical hazards associated with permanent infiltration BMPs at this site, NOVA concludes that the site is only suitable for partial stormwater infiltration in the areas of P-5 and P-6 (Plate 1). In the other areas, the rates were too low for reliable infiltration. This section provides NOVA's assessment of the feasibility of stormwater infiltration BMPs utilizing the information developed by the field exploration described in Section 3, as well as other elements of the site assessment.

8.2 Infiltration Rate

The percolation rate of a soil profile is not the same as its infiltration rate ('I'). Therefore, the measured/calculated field percolation rate was converted to an estimated infiltration rate utilizing the Porchet Method in accordance with guidance contained in the BMP Manual. Table 8-1 provides a summary of the infiltration rates determined by the percolation testing.

Table 8-1. Infiltration Rate Determined by Percolation Testing

Boring Reference	Approximate Elevation (feet, msl)	Total Depth (feet)	Approximate Percolation Test Elevation (feet, msl)	Subsurface Unit Tested[1]	Infiltration Rate (in/hr)	Infiltration Rate (in/hr, FS=2)[2]
P-1	+354	5	+349	Qcol	0.06	0.03
P-2	+355	6	+349	Qcol	0.10	0.05
P-3	+364	6	+358	Kt	0.12	0.06
P-4	+365	5	+360	Kt	0.06	0.03
P-5	+369	6	+363	Qcol	2.99	1.49
P-6	+370	5.5	+364.5	Qcol	2.32	1.16
P-7	+362	5	+357	Kt	0.12	0.06
P-8	+362	5	+357	Kt	0.69	0.34
P-9	+371	5	+365.5	Qcol	0.16	0.08
P-10	+371	5	+365.5	Qcol	0.38	0.19

Note 1: The referenced subsurface units tested colluvium (Qcol), and Cretaceous tonalite (Kt)
Note 2: 'FS' indicates 'Factor of Safety'.

As may be seen by review of Table 8-1, in consideration of the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time, the calculated infiltration rates were modified using a factor of safety (F) of F=2 for preliminary design purposes.

8.3 Review of Geotechnical Feasibility Criteria

8.3.1 Overview

Section C.2 of Appendix C of the BMP Manual provides seven factors that should be considered by the project geotechnical professional while assessing the feasibility of infiltration related to geotechnical conditions. These factors are listed below.

- C.2.1 Soil and Geologic Conditions
- C.2.2 Settlement and Volume Change
- C.2.3 Slope Stability
- C.2.4 Utility Considerations
- C.2.5 Groundwater Mounding
- C.2.6 Retaining Walls and Foundations
- C.2.7 Other Factors

The above geotechnical feasibility criteria are reviewed in the following subsections.

8.3.2 Soil and Geologic Conditions

The sequence of subsurface materials encountered by the borings may be generalized to occur as described below.

- Artificial Fill (Afu). The undocumented artificial fill encountered at the site is generally found in three localized zones, which are presented on Plate 1. Fill in the two eastern areas are less than 3 feet in thickness, observed to be grayish-brown silty sand with gravel of loose consistency. Some construction debris was observed within the fill. This undocumented soil is at risk for wide variations in quality and consistency.

- Quaternary Colluvium (Qcol). The colluvium was encountered either below the fill or from the surface in all borings except TP-21. This unit was generally observed to be olive to reddish-brown silty sand and brown to dark brown clayey sand and sandy clay of loose/soft to medium dense in consistency/relative density. The unit was generally porous with some gravel and cobbles encountered. The colluvium is depicted on Figure

- Weathered Tonalite (Kt). The upper several feet of the Cretaceous-aged weathered tonalite, decomposed to the consistency of a medium to coarse, light brown to gray silty sand. This residual soil is derived from in-place weathering of the parent tonalite. The degree of weathering diminishes with depth, such that this unit may include zones of sound, continuous rock, as well as gravelly zones.

8.3.3 Settlement and Volume Change

The calculated infiltration rates within the fill/colluvium are relatively low and are not suitable for settlement-sensitive improvements. As such these soils will be removed/recompacted and are not representative of the reliable infiltration rate at the site.

8.3.4 Slope Stability

The site is characterized by a relatively sloping ground surface. The steepest topographic gradients range from about +360 feet mean sea level (msl) at the eastern periphery of the site to about +430 feet msl at Lado De Loma Drive, a surface gradient of about 20%. BMPs are required to be sited at least 50 feet away from slopes. This site has limited space to achieve that setback.

8.3.5 Utilities

Stormwater infiltration BMPs should not be sited within 10 feet of underground utilities.

8.3.6 Groundwater Mounding

Stormwater infiltration can result in damaging groundwater mounding during wet periods. Though the infiltration rates in the tonalite were variable from low to high, this unit becomes dense with depth. Over time, this unit could create groundwater mounding due to its low permeability.

8.3.7 Retaining Walls and Foundations

Stormwater infiltration BMPs should not be sited within 10 feet from retaining walls and foundations.

8.3.8 Other

Other risk factors include nuisance perched groundwater resulting from water infiltration perching on the very dense tonalite.

8.4 Preliminary Recommendation for Infiltration

In consideration of the foregoing, it is the preliminary judgment of NOVA that the site is not suitable for reliable infiltration of stormwater BMPs in any of the areas tested with the exception of P-5 and P-6.

BMP facilities with no infiltration conditions should be lined throughout with an impermeable geomembrane to reduce the potential for water-related distress to adjacent structures or improvements. A subdrain system should be installed at the bottom of BMP facilities. Additionally, BMP facilities should be kept at least 10 feet from structural foundations.

Appendix E presents Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions.

9. CONSTRUCTION REVIEW, OBSERVATION, AND TESTING

9.1 Overview

As is discussed in Section 1, the recommendations contained in this report are based upon a limited number of borings and an assumption of general continuity of subsurface conditions between borings.

The recommendations provided in both NOVA's proposal for this work and this report assume that NOVA will be retained to provide consultation and review during the design phase, to interpret this report during the construction phase, and to provide construction monitoring in the form of testing and observation.

9.2 Design Phase Review

NOVA should be retained to provide review of final grading and foundation plans. This review is provided for in NOVA's proposal for this work.

9.3 Construction Observation and Testing

9.3.1 Preconstruction Conference

A preconstruction conference among representatives of the owner, contractor and/or construction manager, and Geotechnical Engineer-of-Record (GEOR) is recommended to discuss the planned construction procedures and quality control requirements.

9.3.2 Special Inspections

Special inspections should be provided per Section 1705 of the California Building Code. The soils special inspector should be a representative of NOVA as the GEOR.

NOVA should be retained to provide construction-related services abstracted below.

- Surveillance during site preparation, grading, and foundation excavation.
- Construction of temporary shoring.

A program of quality control should be developed prior to the beginning of construction. It is the responsibility of the owner, the contractor, and/or the construction manager to determine any additional inspection items required by the architect/engineer or the governing jurisdiction.

9.3.3 Continuous Soils Special Inspection

The earthwork operations listed below should be the object of continuous soils special inspection.

- Tieback installation and testing.
- Foundation and mat subgrade preparation/compaction.

9.3.4 Periodic Soils Special Inspection

The earthwork operations listed below should be the object of periodic soils special inspection, subject to approval by the Building Official.

- Site preparation and removal of existing development features.
- Placement and compaction of utility trench backfill.

9.3.5 Testing During Inspections

The locations and frequencies of compaction test should be determined by the geotechnical engineer at the time of construction. Test locations and frequencies may be subject to modification by the geotechnical engineer based upon soil and moisture conditions encountered, the size and type of compaction equipment used by the contractor, the general trend of compaction test results, and other factors.

Of particular concern to NOVA during earthwork operations will be good practices in moisture conditioning, loose soil placement, and soil compaction. In particular, NOVA will be vigilant with regard to the use of compaction equipment appropriate to the full lift thickness of the type of soil being compacted. Reliance on construction traffic (for example, loaders or dump trucks) to achieve compaction will not be approved.

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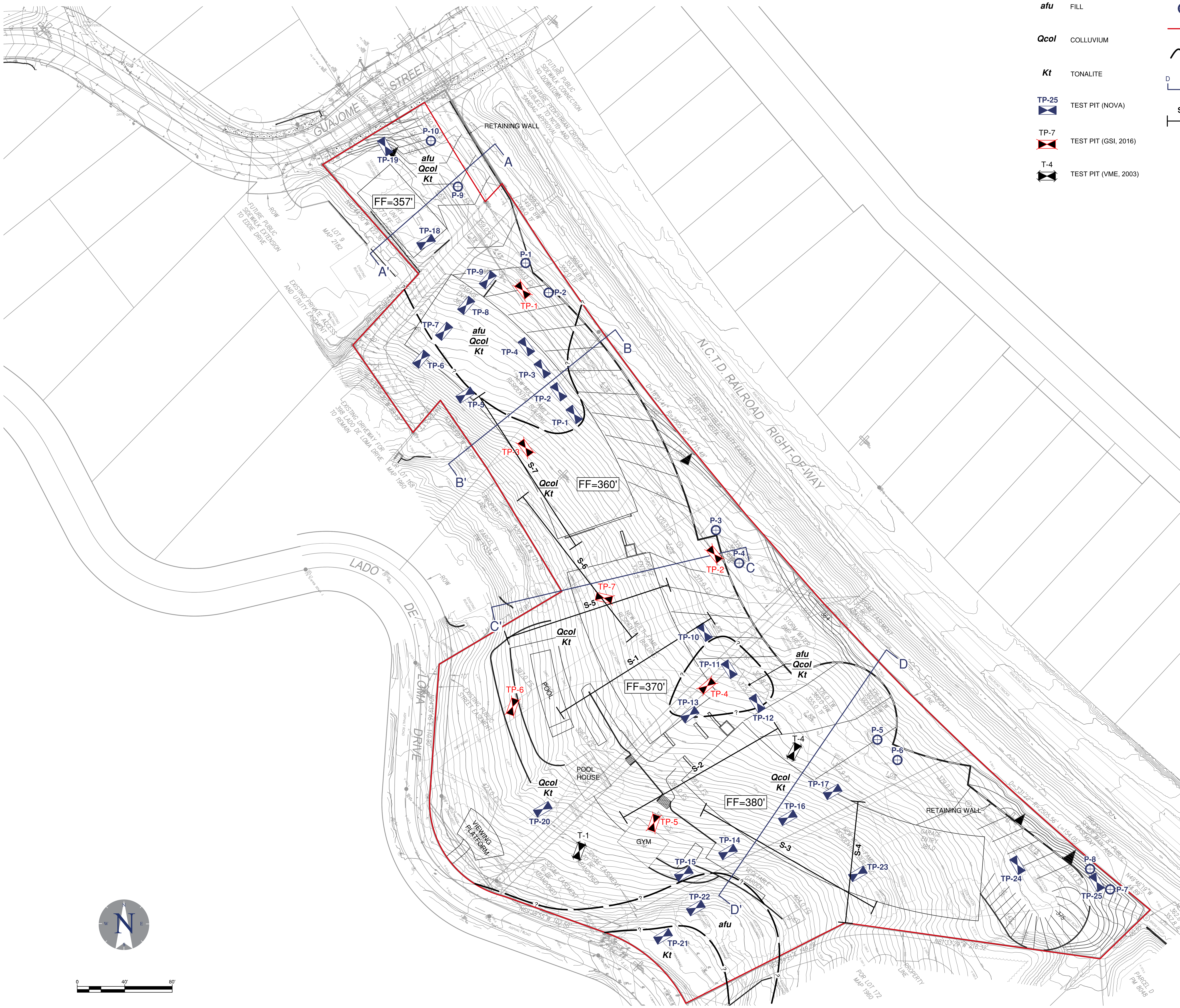
PLATES



KEY TO SYMBOLS

afu	FILL
Qcol	COLLUVIUM
Kt	TONALITE
TP-25	TEST PIT (NOVA)
TP-7	TEST PIT (GSI, 2016)
T-4	TEST PIT (VME, 2003)
P-10	PERCOLATION TEST BORING
	PROPOSED PROPERTY LIMITS
	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN (GSI, 2016)
D D'	GEOLOGIC CROSS-SECTION ALIGNMENT
S-7	SEISMIC LINE

*BASE MAP SOURCE: CONCEPTUAL GRADING PLAN BY PLSA MARCH 4, 2022.

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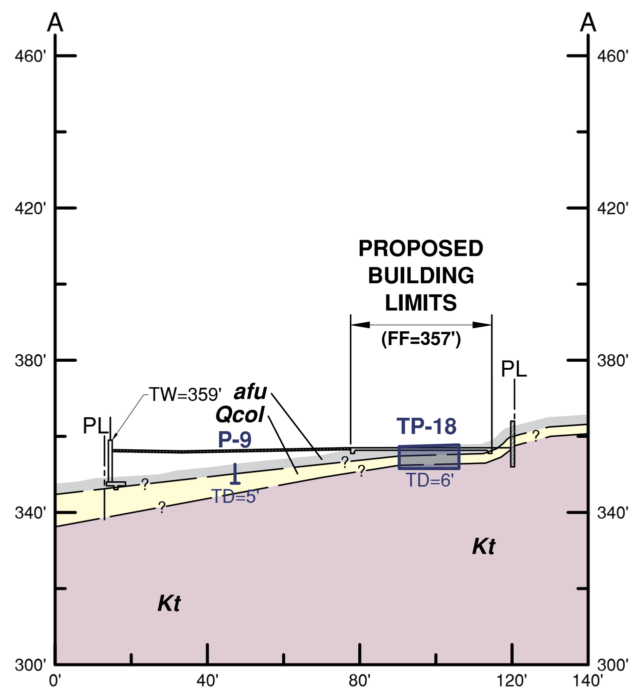
KENSHO HOUSING APARTMENTS

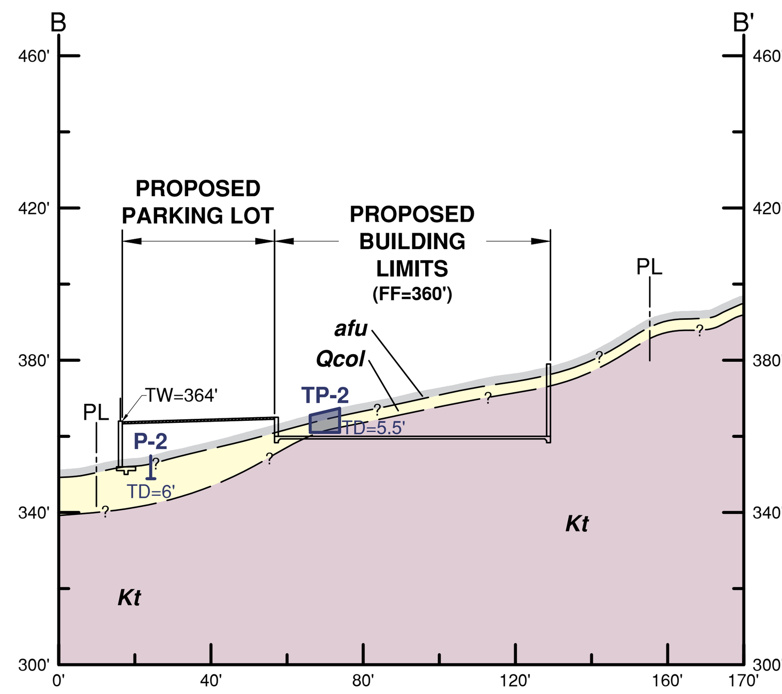
LADO DE LOMA DRIVE, VISTA, CALIFORNIA

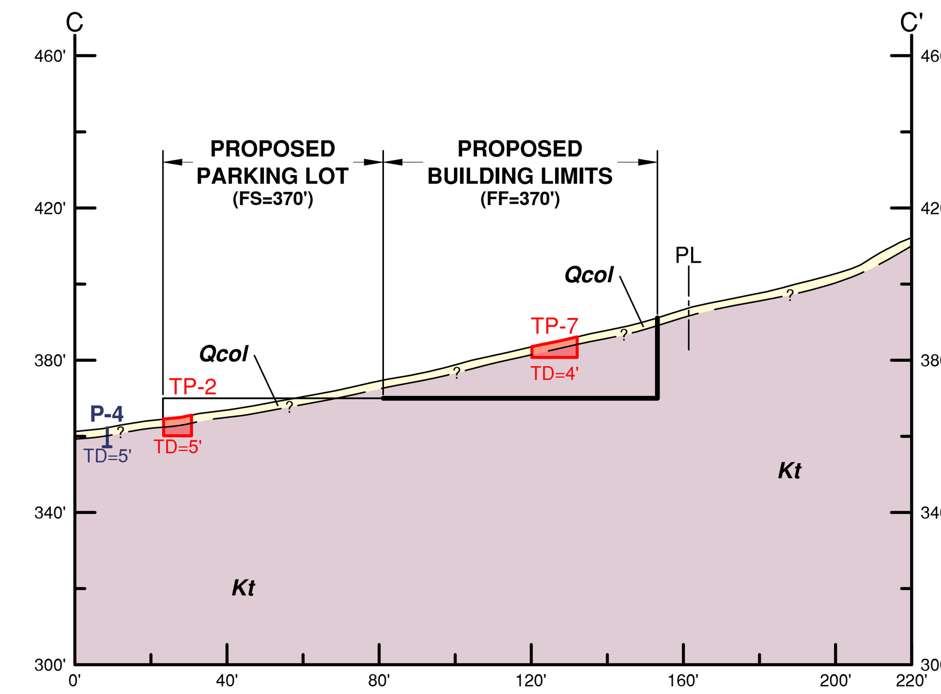
PROJECT NO.:	2021172
DATE:	APRIL 2022
DRAWN BY:	DTJ
REVIEWED BY:	MS
SCALE:	1"=40'
DRAWING TITLE:	

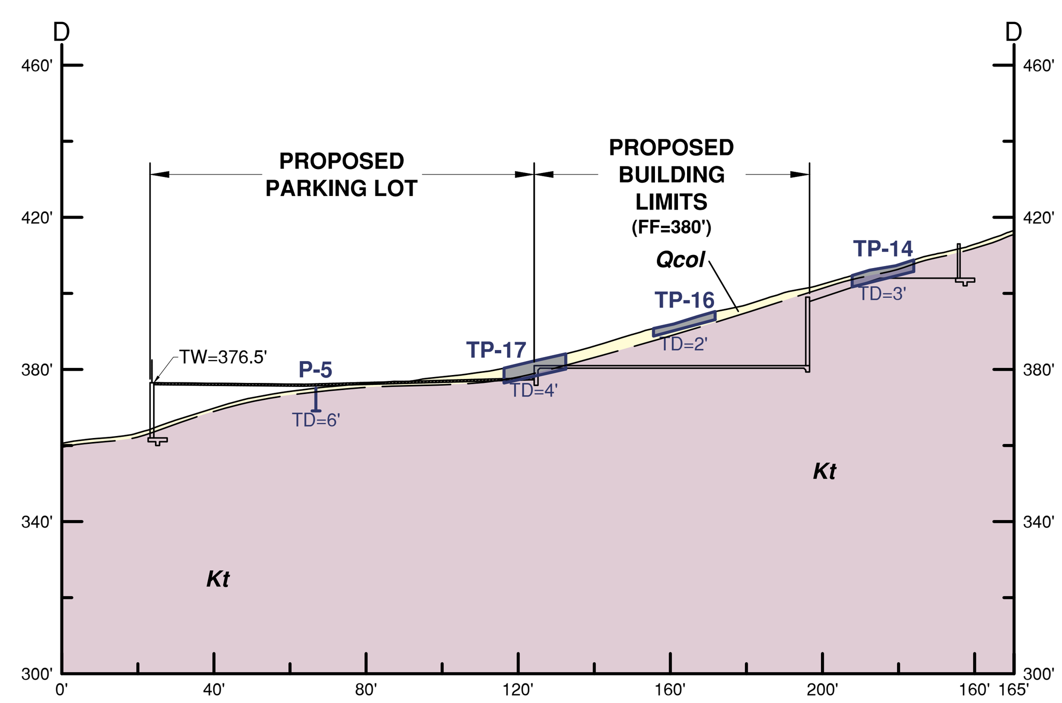
SUBSURFACE INVESTIGATION MAP

PLATE NO. 1 OF 2



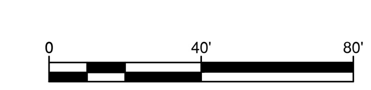






KEY TO SYMBOLS

afu	FILL
Qcol	COLLUVIUM
Kt	TONALITE
TP-17	TEST PIT (NOVA)
TP-7	TEST PIT (GSI, 2016)
P-9 TD=5'	PERCOLATION TEST BORING
	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN



*BASE MAP SOURCE: CONCEPTUAL GRADING PLAN BY PLSA MARCH 4, 2022.

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SCALE:	1"=40'
DRAWING TITLE:	

GEOLOGIC CROSS-SECTIONS
AA', BB', CC', AND DD'

APPENDIX A
USE OF THE GEOTECHNICAL REPORT

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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IIGER06045.0M

APPENDIX B

LOGS OF TEST PITS AND PERCOLATION BORINGS

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVEL MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVEL WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVEL WITH OR WITHOUT SAND
			GP	POORLY GRADED GRAVEL WITH OR WITHOUT SAND
		GRAVEL WITH 15% OR MORE FINES	GM	SILTY GRAVEL WITH OR WITHOUT SAND
			GC	CLAYEY GRAVEL WITH OR WITHOUT SAND
	SAND MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 15% FINES	SW	WELL-GRADED SAND WITH OR WITHOUT GRAVEL
			SP	POORLY GRADED SAND WITH OR WITHOUT GRAVEL
		SAND WITH 15% OR MORE FINES	SM	SILTY SAND WITH OR WITHOUT GRAVEL
			SC	CLAYEY SAND WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	SILT WITH OR WITHOUT SAND OR GRAVEL
			CL	LEAN CLAY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILT OR CLAY OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	ELASTIC SILT WITH OR WITHOUT SAND OR GRAVEL
			CH	FAT CLAY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILT OR CLAY OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GROUNDWATER / STABILIZED

PERCHED GROUNDWATER

BULK SAMPLE

SPT SAMPLE (ASTM D1586)

MOD. CAL. SAMPLE (ASTM D3550)

* NO SAMPLE RECOVERY

GEOLOGIC CONTACT

SOIL TYPE CHANGE

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	SPT N60 BLOWS/FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	SPT N60 BLOWS/FOOT	POCKET PENETROMETER MEASUREMENT (TSF)
VERY SOFT	0 - 2	0 - 0.25
SOFT	2 - 4	0.25 - 0.50
MEDIUM STIFF	4 - 8	0.50 - 1.0
STIFF	8 - 15	1.0 - 2.0
VERY STIFF	15 - 30	2.0 - 4.0
HARD	OVER 30	OVER 4.0

NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).
IF THE SEATING INTERVAL (1st 6 INCH INTERVAL) IS NOT ACHEIVED, N IS REPORTED AS REF.

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SUBSURFACE EXPLORATION LEGEND

LOG OF TEST PIT TP-1

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 380 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	X		SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO COARSE GRAINED, POROUS, SOME LARGE GRAVEL, COBBLE, CONSTRUCTION DEBRIS*		
	X		SM		**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO COARSE GRAINED*		
5					*TEST PIT TERMINATED AT 4.5 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.1

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.:2021172

LOG OF TEST PIT TP-2

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR	
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A	
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 365 MSL	

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO COARSE GRAINED, POROUS, ABUNDANT LARGE GRAVEL, COBBLE, CONSTRUCTION DEBRIS, SOME ASPHALT*		
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
5			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 5.5 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.2

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-3

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 366 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO COARSE GRAINED, POROUS, ABUNDANT LARGE GRAVEL, CONSTRUCTION DEBRIS*		
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
5			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 5 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.3

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-4

DATE EXCAVATED: AUGUST 10, 2021 **EQUIPMENT:** 303.5CR

EXCAVATION DESCRIPTION: TEST PIT **GPS COORD.:** N/A

GROUNDWATER DEPTH: NOT ENCOUNTERED **ELEVATION:** ± 367 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO COARSE GRAINED, POROUS, ABUNDANT LARGE GRAVEL, SOME ASPHALT DEBRIS*		
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
5			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 5 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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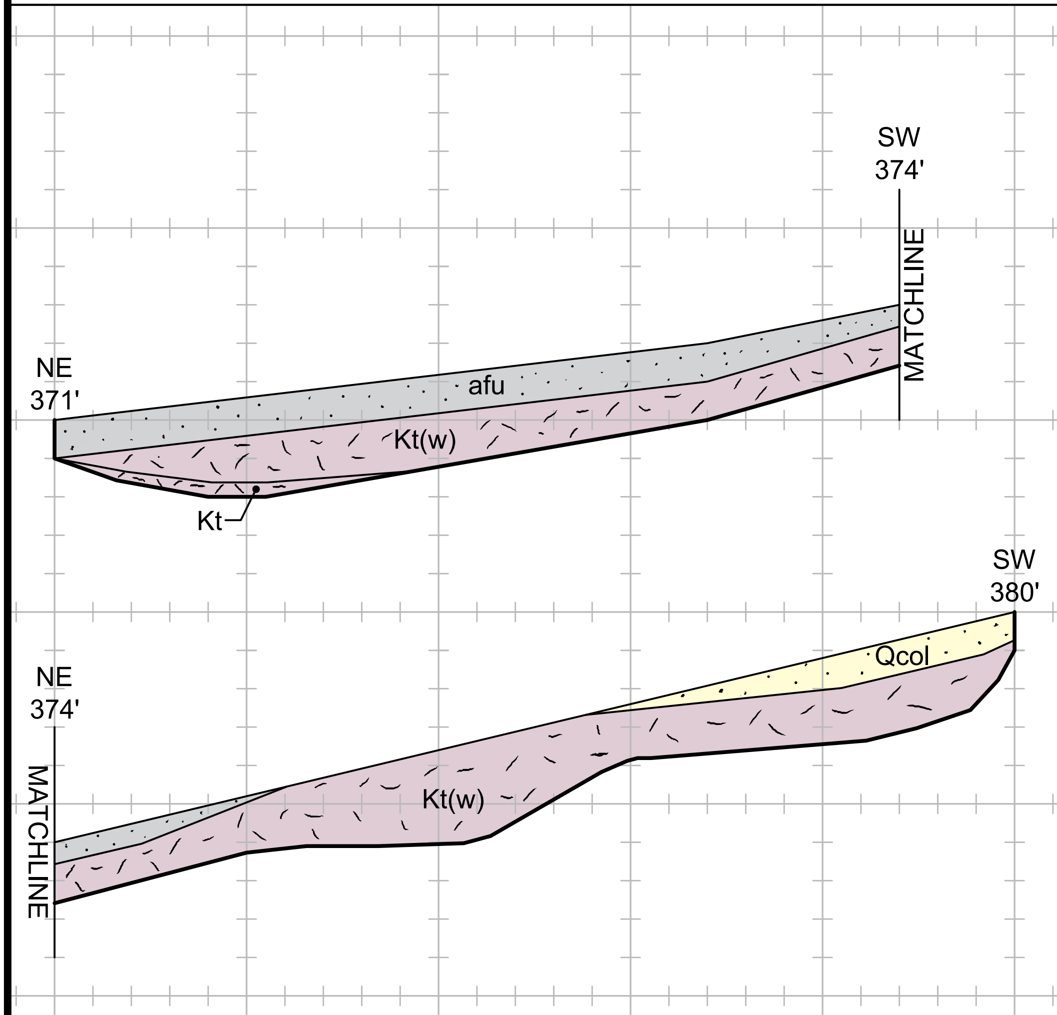
PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.4

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

PROJECT NAME: Pheasant Hill Site Development	**DATE:** 08-10-2021	**TEST PIT NO.:** TP-5	**SURFACE SLOPE:** -
PROJECT NO.: 2021172	**LOGGED BY:** GN	**ELEVATION:** 371-380	**GROUNDWATER:** NO
EQUIPMENT: 303.5CR	**APPROVED BY:** MS	**TREND:** NE-SW	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:		USCS	LAB
0'-1'	**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO COARSE GRAINED, POROUS , SOME CONSTRUCTION DEBRIS,*	SM	
0'-1'	**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, DRY, DENSE, FINE TO COARSE GRAINED, POROUS*	SM	0-1
1'-2.5'	**TONALITE (Kt): (WEATHERED)** *SILTY SAND; ORANGE BROWN, DRY, DENSE, MEDIUM TO COARSE GRAINED*	SM	
2.5'-3'	**TONALITE (Kt):** *SILTY SAND; REDDISH BROWN, DRY, VERY DENSE, MEDIUM TO COARSE GRAINED*	SM	2-3
3'	*PRACTICAL REFUSAL @3FT ON TONALITE*		

TOTAL DEPTH: 3'

BACKFILLED: Y

COMPACTED: LIGHT

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LOG OF TEST PIT TP-6

DATE EXCAVATED: AUGUST 10, 2021 **EQUIPMENT:** 303.5CR

EXCAVATION DESCRIPTION: TEST PIT **GPS COORD.:** N/A

GROUNDWATER DEPTH: NOT ENCOUNTERED **ELEVATION:** ± 375 MSL TO ± 376 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE GRAINED*		
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; OLIVE BROWN WITH REDDISH BROWN STAINING, DRY, DENSE, MEDIUM TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 2 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.6

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-7

DATE EXCAVATED: AUGUST 10, 2021 **EQUIPMENT:** 303.5CR

EXCAVATION DESCRIPTION: TEST PIT **GPS COORD.:** N/A

GROUNDWATER DEPTH: NOT ENCOUNTERED **ELEVATION:** ± 370 MSL TO ± 371 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO MEDIUM GRAINED, SOME GRAVEL*		
	X		SM		**COLLUVIUM (Qcol):** *SILTY SAND; OLIVE BROWN, DRY, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; OLIVE BROWN, DRY, VERY DENSE, MEDIUM TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 2.5 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.7

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-8

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 362 MSL TO ± 364 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, MEDIUM DENSE, FINE TO COARSE GRAINED, ABUNDANT LARGE GRAVEL* *LENSE OF LARGE WHITE GRAVEL AND COBBLE*		
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, DRY, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; OLIVE BROWN, DRY, DENSE, MEDIUM TO COARSE GRAINED*		
5					*TEST PIT TERMINATED AT 4.5 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.8

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.:2021172

LOG OF TEST PIT TP-9

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 357 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, MEDIUM DENSE, FINE TO COARSE GRAINED, ABUNDANT GRAVEL* *LENSE OF LARGE WHITE GRAVEL AND COBBLE*		
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, DRY, MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
	☒		SC		*CLAYEY SAND; REDDISH BROWN, MOIST, DENSE, POROUS*	AL EI SA	*29 LOW*
					TEST PIT TERMINATED AT 4 FT DUE TO PRACTICAL REFUSAL. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.9

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-10

DATE EXCAVATED: AUGUST 10, 2021
EQUIPMENT: 303.5CR
EXCAVATION DESCRIPTION: TEST PIT
GPS COORD.: N/A
GROUNDWATER DEPTH: NOT ENCOUNTERED
ELEVATION: ± 375 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE, FINE TO COARSE GRAINED, POROUS*		
	X		CL		*SANDY CLAY; DARK BROWN, MOIST, SOFT, ABUNDANT LARGE GRAVEL, SLIGHTLY POROUS*	CR AL SA	
	X		SM		*SILTY SAND; REDDISH BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, POROUS*		
5			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND, DARK REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 6 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.10

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-11

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 375 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	X		SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, VERY DENSE, FINE TO COARSE GRAINED, ABUNDANT GRAVEL, 3" LAYER OF DECOMPOSED GRANITE*		
			SC/CL		**COLLUVIUM (Qcol):** *CLAYEY SAND/SANDY CLAY; DARK BROWN, MOIST, LOOSE/SOFT, FINE TO COARSE GRAINED, ABUNDANT GRAVEL, SLIGHTLY POROUS*		
			SM		*SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED, POROUS*		
5							
					TEST PIT TERMINATED AT 6 FT DUE TO PRACTICAL REFUSAL. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.11

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.:2021172

LOG OF TEST PIT TP-12

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 375 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, VERY DENSE , FINE TO COARSE GRAINED, ABUNDANT GRAVEL, 3 INCH LAYER OF CLASS II BASE*		
	X		SC/CL		**COLLUVIUM (Qcol):** *CLAYEY SAND/SANDY CLAY; DARK BROWN, MOIST, LOOSE/SOFT, FINE TO COURSE GRAINED, SLIGHTLY POROUS*		
5	X		SM		*SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED, SOME GRAVEL, POROUS*	MD SA	
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; OLIVE BROWN, DRY, VERY DENSE, MEDIUM TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 8 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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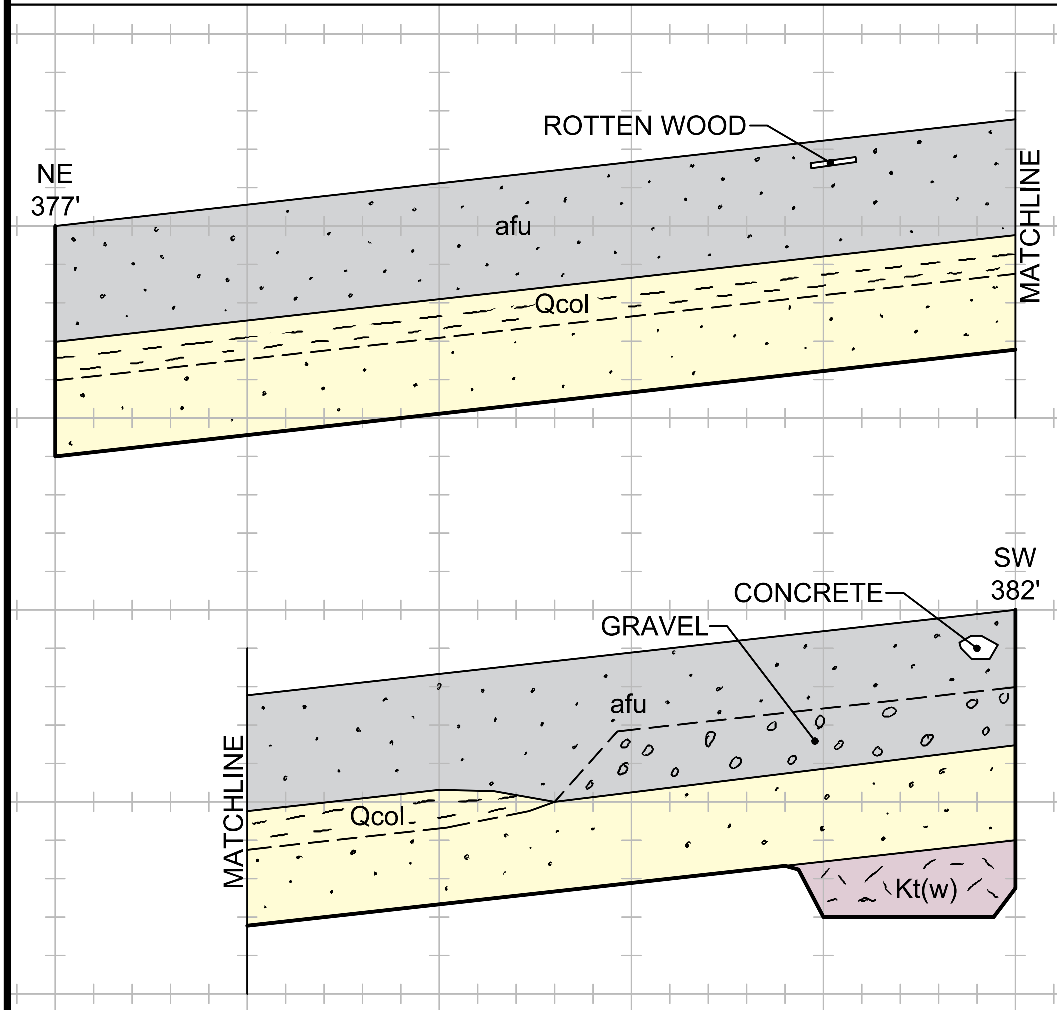
PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.12

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

PROJECT NAME: Pheasant Hill Site Development	DATE: 08-10-2021	TEST PIT NO.: TP-13	SURFACE SLOPE: -
PROJECT NO.: 2021172	LOGGED BY: GN	ELEVATION: 377-382	GROUNDWATER: NO
EQUIPMENT: 303.5CR	APPROVED BY: -	TREND: NE-SW	SCALE: 1"=5'

GRAPHICAL REPRESENTATION BELOW:



TOTAL DEPTH: 6-7

BACKFILLED: Y

COMPACTED: LIGHT

SOIL DESCRIPTION:		USCS	LAB
0'-3'	**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, SCATTERED CONCRETE DEBRIS, ROTTEN WOOD*	SM	0-3
3'-4'	**COLLUVIUM (Qcol):** *CLAYEY SAND/SANDY CLAY; DARK BROWN, MOIST, LOSE/SOFT, FINE TO COARSE GRAINED, SLIGHTLY POROUS*	SC/CL	3-4
4'-6'	*SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED,*	SM	
6'	*PRACTICAL REFUSAL @6FT ON TONALITE*		
0'-2'	**FILL (afu):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, SCATTERED CONCRETE DEBRIS, ROTTEN WOOD*	SM	
2'-4'	*FREQUENT GRAVEL*		
4'-6'	**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED*	SM	
6'-7'	**TONALITE (Kt) (WEATHERED):** *SILTY SAND; LIGHT BROWN, DRY, VERY DENSE, MEDIUM TO COARSE GRAINED*	SM	
7'	*PRACTICAL REFUSAL @7FT ON TONALITE*		

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LOG OF TEST PIT TP-14

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 403 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	X		SM		**COLLUVIUM (Qcol):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO COURSE GRAINED, SCATTERED CALICHE, POROUS*		
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; GRAYISH BROWN, DRY TO SLIGHTLY MOIST, DENSE, FINE TO COURSE GRAINED*		
			SM		**TONALITE (Kt):** *SILTY SAND; ORANGE BROWN, DRY TO SLIGHTLY MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 3 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
10							
15							

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LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.14

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-15

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 412 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, POROUS*		
	X		SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; GRAYISH BROWN, MOIST, DENSE, MEDIUM TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 3 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
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FIGURE B.15

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-16

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 366 MSL TO ± 387 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	X		SM		**COLLUVIUM (Qcol):** *SILTY SAND; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, POROUS* *BROWN, MOIST*		
					TEST PIT TERMINATED AT 2 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
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FIGURE B.16

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-17

DATE EXCAVATED:	AUGUST 10, 2021	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 380 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	X		SM		**COLLUVIUM (Qcol):** *SILTY SAND; GRAYISH BROWN, DRY, MEDIUM DENSE TO DENSE, FINE TO MEDIUM GRAINED, POROUS*		
	X		SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; OLIVE BROWN, MOIST, DENSE, FINE TO COARSE GRAINED*		
5					*TEST PIT TERMINATED AT 4 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10							
15							

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FIGURE B.17

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-18

DATE EXCAVATED: JANUARY 12, 2022
EQUIPMENT: 303.5CR
EXCAVATION DESCRIPTION: TEST PIT
GPS COORD.: N/A
GROUNDWATER DEPTH: NOT ENCOUNTERED
ELEVATION: ± 358 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0	X		SM		**FILL (afu):** *SILTY SAND; DARK BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
	X		SC		*CLAYEY SAND; LIGHT BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED ROOTLETS*	CR MD EI AL SA	*10 VERY LOW*
5			SM		**TONALITE (Kt):** *SILTY SAND; ORANGE BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 6 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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FIGURE B.18

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-19

DATE EXCAVATED:	JANUARY 12, 2022	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 352 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0	X		SM		**FILL (afu):** *SILTY SAND; DARK BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, ABUNDANT ROOTS*	MD SA RV	
			SM		**COLLUVIUM (Qcol):** *SILTY SAND; LIGHT BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED COARSE GRAINS*		
5			SM		**TONALITE (Kt):** *SILTY SAND; ORANGE BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 6 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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FIGURE B.19

LOGGED BY: AR	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-20

DATE EXCAVATED: JANUARY 12, 2022
EQUIPMENT: 303.5CR
EXCAVATION DESCRIPTION: TEST PIT
GPS COORD.: N/A
GROUNDWATER DEPTH: NOT ENCOUNTERED
ELEVATION: ± 408 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; DARK RED BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED*		
	X		SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; YELLOW BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
			SM		**TONALITE (Kt):** *SILTY SAND; YELLOW BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
5					*TEST PIT TERMINATED AT 4 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10							
15							

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FIGURE B.20

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-21

DATE EXCAVATED:	JANUARY 12, 2022	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 428 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0	X		SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; ORANGE BROWN, MOIST, DENSE, FINE TO COARSE GRAINED, TRACE ROOTS*	RV	
	X		SM		**TONALITE (Kt):** *SILTY SAND; ORANGE BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 3 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
10							
15							

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FIGURE B.21

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-22

DATE EXCAVATED:	JANUARY 12, 2022	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 420 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0	X (0–5)		SM		**FILL (afu):** *SILTY SAND; DARK BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, ABUNDANT REFUSE AND DEBRIS (CONCRETE, BRICK, PIPE, TRASH BAG)*	CR SA	
5					*DARK RED BROWN*		
8					*TEST PIT TERMINATED AT 8 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10							
15							

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FIGURE B.22

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-23

DATE EXCAVATED:	JANUARY 12, 2022	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 395 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; DARK RED BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED, SOME ROOTS*		
	X		SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; ORANGE BROWN, MOIST, DENSE, FINE TO COARSE GRAINED* *VERY DENSE*	MD DS	
					TEST PIT TERMINATED AT 4 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5							
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FIGURE B.23

LOGGED BY: AR	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF TEST PIT TP-24

DATE EXCAVATED: JANUARY 12, 2022
EQUIPMENT: 303.5CR
EXCAVATION DESCRIPTION: TEST PIT
GPS COORD.: N/A
GROUNDWATER DEPTH: NOT ENCOUNTERED
ELEVATION: ± 368 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; LIGHT BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
			SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; YELLOW BROWN, MOIST, DENSE, FINE TO COARSE GRAINED*		
5			SM		**TONALITE (Kt):** *SILTY SAND; YELLOW BROWN, MOIST, VERY DENSE, FINE TO COARSE GRAINED*		
					TEST PIT TERMINATED AT 4 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10							
15							

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.24

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF TEST PIT TP-25

DATE EXCAVATED:	JANUARY 12, 2022	**EQUIPMENT:**	303.5CR
EXCAVATION DESCRIPTION:	TEST PIT	**GPS COORD.:**	N/A
GROUNDWATER DEPTH:	NOT ENCOUNTERED	**ELEVATION:**	± 363 MSL

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

DEPTH (FT)	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS* *(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LABORATORY	REMARKS
0			SM		**COLLUVIUM (Qcol):** *SILTY SAND; DARK BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED,*		
	X		SM		**TONALITE (Kt): (WEATHERED)** *SILTY SAND; ORANGE BROWN, MOIST, DENSE, FINE TO COARSE GRAINED* *VERY DENSE*	SA	
5					*TEST PIT TERMINATED AT 4 FT DUE TO PRACTICAL REFUSAL ON VERY DENSE TONALITE. NO GROUNDWATER ENCOUNTERED. NO CAVING.*		
10							
15							

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LADO DE LOMA DRIVE
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FIGURE B.25

LOGGED BY: AR	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-1

DATE DRILLED: AUGUST 10, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 355 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							SM	**FILL (afu):** *SILTY SAND WITH GRAVEL; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SM	**COLLUVIUM (Qcol):** *SILTY SAND; BROWN, DRY, MEDIUM DENSE, FINE TO MEDIUM GRAINED, ABUNDANT GRAVEL, LOOSE*	
	X						CL	*SANDY CLAY; BROWN, MOIST, STIFF*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
20									
25									
30									

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LADO DE LOMA DRIVE
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FIGURE B.26

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-2

DATE DRILLED: AUGUST 10, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 355 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**FILL (afu):** *SILTY SAND WITH GRAVEL; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SM	**COLLUVIUM (Qcol):** *SILTY SAND; BROWN, DRY, MEDIUM DENSE, FINE TO MEDIUM GRAINED, ABUNDANT GRAVEL, LOOSE*	
5							CL	*SANDY CLAY; BROWN, MOIST, STIFF*	
								BORING TERMINATED AT 6 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.	
10									
15									
20									
25									
30									

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.27

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-3

DATE DRILLED: AUGUST 10, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 364 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0	X						SM	**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, DRY, DENSE, FINE TO MEDIUM GRAINED*	
5	X						SM	**TONALITE (WEATHERED) (Kt):** *SILTY SAND; OLIVE BROWN, MOIST, VERY DENSE, FINE GRAINED*	
10								*BORING TERMINATED AT 6 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
15									
20									
25									
30									

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FIGURE B.28

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-4

DATE DRILLED: AUGUST 10, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 364 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**COLLUVIUM (Qcol):** *SILTY SAND; REDDISH BROWN, DRY, DENSE, FINE TO MEDIUM GRAINED*	
							SM	**TONALITE (WEATHERED) (Kt):** *SILTY SAND; OLIVE BROWN, MOIST, VERY DENSE, FINE GRAINED*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
20									
25									
30									

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LADO DE LOMA DRIVE
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FIGURE B.29

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-5

DATE DRILLED: AUGUST 10, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 371 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**COLLUVIUM (Qcol):** *SILTY SAND WITH GRAVEL; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SM	**TONALITE (WEATHERED) (Kt):** *SILTY SAND; REDDISH BROWN, DRY, DENSE, FINE GRAINED*	
5	X								
								BORING TERMINATED AT 6 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.	
10									
15									
20									
25									
30									

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VISTA, CALIFORNIA

FIGURE B.30

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-6

DATE DRILLED: AUGUST 10, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 371 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							SM	**COLLUVIUM (Qcol):** *SILTY SAND WITH GRAVEL; GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SM	**TONALITE (WEATHERED) (Kt):** *SILTY SAND; REDDISH BROWN, DRY, DENSE, FINE GRAINED*	
5								*BORING TERMINATED AT 5.5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
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VISTA, CALIFORNIA

FIGURE B.31

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-7

DATE DRILLED: JANUARY 12, 2022 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 362 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**COLLUVIUM (Qcol):** *SILTY SAND; DARK RED BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED*	
							SM	**TONALITE (WEATHERED) (Kt):** *SILTY SAND; YELLOW BROWN, MOIST, DENSE, FINE TO COARSE GRAINED* *VERY DENSE*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
20									
25									
30									

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PROPOSED PHEASANT HILL RESIDENTIAL APARTMENTS
LADO DE LOMA DRIVE
VISTA, CALIFORNIA

FIGURE B.32

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-8

DATE DRILLED: JANUARY 12, 2022 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 362 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**COLLUVIUM (Qcol):** *SILTY SAND; DARK RED BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, SCATTERED ROOTLETS*	
							SM	**TONALITE (WEATHERED) (Kt):** *SILTY SAND; YELLOW BROWN, MOIST, DENSE, FINE TO COARSE GRAINED* *VERY DENSE*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
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25									
30									

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LADO DE LOMA DRIVE
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FIGURE B.33

LOGGED BY: AR | REVIEWED BY: MS | PROJECT NO.: 2021172

LOG OF PERCOLATION BORING P-9

DATE DRILLED: JANUARY 12, 2022 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 352 FT MSL **DRILLING EQUP.:** CAT 303.5 CR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: SOLID STEM AUGER **NOTES:**

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**FILL (afu):** *SILTY SAND; DARK BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SM	**COLLUVIUM (Qcol):** *SILTY SAND; BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SC	*CLAYEY SAND; BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
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VISTA, CALIFORNIA

FIGURE B.34

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LOG OF PERCOLATION BORING P-10

DATE DRILLED: JANUARY 12, 2022
DRILLING METHOD: HOLLOW STEM AUGER
ELEVATION: ± 350 FT MSL
DRILLING EQUP.: CAT 303.5 CR
GROUNDWATER DEPTH: NOT ENCOUNTERED
SAMPLE METHOD: SOLID STEM AUGER
NOTES:

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0							SM	**FILL (afu):** *SILTY SAND; BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
							SM	**COLLUVIUM (Qcol):** *SILTY SAND; LIGHT BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED ROOTLETS*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION TEST WELL. NO GROUNDWATER ENCOUNTERED.*	
10									
15									
20									
25									
30									

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FIGURE B.35

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LOGS OF TEST PITS BY OTHERS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Group Symbols	Typical Names
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

3" 3/4" #4 #10 #40 #200 U.S. Standard Sieve

Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C	Core Sample
S	SPT Sample
B	Bulk Sample
–	Groundwater
Qp	Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

GeoSoils, Inc.

W.O. 7089-A-SC
RINA, LLC
Pheasant Hills, Vista
Logged By: RGC
June 2 & 3, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	±355' MSL	0-3	SM				**UNDOCUMENTED FILL:** SILTY SAND with GRAVEL, dark brown to brown, dry, loose; porous, burrowed, 5 to 10 perfect angular gravels.
		3-4½	SM				**COLLUVIUM:** SILTY SAND, dark brown, moist, loose; porous.
		4½-6	CL				**WEATHERED BEDROCK:** SANDY CLAY, yellowish brown, moist, stiff.
		6-8	SC				**BEDROCK:** Decomposed Granite breaking to CLAYEY SAND upon excavation.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 6-2-2016
TP-2	±363' MSL	0-2½	SM	Bulk @ 1-2			**COLLUVIUM:** SILTY SAND, brown, dry, loose; burrowed, porous, few roots.
		2½-3½	SW/SC				**BEDROCK:** Decomposed Granite breaking to SILTY SAND with CLAY upon excavation, dark brown, moist, medium dense. Slightly weathered.
		3½-5	SW/SM				Decomposed Granite breaking to SILTY SAND upon excavation, yellowish brown, moist, dense.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 6-2-2016

GeoSoils, Inc.

W.O. 7089-A-SC
RINA, LLC
Pheasant Hills, Vista
Logged By: RGC
June 2 & 3, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	±378' MSL	0-2	SM				**COLLUVIUM:** SILTY SAND, grayish brown, dry, loose; few roots, burrowed, porous.
		2-5	SM/SW				**BEDROCK:** Decomposed Granite breaking to SILTY SAND and SAND upon excavation, brown, moist, medium dense.
		5-8	SW				Decomposed Granite breaking to SAND upon excavation, gray brown, slightly moist, dense.
							Total Depth = 8' (Practical Refusal) No Groundwater/Caving Encountered Backfilled 5-2-2016
TP-4	±376' MSL	0-3	SM				**UNDOCUMENTED FILL:** SILTY SAND, brown, dry, loose; fine grained, few roots in upper 6 inches.
		3-6	SM/SC				**COLLUVIUM:** SILTY SAND with CLAY, brown, moist, loose; porous.
		6-8	SW/SM				**BEDROCK:** Decomposed Granite breaking to SILTY SAND and SAND upon excavation, light brown, slightly moist, dense.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 5-2-2016

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June 2 & 3, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	±395' MSL	0-3	SM				**COLLUVIUM:** SILTY SAND, gray brown, dry, loose; porous, burrowed, few roots, becomes moist at 1 foot.
		3-4	SM				**BEDROCK:** Decomposed Granite breaking to SILTY SAND upon excavation, brown, moist, loose; porous.
		4-5	SM				Decomposed Granite breaking to SILTY SAND upon excavation, yellowish brown, moist, dense.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 6-3-2016
TP-6	±405' MSL	0-2½	SM				**COLLUVIUM:** SILTY SAND, dark brown to brown, dry, loose; porous, burrowed, becomes moist at 1 foot.
		2½-4	SM				**BEDROCK:** Decomposed Granite breaking to SILTY SAND upon excavation, brown, moist, loose to medium dense; slightly porous, weathered.
		4-5	SW				Decomposed Granite breaking to SAND with SILT upon excavation, yellowish brown, moist, dense.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 6-3-2016

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Pheasant Hills, Vista
Logged By: RGC
June 2 & 3, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	±385' MSL	0-2½	SM				**COLLUVIUM:** SILTY SAND, gray brown, dry, loose; burrowed, porous, few roots.
		2½-3	SM				**BEDROCK:** Decomposed Granite breaking to SILTY SAND upon excavation, brown, dry, loose to medium dense.
		3-4	SM/SW	Bulk @ 3-4			Decomposed Granite breaking to SILTY SAND and SAND upon excavation, yellowish brown, moist, dense.
							Total Depth = 4' No Groundwater/Caving Encountered Backfilled 6-3-2016

APPENDIX C
RECORDS OF LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:

- **CLASSIFICATION:** *Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soils Classification System and are presented on the exploration logs in Appendix B.*

- **GRADATION ANALYSIS (ASTM D6913):** *Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM D6913. The results of the tests are summarized on Figure C.2 through C.8.*

- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557 METHOD A,B,C):** *The maximum dry density and optimum moisture content of typical soils were determined in the laboratory in accordance with ASTM Standard Test D1557, Method A, Method B, Method C.*

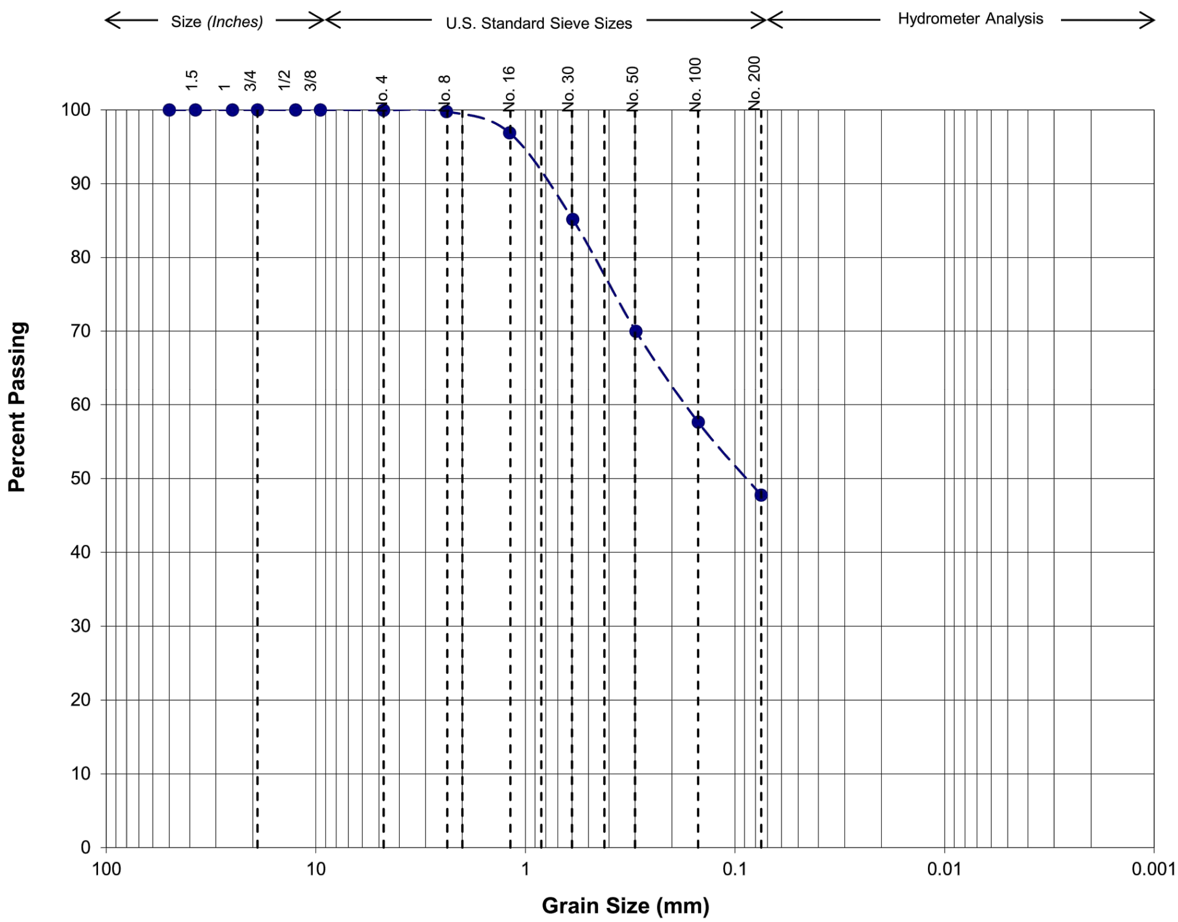
- **ATTERBERG LIMITS (ASTM D 4318):** *Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System.*

- **EXPANSION INDEX (ASTM D4829):** *The expansion index of selected materials was evaluated in general accordance with ASTM D4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours.*

- **R-VALUE (CT 301 and ASTM D 2844):** *The resistance value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D 2844. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results.*

- **CORROSIVITY TEST (CAL. TEST METHOD 417, 422, 643):** *Soil PH, and minimum resistivity tests were performed on a representative soil sample in general accordance with test method CT 643. The sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively.*

- **DIRECT SHEAR (ASTM D3080):** *Direct shear tests were performed on remolded samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions.*



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	TP-9
Depth (ft):	3.5 - 4
USCS Soil Type:	SC
Passing No. 200 (%):	48

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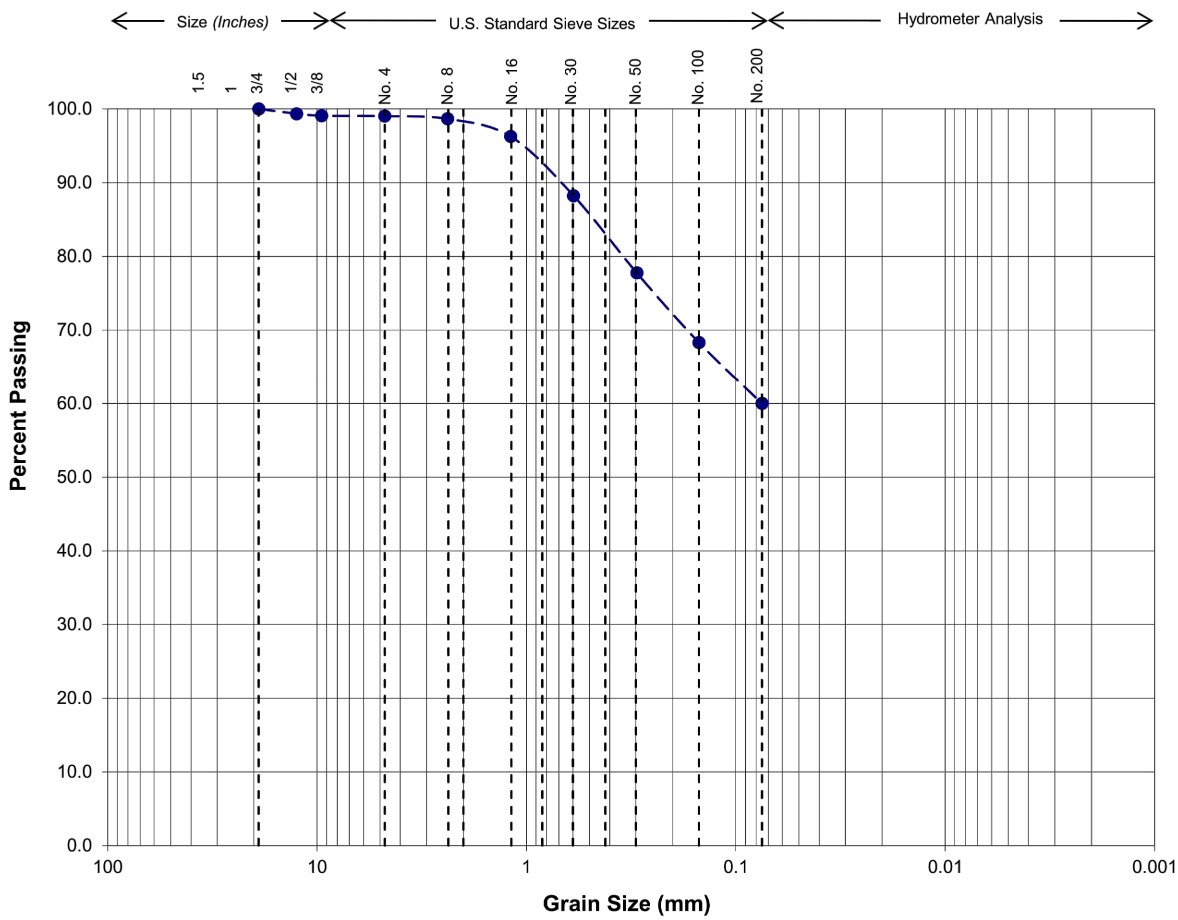
GRADATION ANALYSIS TEST RESULTS

PHEASANT HILL RESIDENTIAL APARTMENTS

LADO DE LOMA DRIVE

VISTA, CA 92083

BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.2



<table>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location:	TP-10
Depth (ft):	1-3
USCS Soil Type:	CL
Passing No. 200 (%):	60

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GRADATION ANALYSIS TEST RESULTS

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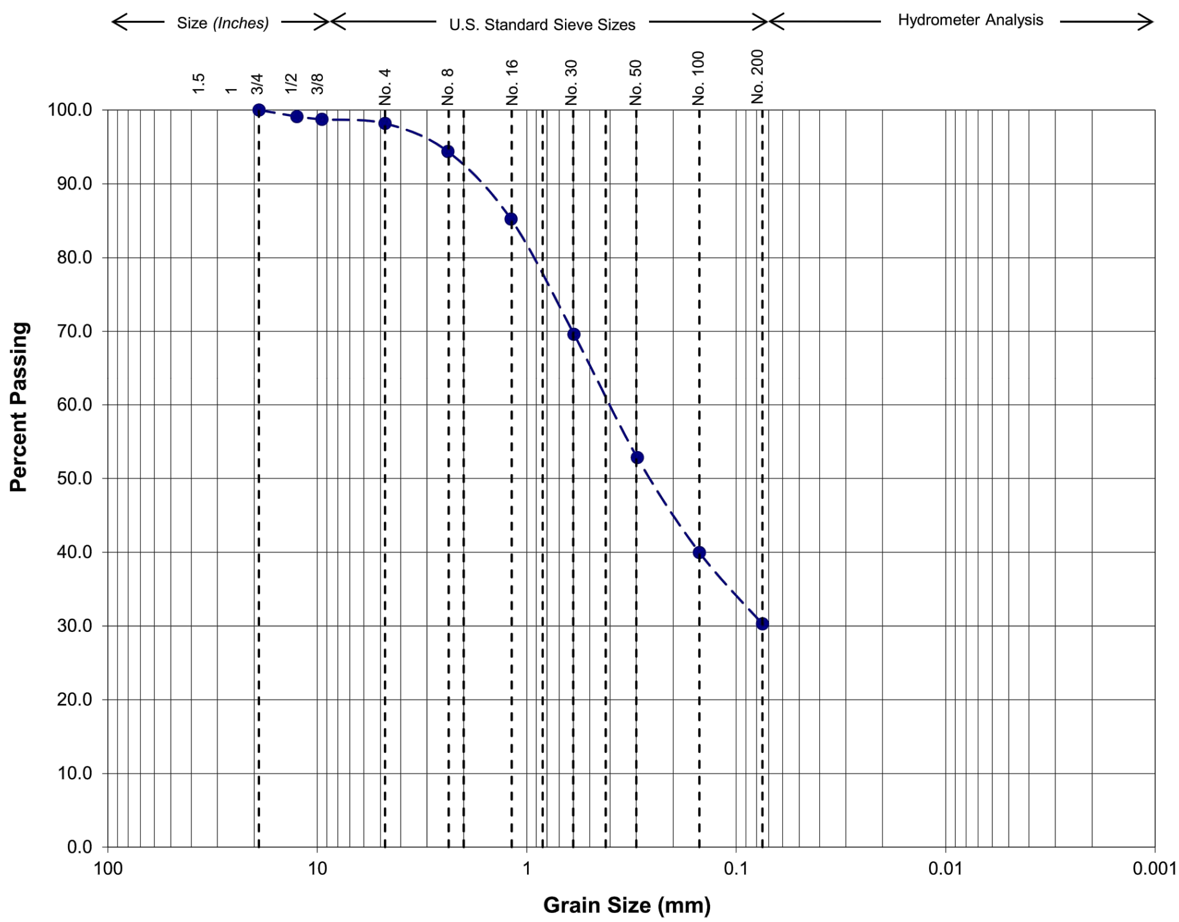
LADO DE LOMA DRIVE

VISTA, CA 92083

BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-12

Depth (ft): 3-7

USCS Soil Type: SM

Passing No. 200 (%): 30

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GRADATION ANALYSIS TEST RESULTS

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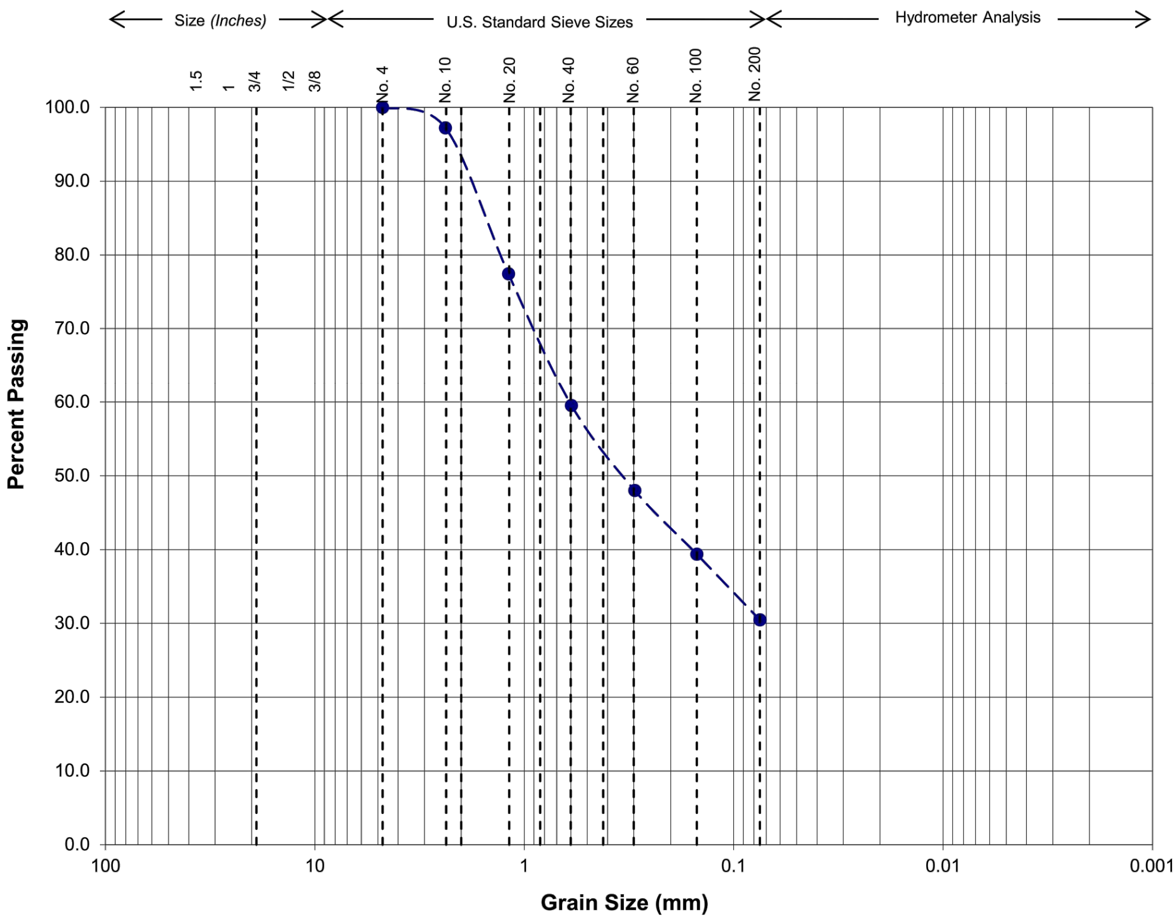
BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.4



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-18

Depth (ft): 3.5 - 5.5

USCS Soil Type: SC

Passing No. 200 (%): 30

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GRADATION ANALYSIS TEST RESULTS

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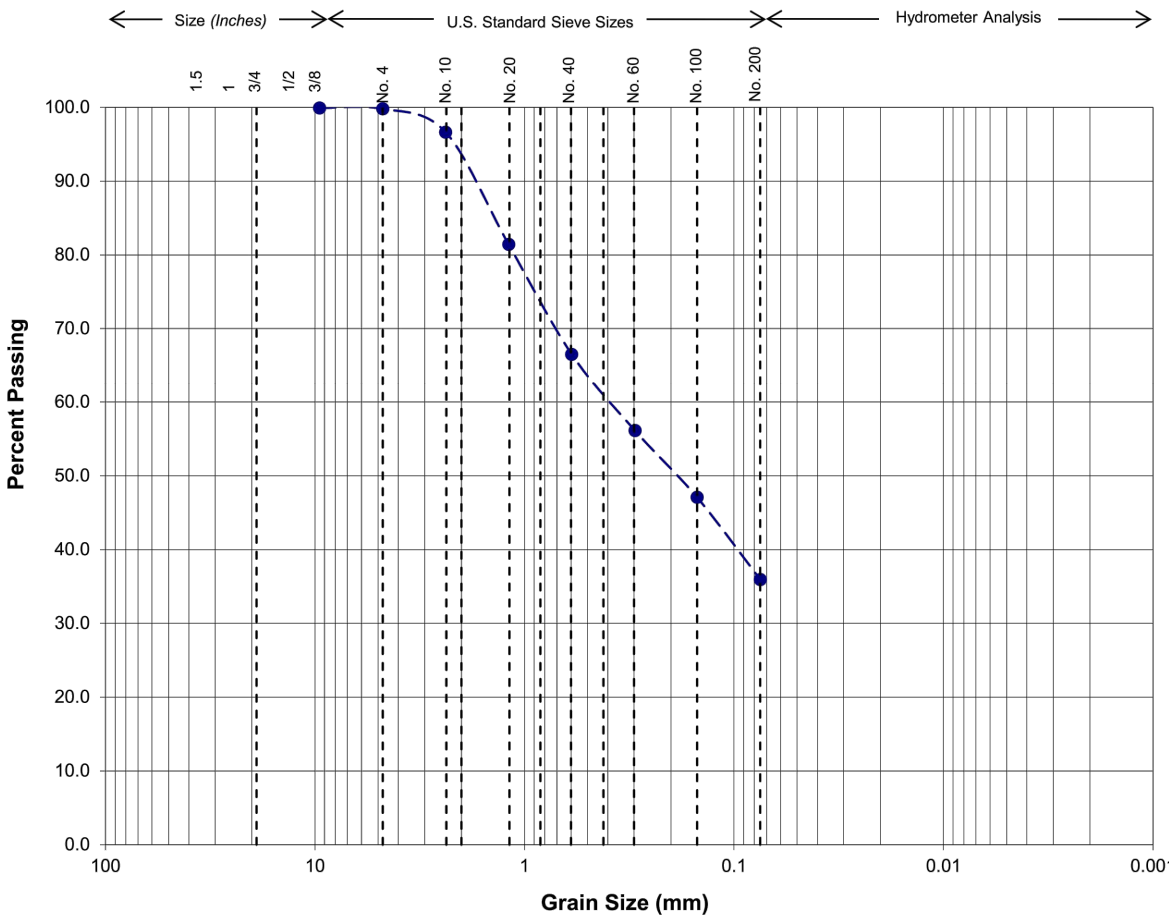
BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.5



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-19

Depth (ft): 0 - 3

USCS Soil Type: SM

Passing No. 200 (%): 36

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GRADATION ANALYSIS TEST RESULTS

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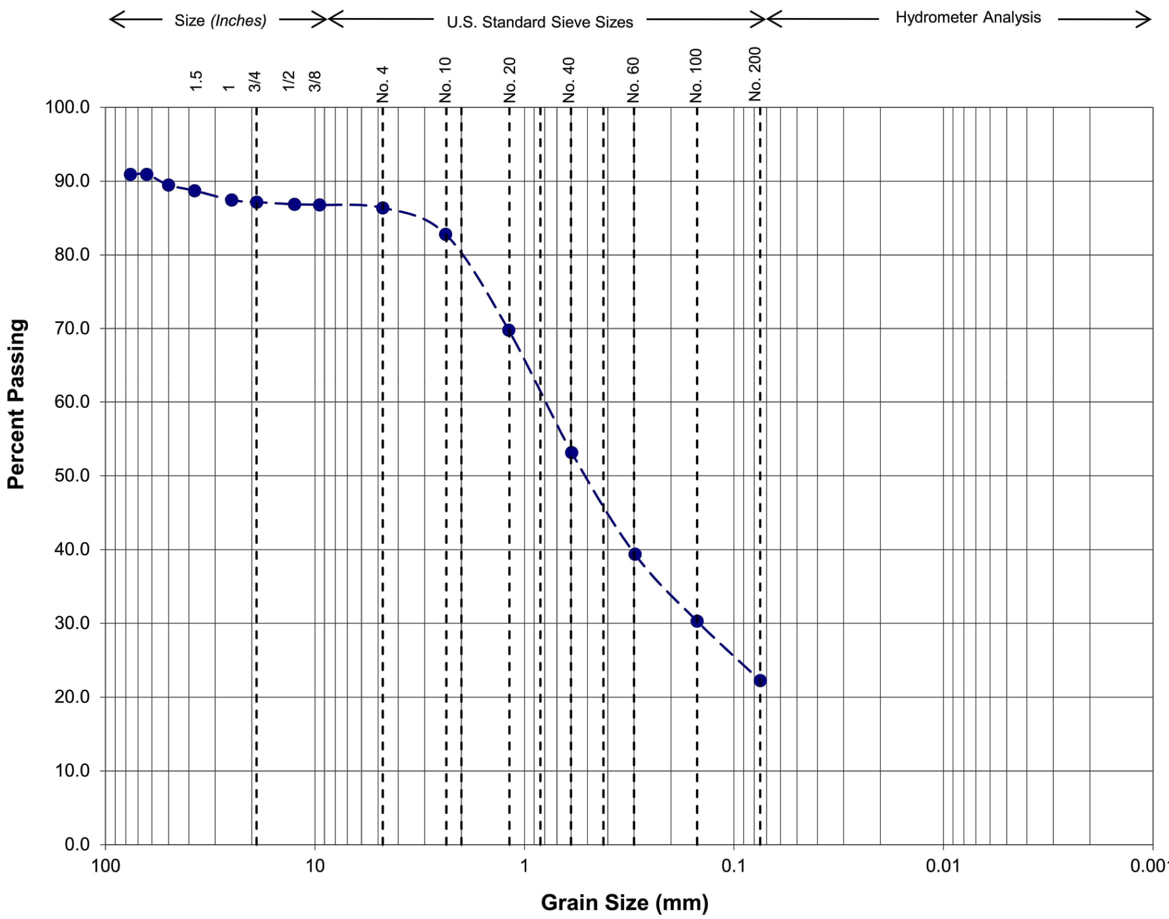
VISTA, CA 92083

BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.6



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-22

Depth (ft): 0-5

USCS Soil Type: SM

Passing No. 200 (%): 22

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GRADATION ANALYSIS TEST RESULTS

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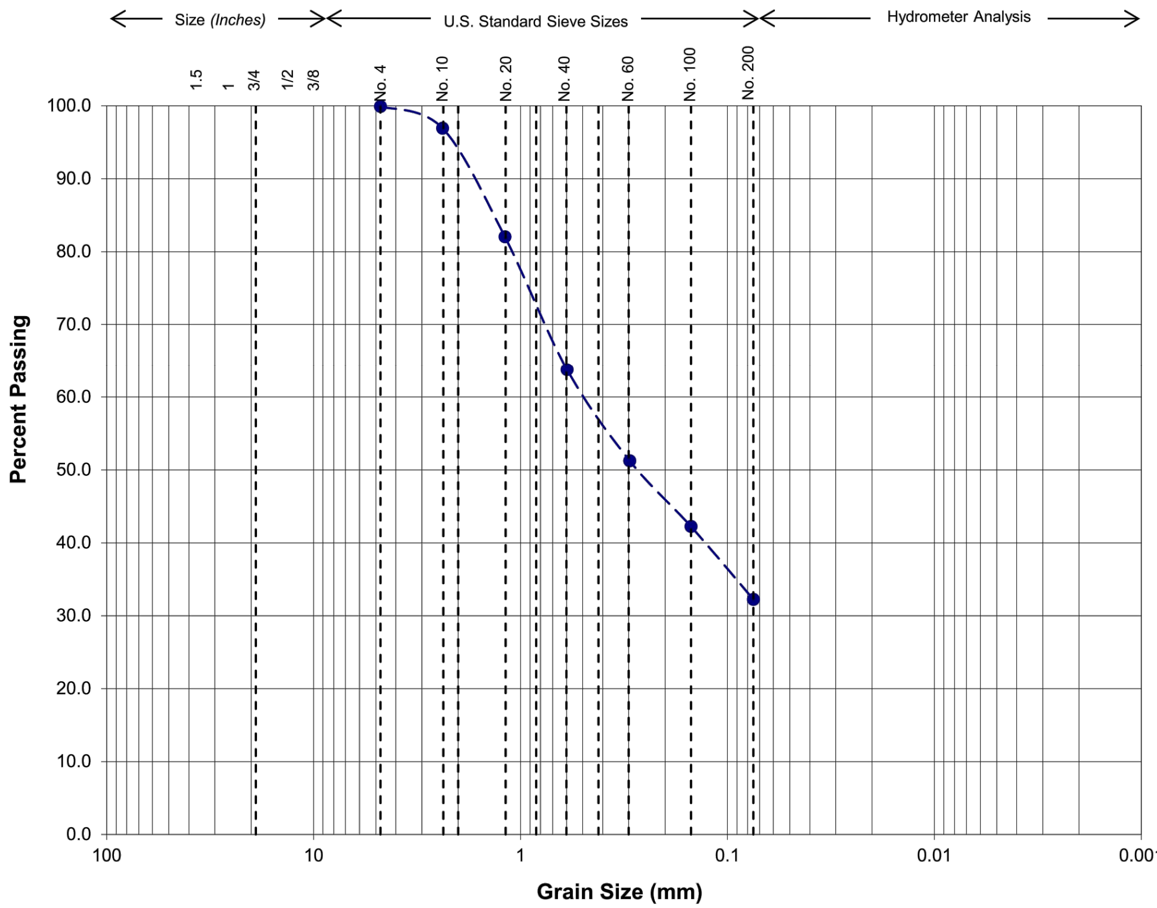
BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.7



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-25

Depth (ft): 1-4

USCS Soil Type: SM

Passing No. 200 (%): 32

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GRADATION ANALYSIS TEST RESULTS

PHEASANT HILL RESIDENTIAL APARTMENTS

LADO DE LOMA DRIVE

VISTA, CA 92083

BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.8

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP - 5	2 - 3	Orange Brown Silty Sand	131.7	9.0
TP - 12	3 - 7	Brown Silty Sand	132.2	8.0
TP - 13	0 - 3	Dark Brown Clayey Silty Sand	133.9	6.6
TP - 19	0 - 3	Dark Brown Silty Sand	132.7	7.9
TP - 23	1 - 4	Orange Brown Silty Sand	130.5	9.2

Atterberg Limits (ASTM D4318)

Sample Location	Sample Depth (ft.)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	USCS (% Finer than No. 40)
TP - 9	3.5 - 4	36	17	19	CL
TP - 10	1 - 3	43	16	27	CL
TP - 18	3.5 - 5.5	37	19	19	CL

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft.)	Expansion Index	Expansion Potential
TP - 9	3.5 - 4	29	Low
TP - 18	3.5 - 5.5	10	Very Low

Resistance Value (Cal. Test Method 301 & ASTM D2844)

Sample Location	Sample Depth (ft.)	Soil Description	R-Value
TP - 13	0 - 3	Dark Brown Silty Sand	44
TP - 19	0 - 3	Dark Brown Silty Sand	18
TP - 21	0 - 3	Orange Brown Silty Sand	70

Corrosivity (Cal. Test Method 417,422,643)

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
TP - 10	1 - 3	8.3	730	<30	<0.003	21	0.002
TP - 13	0 - 3	8.2	17000	<30	<0.003	43	0.004
TP - 18	3.5 - 5.5	8.8	2900	42	0.004	32	0.003
TP - 22	0 - 5	7.3	1200	240	0.024	64	0.006

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LAB TEST RESULTS

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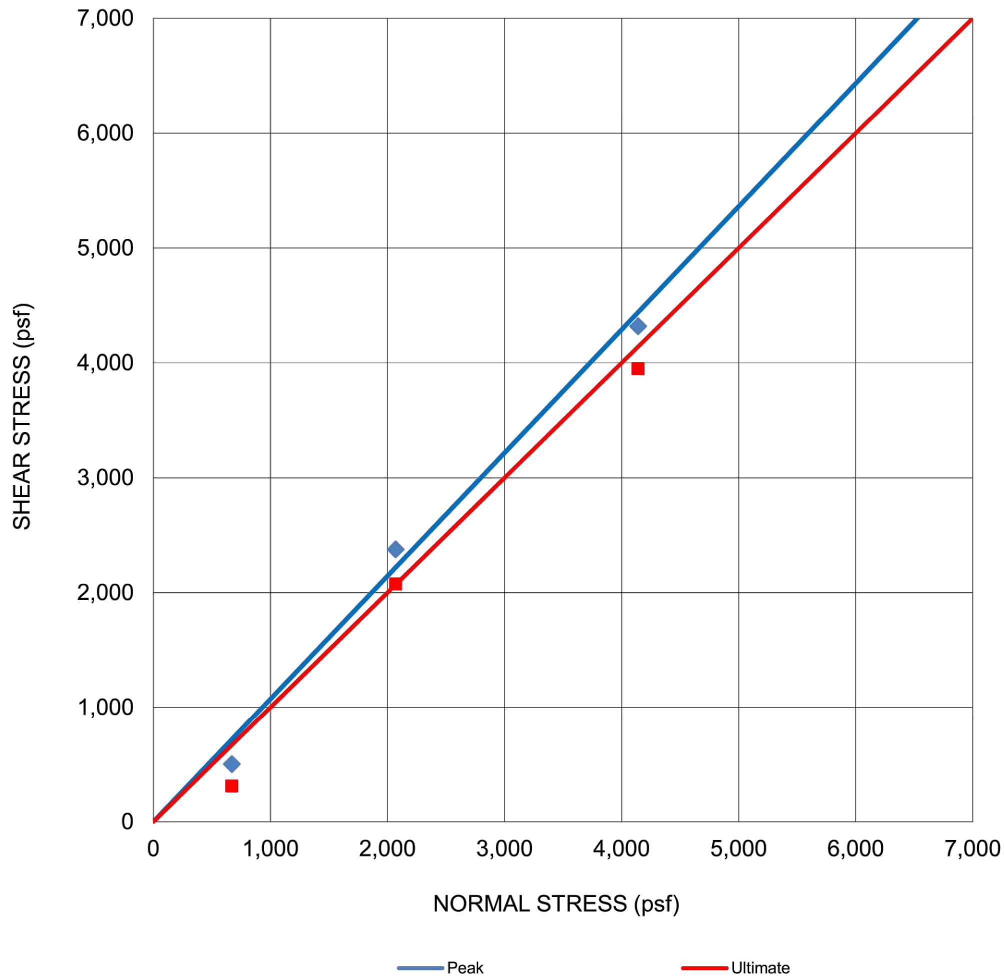
BY: GN	DATE: FEB 2022	PROJECT: 2021172	APPENDIX: C.9



Sample: TP-23@ 1'-4'

Description: Weathered Granitics (Kt)

Sample Type: In Situ

Strain Rate: 0.0015 in./min.

	Peak	Ultimate
φ'	47 °	45 °
C'	0 psf	0 psf

	In Situ	As Tested
γ_d	116.5 pcf	116.5 pcf
w_c	9.8 %	17.6 %

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DIRECT SHEAR TEST RESULTS

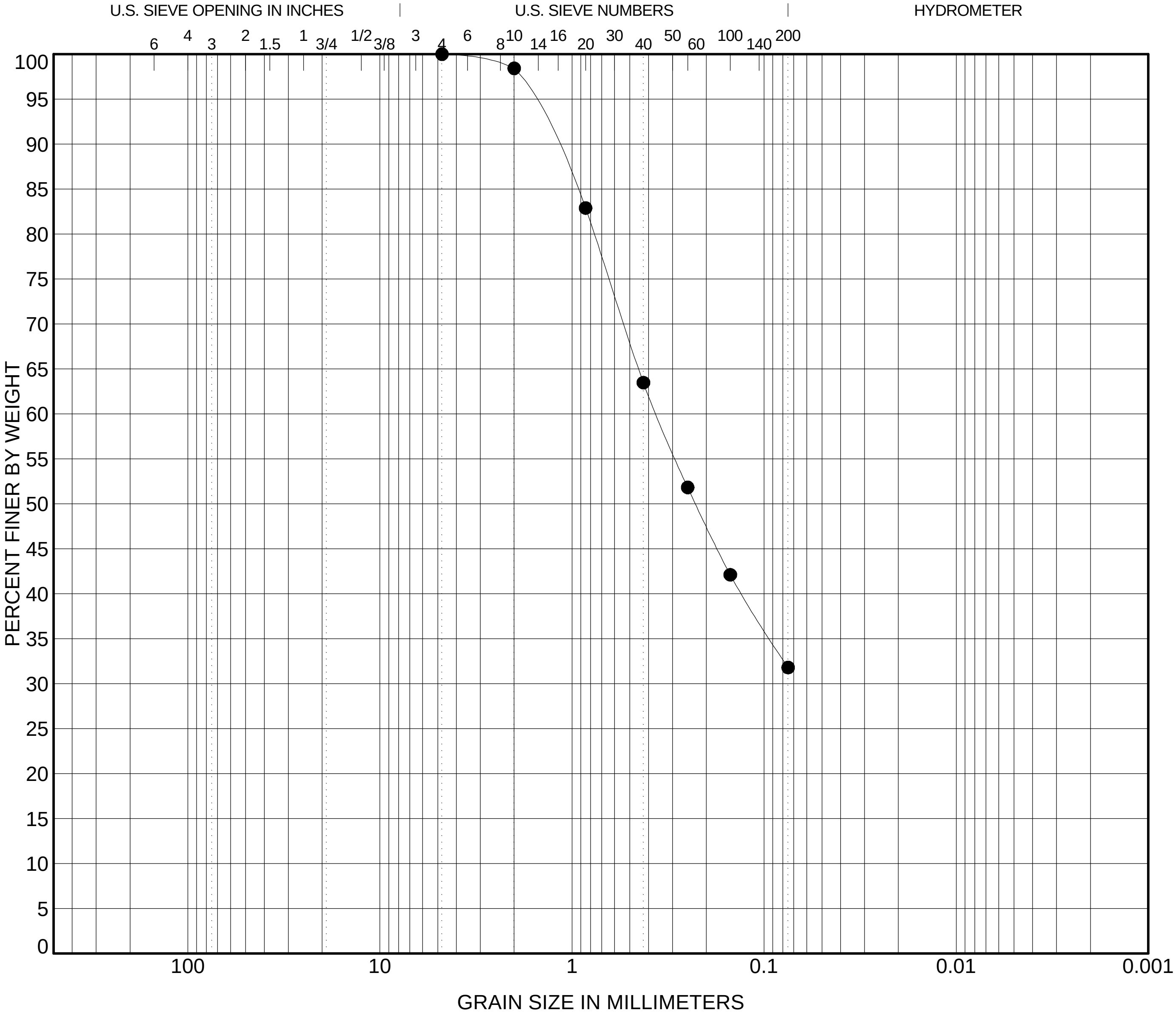
PHEASANT HILL RESIDENTIAL APARTMENTS

LADO DE LOMA DRIVE

VISTA, CA 92083

BY: GN	DATE: FEB 2022	PROJECT: 2021172	FIGURE: C.10

LABORATORY TESTING BY OTHERS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	TP-2	3.0	Silty Sand					

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-2	3.0	4.75	0.363			0.0	68.2	31.8	

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Fax: 760-931-0915

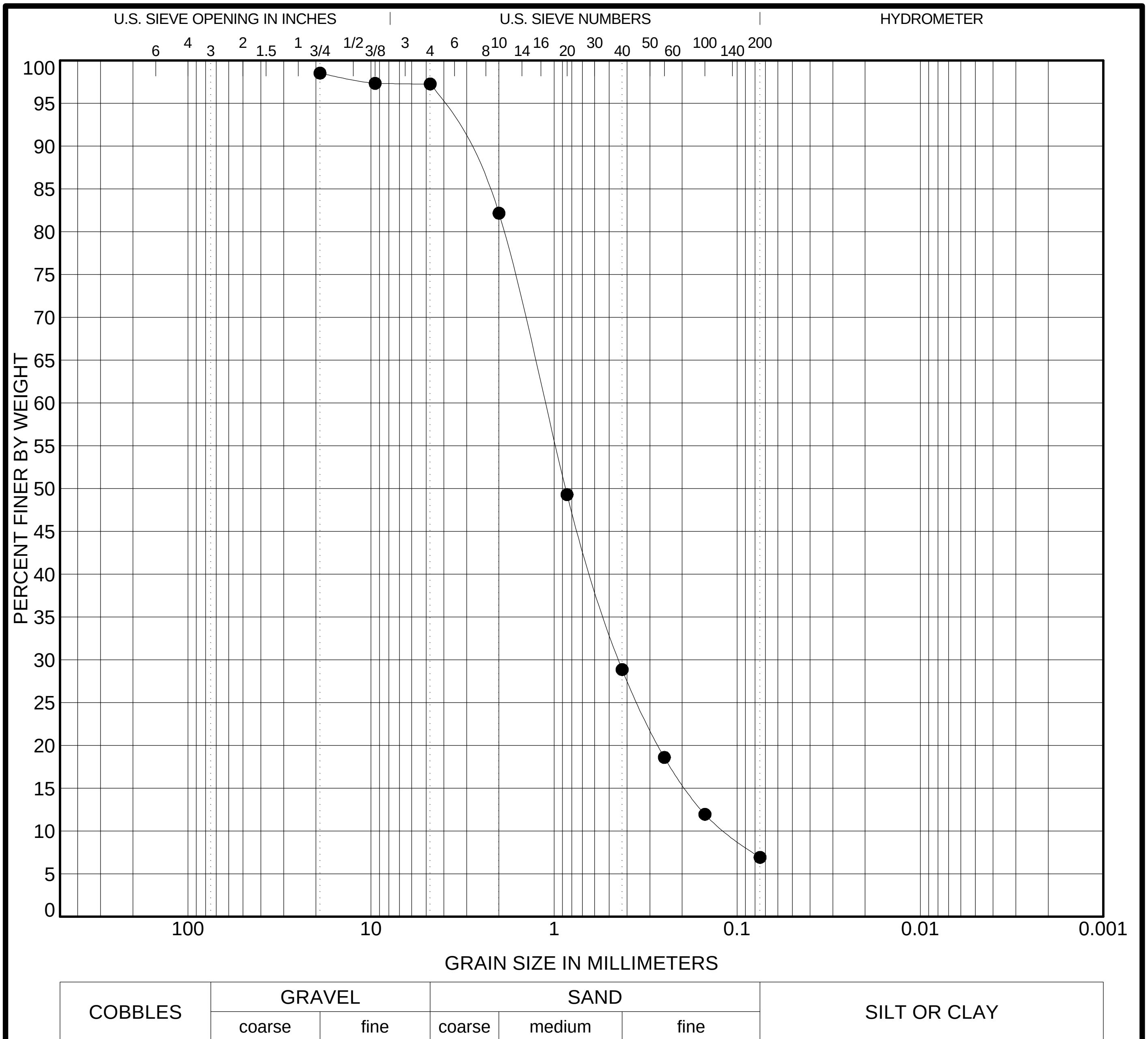
GRAIN SIZE DISTRIBUTION

Project: RINA LLC

Number: 7089-A-SC

Date: July 2016

Figure: D - 1



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	TP-7	3.0	Silty Sand				1.52	9.80

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-7	3.0	19	1.123	0.442	0.115	1.3	90.3	6.9	

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GRAIN SIZE DISTRIBUTION

Project: RINA LLC

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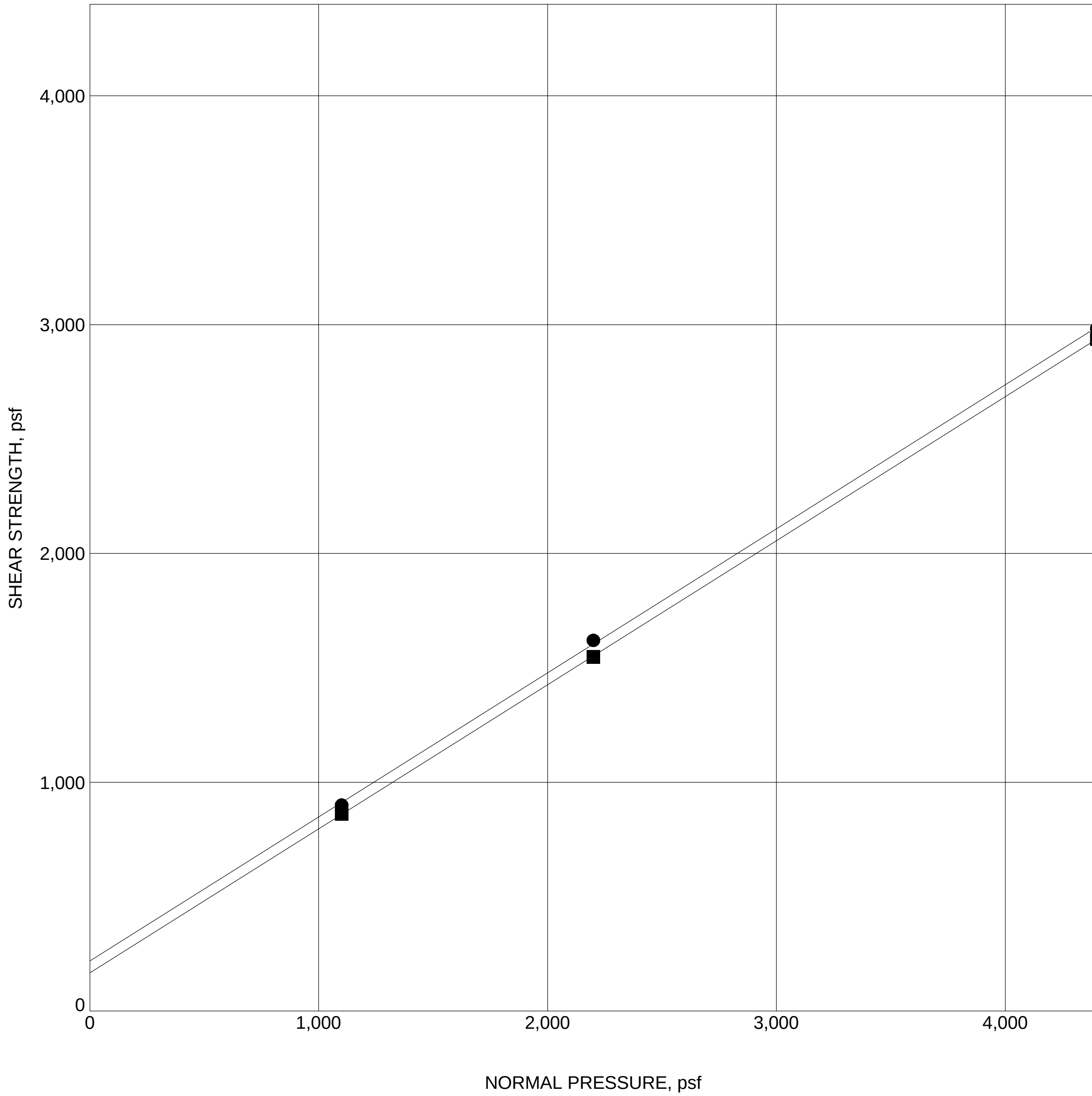
Date: July 2016

Figure: D - 2



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	TP-2	3.0	Primary Shear	Remolded	117.0	10.0	218	32
■	TP-2	3.1	Residual Shear	Remolded	117.0	10.0	166	32

Note: Sample Innundated prior to testing

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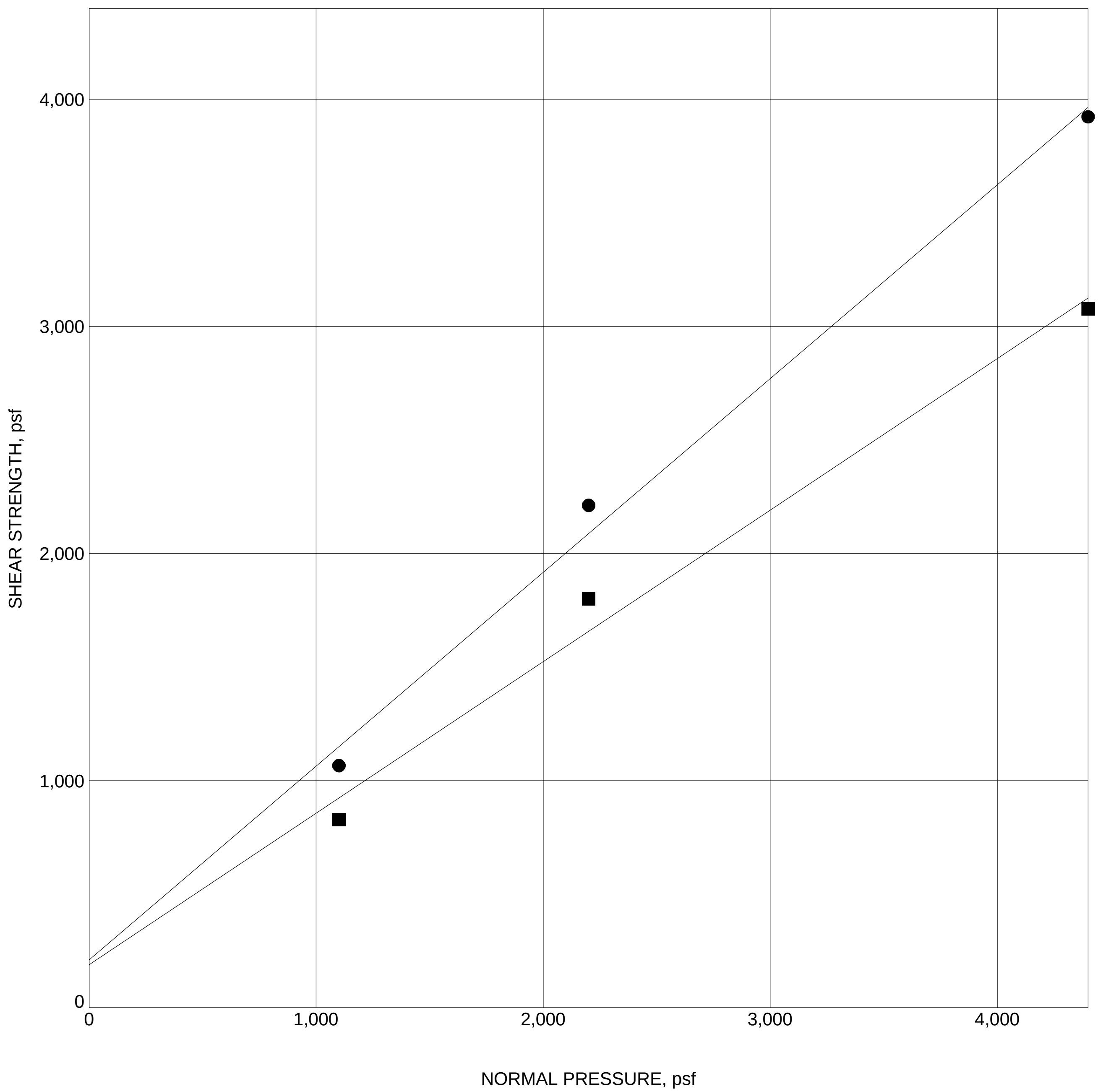
DIRECT SHEAR TEST

Project: RINA LLC

Number: 7089-A-SC

Date: July 2016

Figure: D - 3



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	TP-7	3.0	Primary Shear	Remolded	118.8	9.0	210	40
■	TP-7	3.1	Residual Shear	Remolded	118.8	9.0	189	34

Note: Sample Innundated prior to testing

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DIRECT SHEAR TEST

Project: RINA LLC

Number: 7089-A-SC

Date: July 2016

Figure: D - 4

SUMMARY OF LABORATORY TEST DATA

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QCI Project No.: 16-029-006h
Date: June 10, 2016
Summarized by: KA

W.O. 7089-A-SC
Project Name: Pheasant Hills
Client: Rina LLC

Corrosivity Test Results

Sample ID	Sample Depth (ft)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 % By Weight	Resistivity CT-532 (643) (ohm-cm)
TP-2	2-3’	8.79	180	0.0170	660

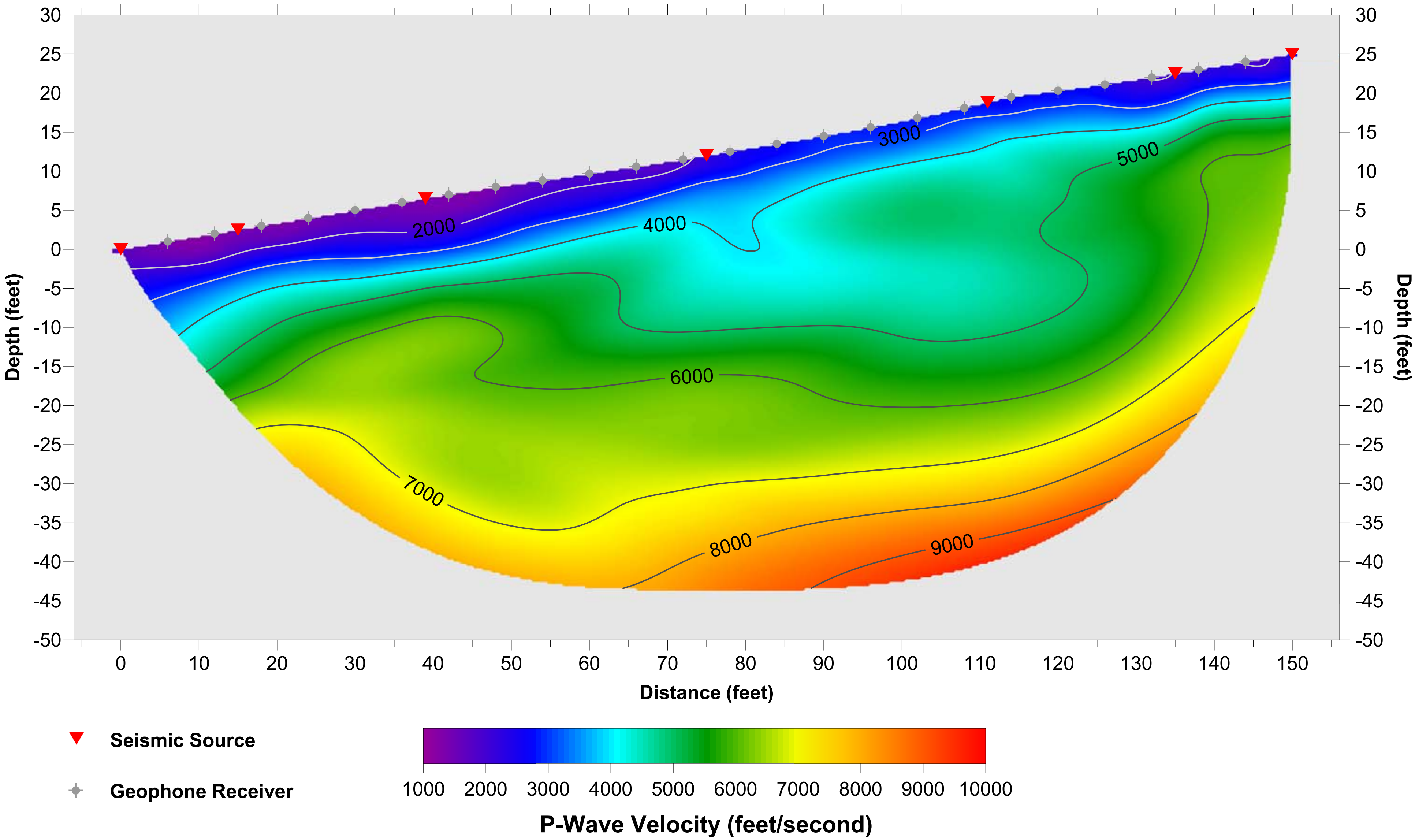
576 East Lambert Road, Brea, California 92821; Tel: 714-671-1050; Fax: 714-671-1090

APPENDIX D
RECORDS OF GEOPHYSICAL TESTING

SEISMIC LINE S-1

South 38° West →

REFRACTION TOMOGRAPHIC MODEL



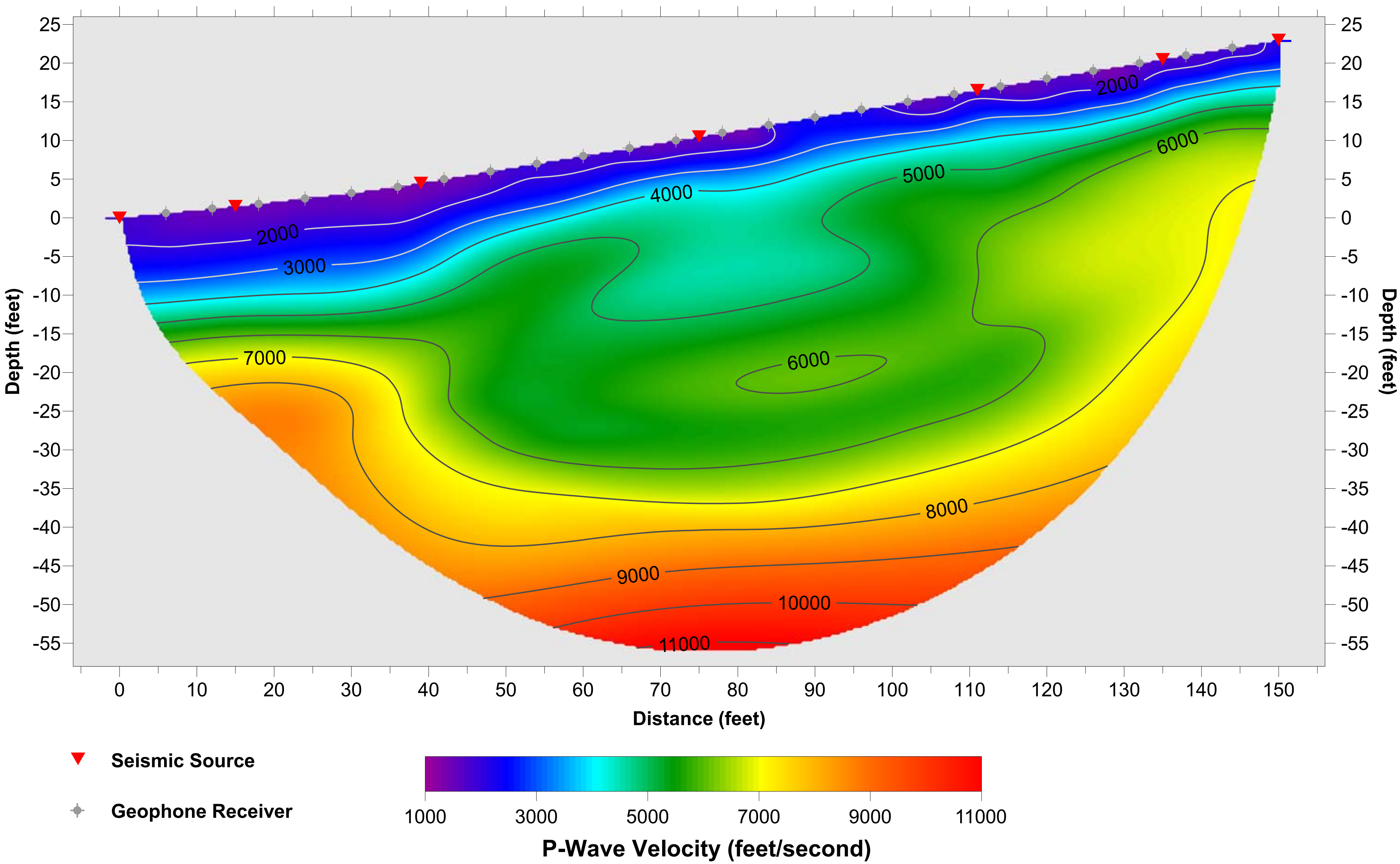
SCALE: 1:1 (Horizontal = Vertical)

RMS error 2.9%, Rayfract Version 4.02

SEISMIC LINE S-2

South 60° West →

REFRACTION TOMOGRAPHIC MODEL



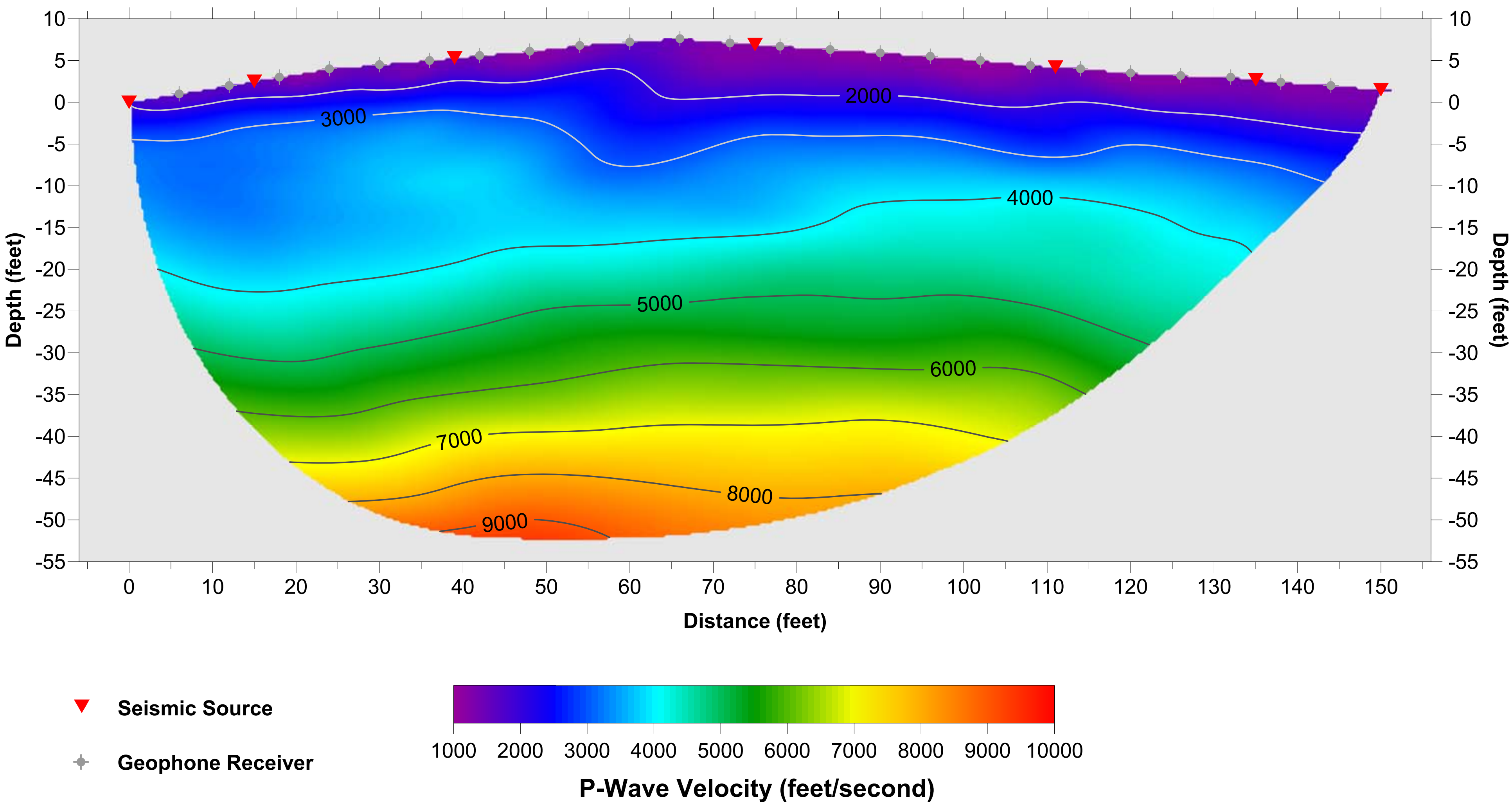
SCALE: 1:1 (Horizontal = Vertical)

RMS error 2.1%, Rayfract Version 4.02

SEISMIC LINE S-3

South 62° East →

REFRACTION TOMOGRAPHIC MODEL



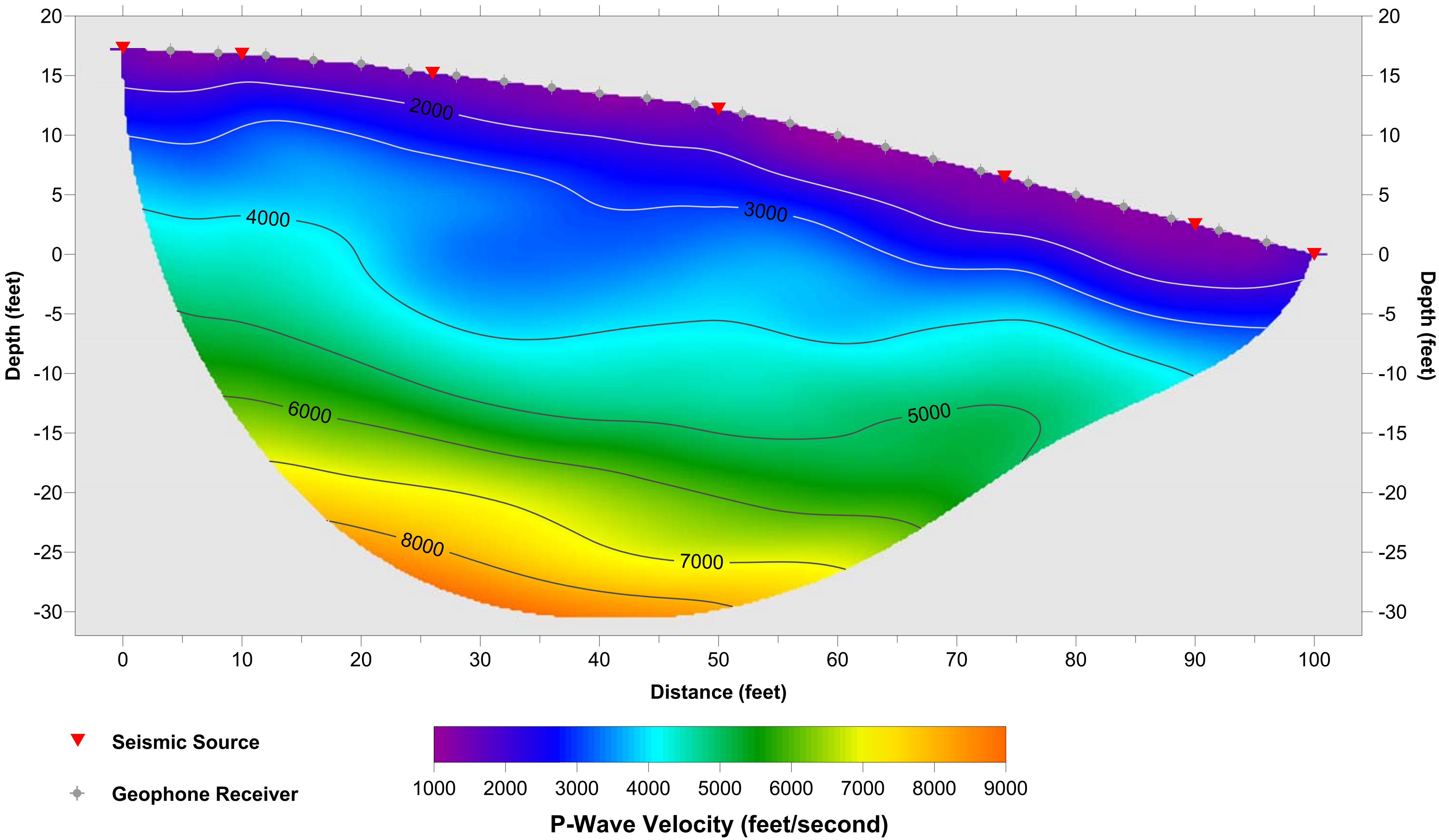
SCALE: 1:1 (Horizontal = Vertical)

RMS error 1.9%, Rayfract Version 4.02

SEISMIC LINE S-4

North 5° East →

REFRACTION TOMOGRAPHIC MODEL



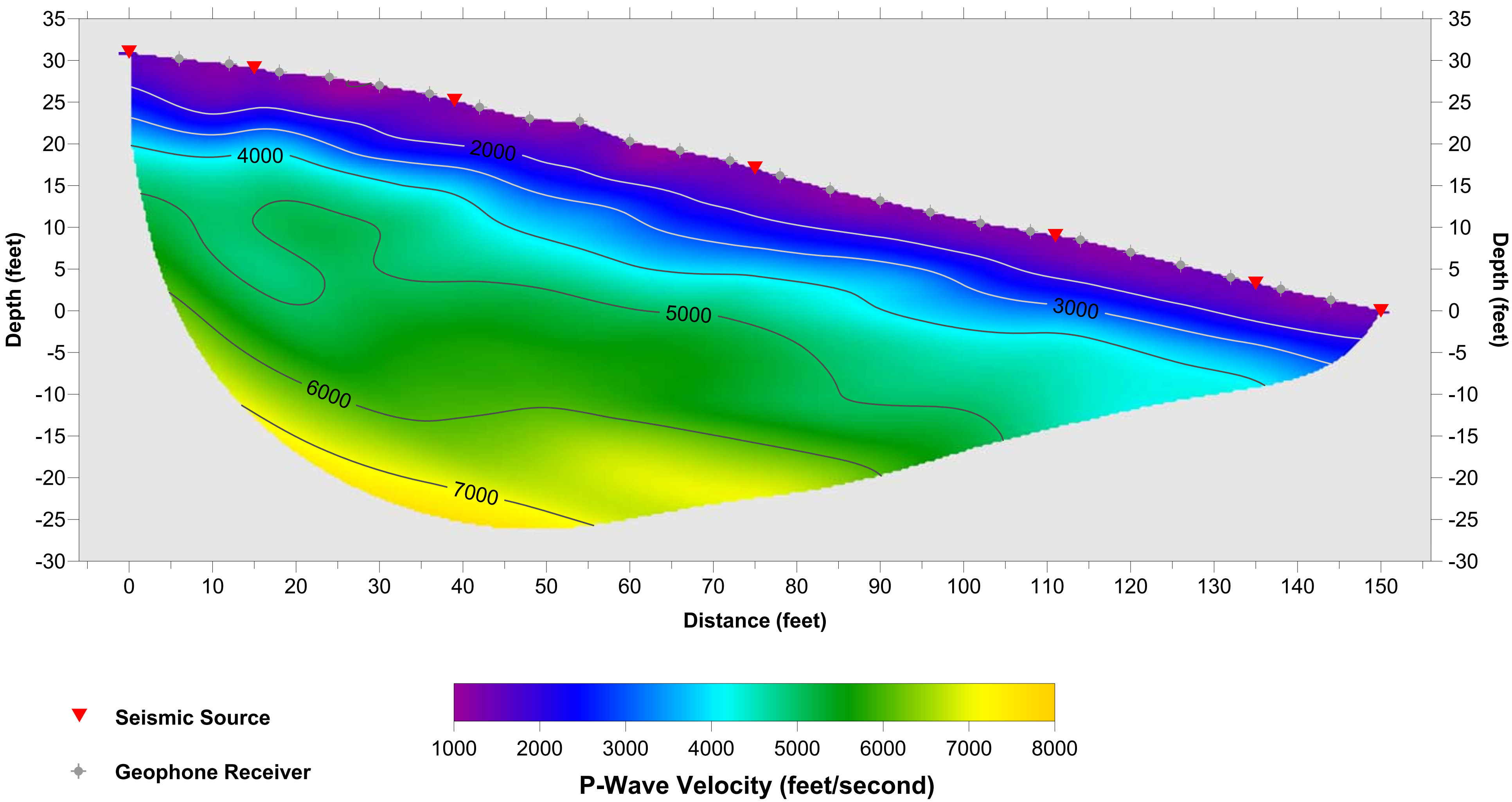
SCALE: 1:1 (Horizontal = Vertical)

RMS error 2.1%, Rayfract Version 4.02

SEISMIC LINE S-5

North 66° East →

REFRACTION TOMOGRAPHIC MODEL



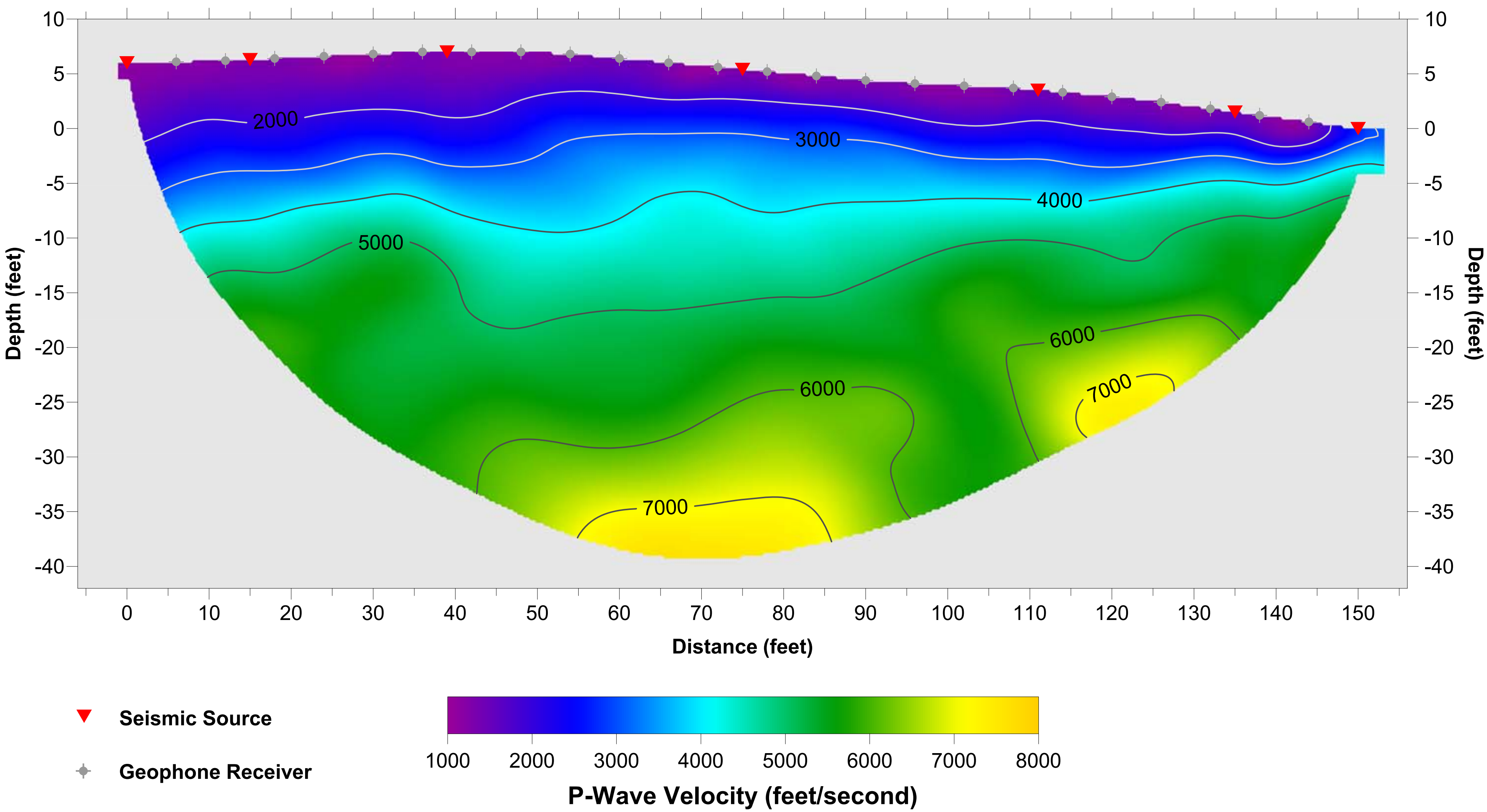
SCALE: 1:1 (Horizontal = Vertical)

RMS error 2.8%, Rayfract Version 4.02

SEISMIC LINE S-6

South 36° East →

REFRACTION TOMOGRAPHIC MODEL



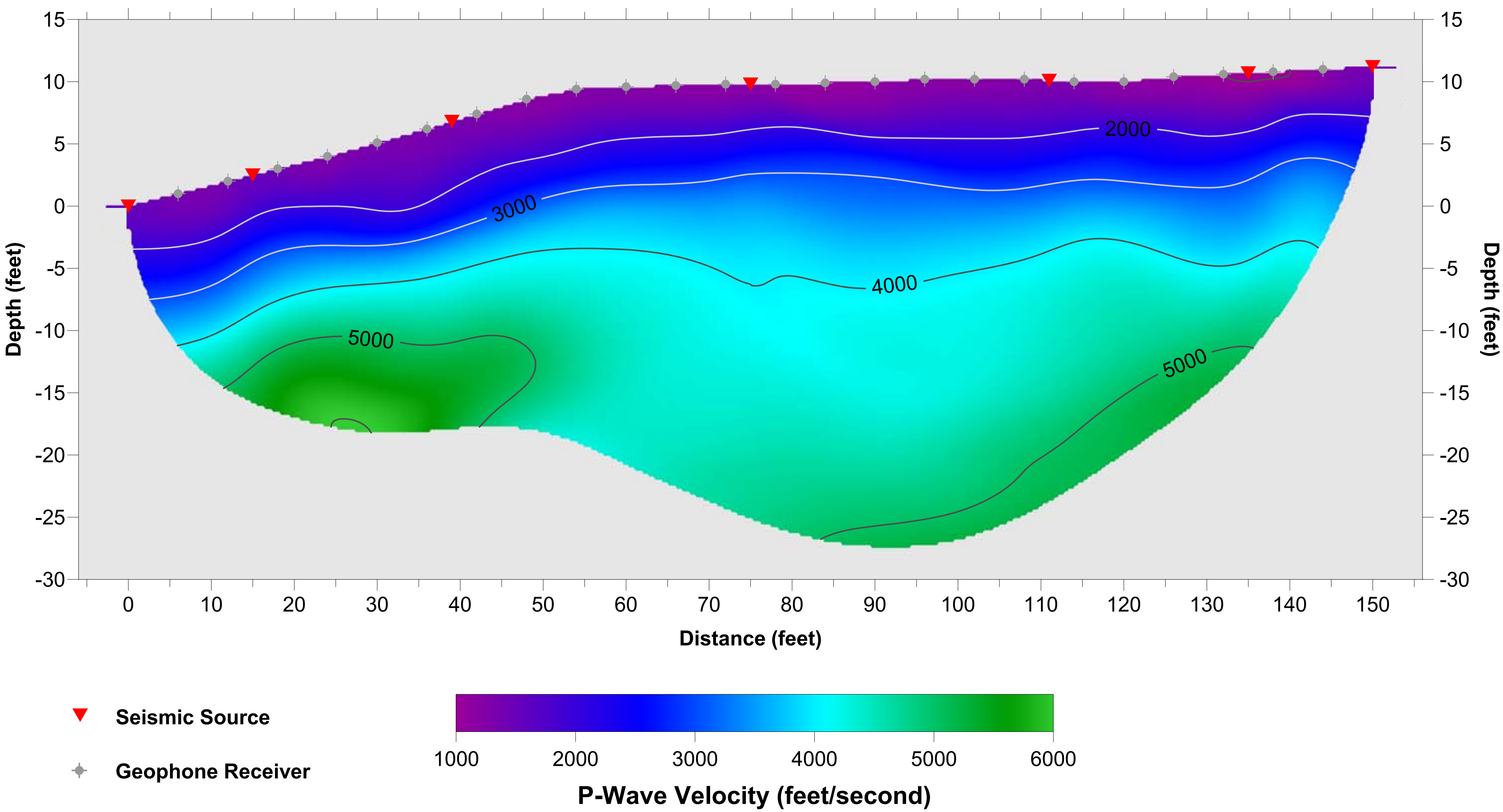
SCALE: Vertical Exaggeration 1.33X

RMS error 4.2%, Rayfract Version 4.02

SEISMIC LINE S-7

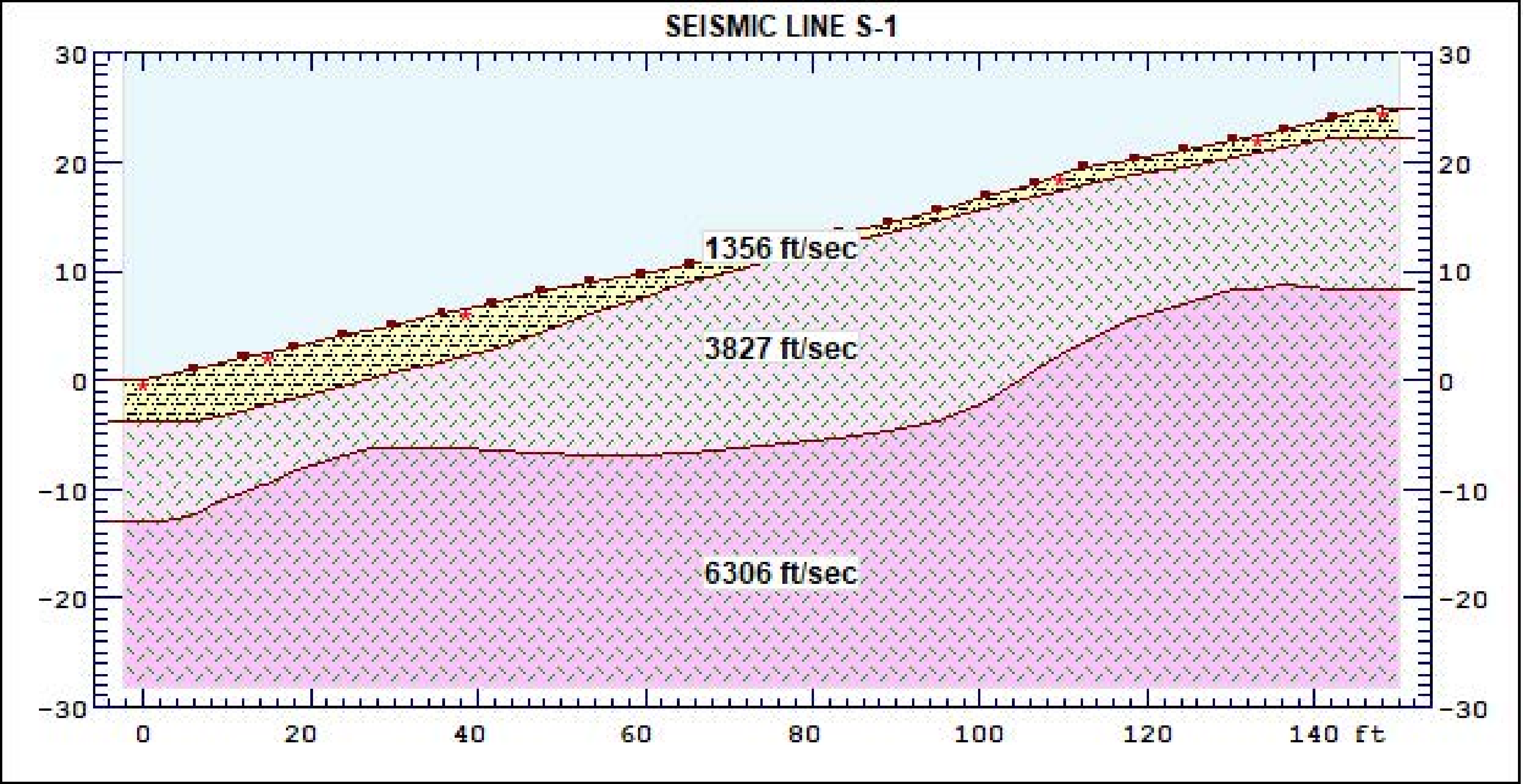
South 35° East →

REFRACTION TOMOGRAPHIC MODEL

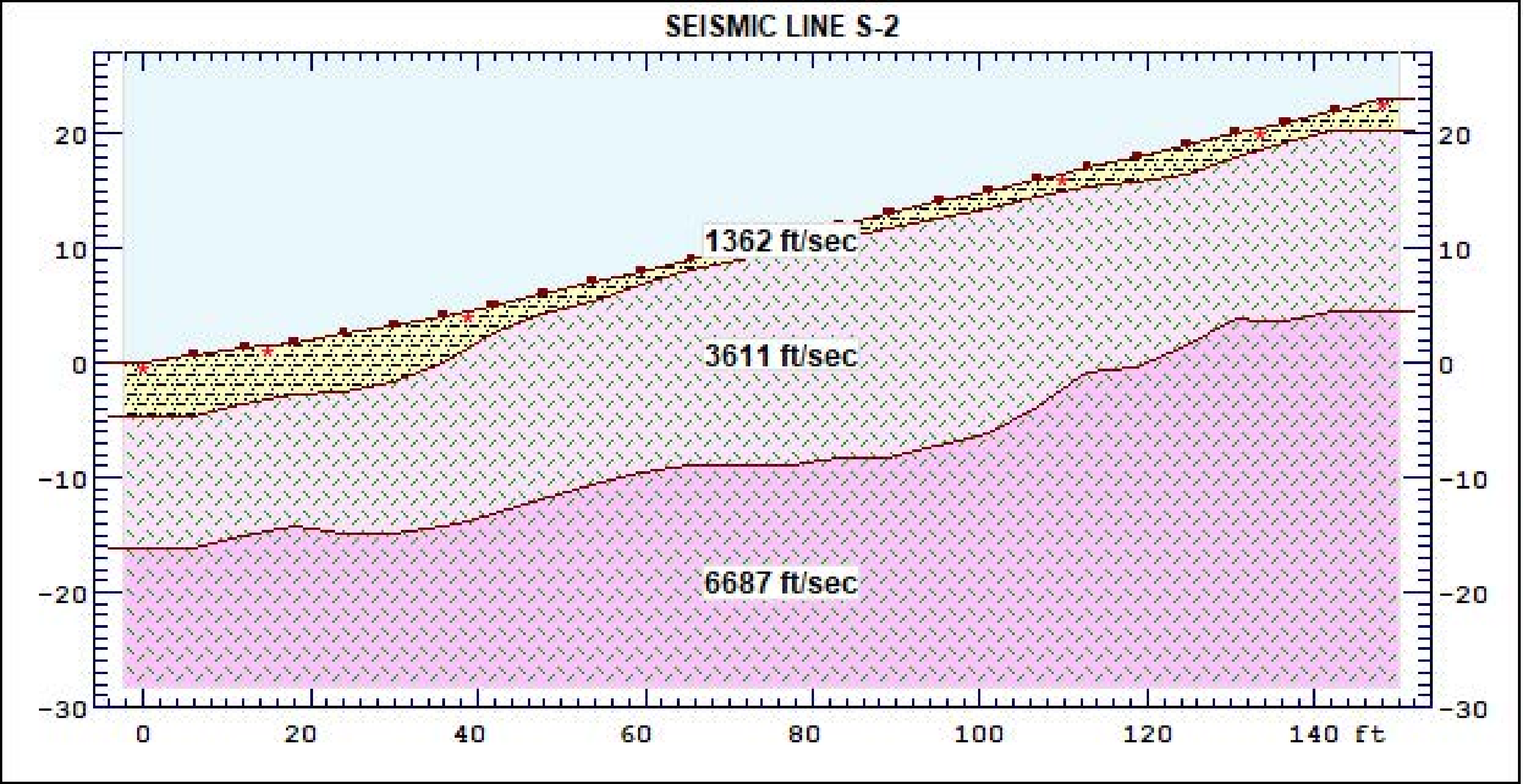


SCALE: Vertical Exaggeration 1.5X

RMS error 3.4%, Rayfract Version 4.02



SEISMIC LINE S-1
1356 ft/sec
3827 ft/sec
6306 ft/sec
30
20
10
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft


SEISMIC LINE S-2
1362 ft/sec
3611 ft/sec
6687 ft/sec
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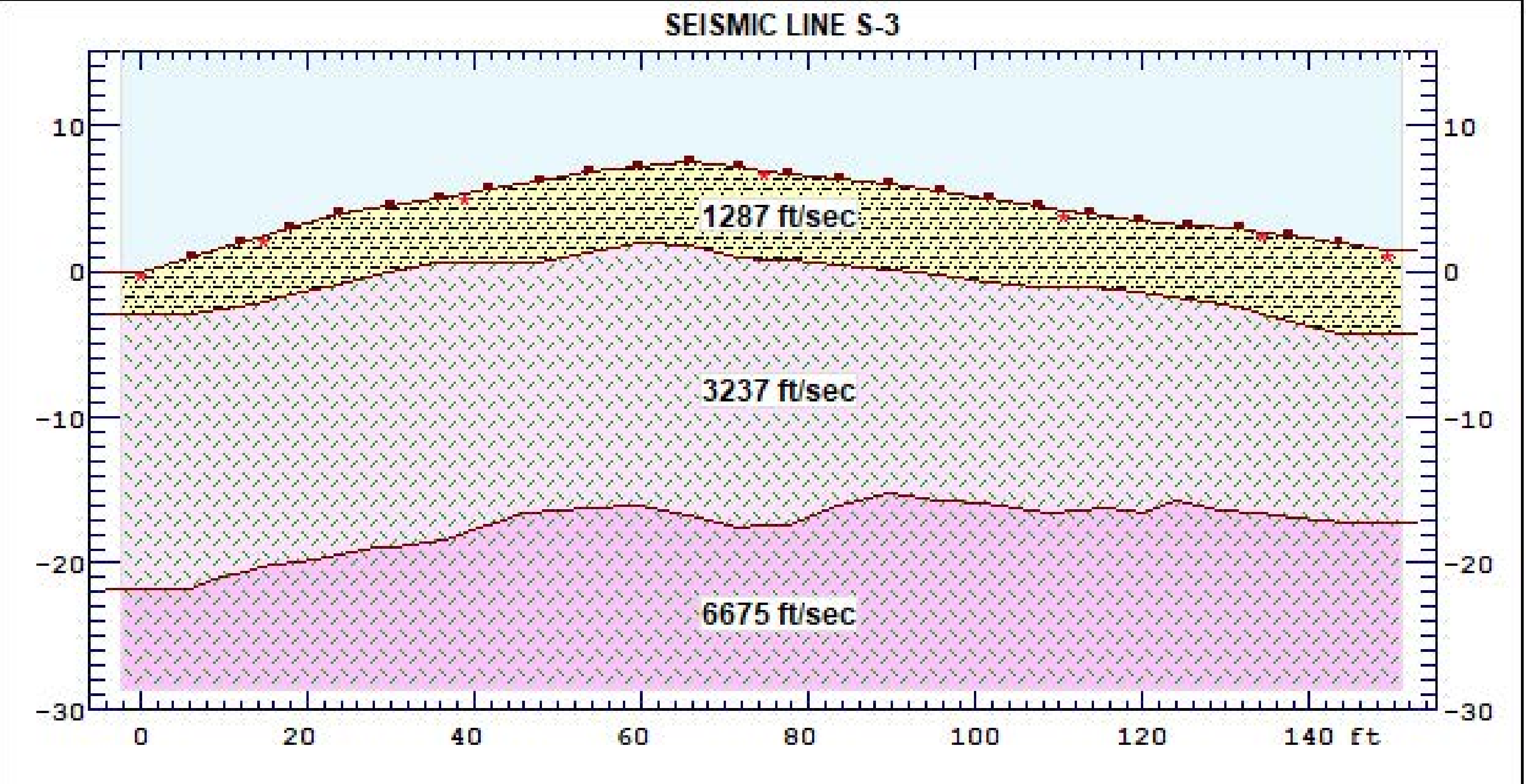
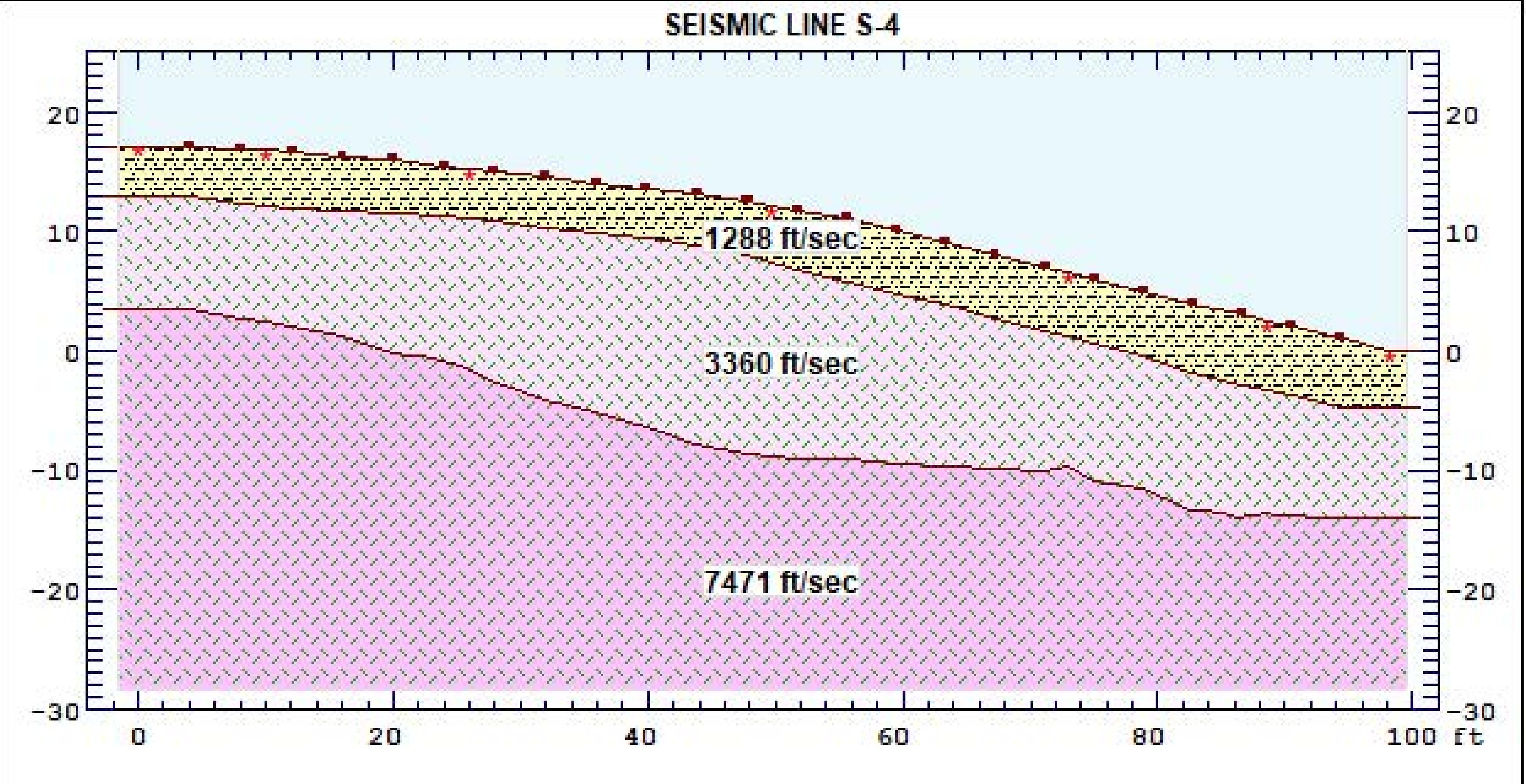
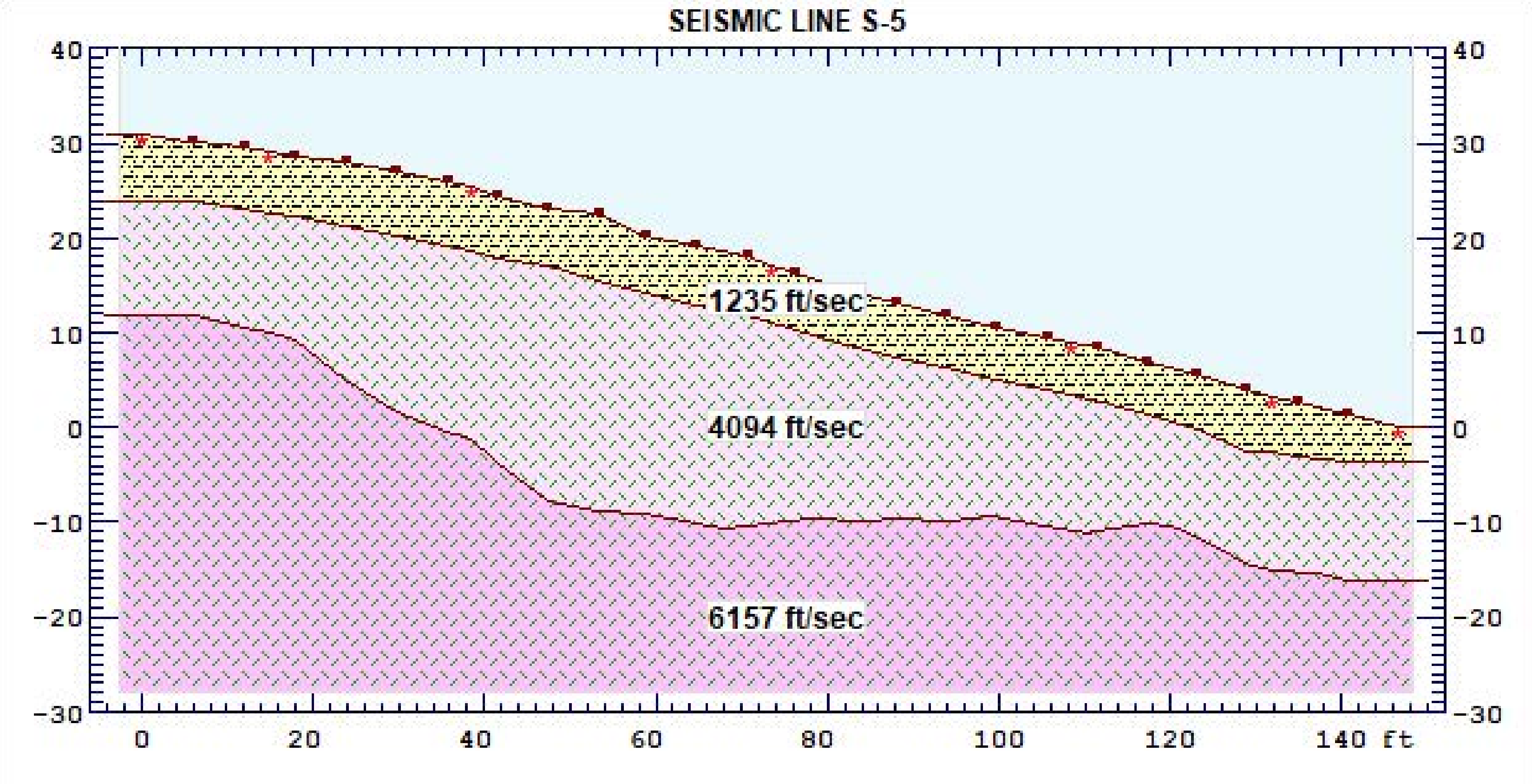
10
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft

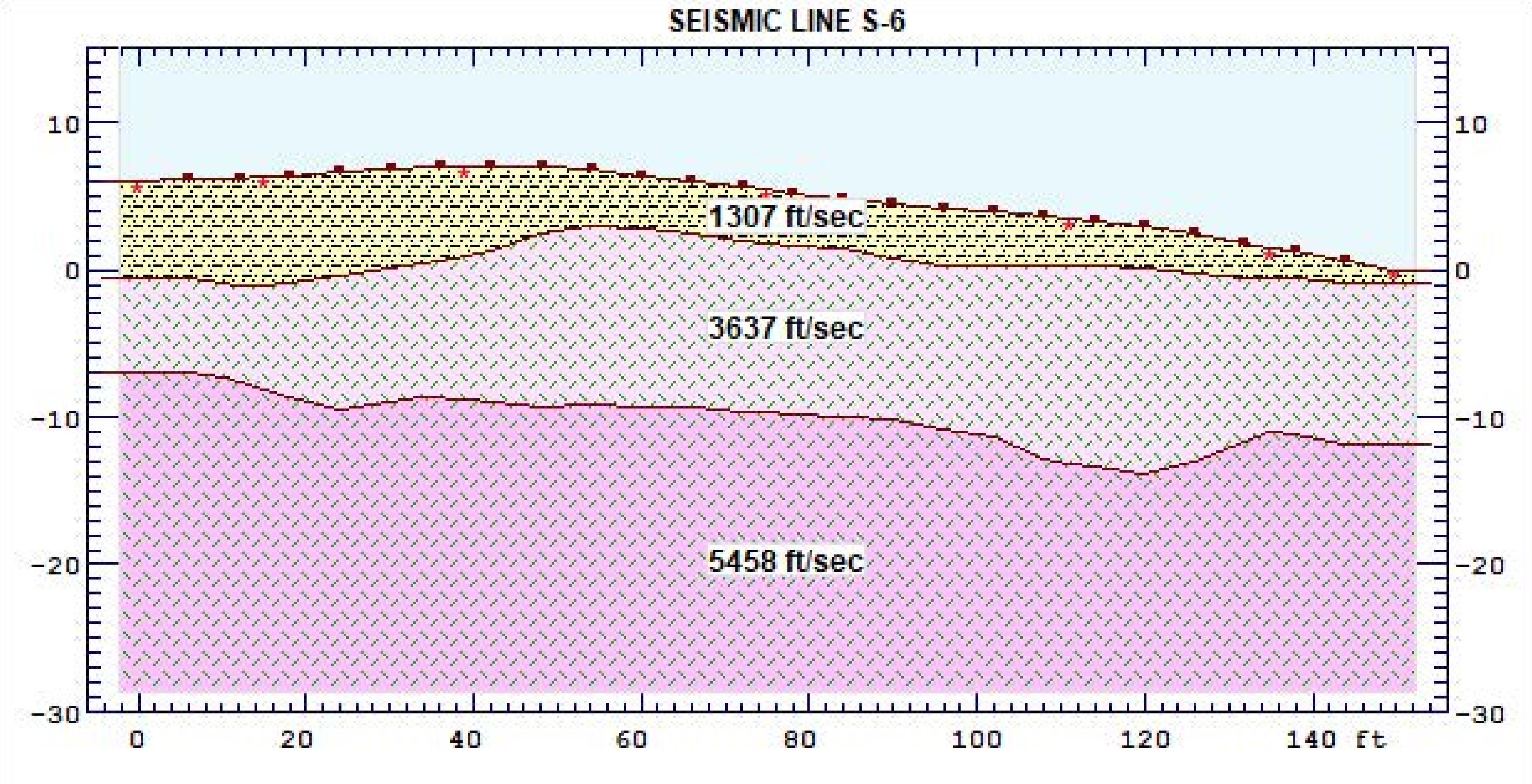


SEISMIC LINE S-3
1287 ft/sec
3237 ft/sec
6675 ft/sec
10
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft

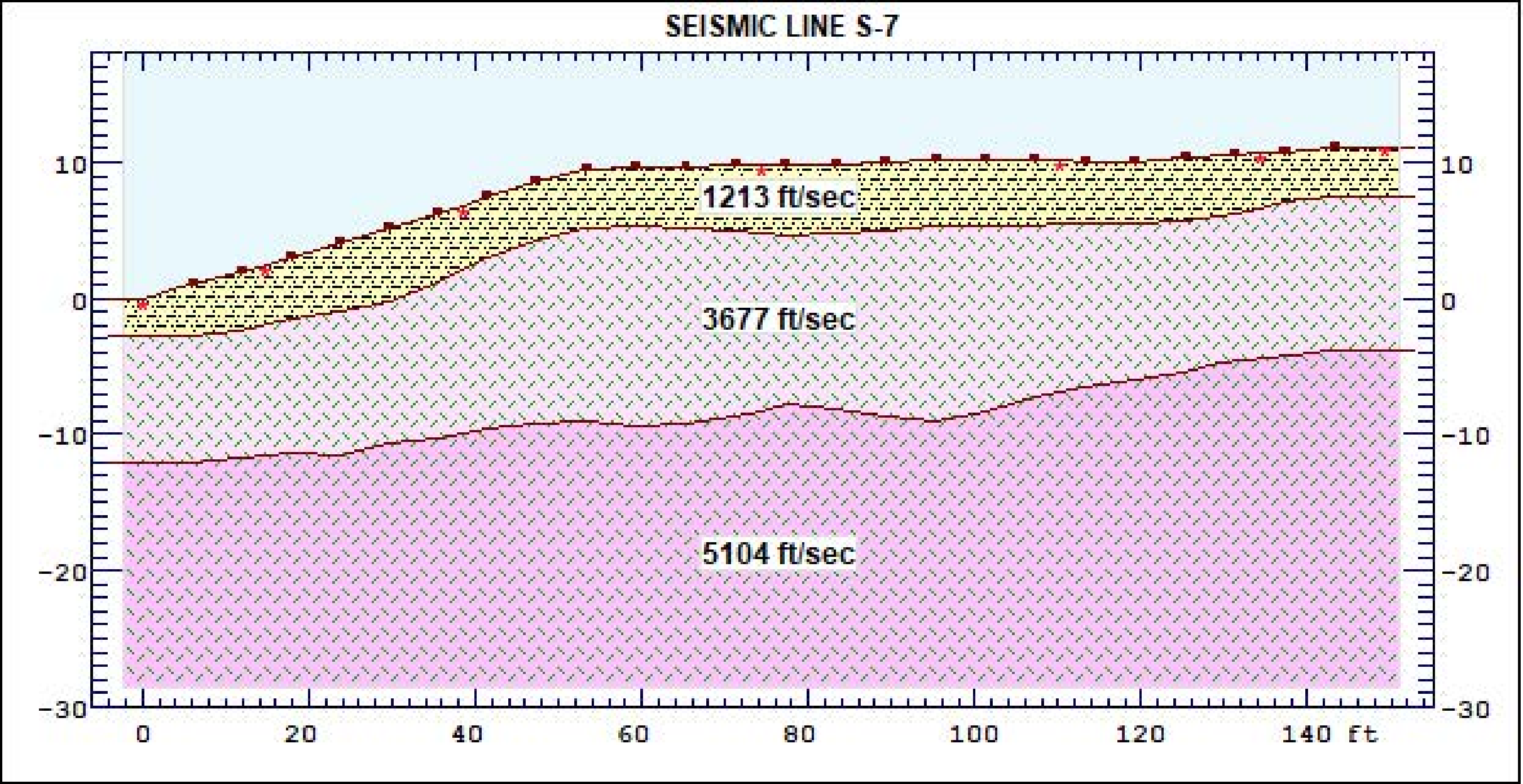

SEISMIC LINE S-4
1288 ft/sec
3360 ft/sec
7471 ft/sec
20
10
0
-10
-20
-30
0
20
40
60
80
100 ft



SEISMIC LINE S-5
1235 ft/sec
4094 ft/sec
6157 ft/sec
40
30
20
10
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft



SEISMIC LINE S-6
1307 ft/sec
3637 ft/sec
5458 ft/sec
10
0
−10
−20
−30
0
20
40
60
80
100
120
140 ft


SEISMIC LINE S-7
1213 ft/sec
3677 ft/sec
5104 ft/sec
10
0
−10
−20
−30
0
20
40
60
80
100
120
140 ft

APPENDIX E

WORKSHEET C.4-1: CATEGORIZATION OF INFILTRATION FEASIBILITY

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		**Worksheet C.4-1**	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?** Note that it is not necessary to investigate each and every criterion in the worksheet if infiltration is precluded. Instead a letter of justification from a geotechnical professional familiar with the local conditions substantiating any geotechnical issues will be required.			
Criteria	Screening Question	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: The infiltration rates of the existing soil for locations P-1 through P-4 and P-7 through P-10 based on the on-site infiltration study were calculated to be less than 0.5 inches per hour (P-1=0.03 in/hr; P-2 =0.05 in/hr; P-3=0.06 in/hr, P-4=0.03 in/hr, P-7=0.06 in/hr, P-8=0.34 in/hr, P-9=0.08 in/hr, and P-10= 0.19 inches per hour) after applying a minimum factor of safety (F) of FS=2.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: No. See Criterion 1.			

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: No. See Criterion 1.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: No. See Criterion 1.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	Proceed to Part 2	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by County staff to substantiate findings.

Worksheet C.4-1 Page 3 of 4			
Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria **Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: The infiltration rates of the existing soil for locations P-1 through P-4 and P-7 through P-10 based on the on-site infiltration study were calculated to be less than 0.5 inches per hour (P-1=0.03 in/hr; P-2 =0.05 in/hr; P-3=0.06 in/hr, P-4=0.03 in/hr, P-7=0.06 in/hr, P-8=0.34 in/hr, P-9=0.08 in/hr, and P-10= 0.19 inches per hour) after applying a minimum factor of safety (F) of FS=2. However, infiltration will increase the risk of geotechnical hazards as described below and in NOVA's reporting (NOVA 2022).			
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: C2.1 A geologic investigation was performed at the subject site. C2.2 Settlement due to water infiltration is possible due to the loose fill/colluvium on site. C2.3 Infiltration has the potential to cause slope failures. BMPs are to be sited a minimum of 50 feet away from any slope. This setback may not be achieved due to the limited space at the site. C2.4 BMPs are to be sited a minimum of 10 feet away from all underground utilities. C2.5 Stormwater infiltration can result in damaging ground water mounding during wet periods. The dense tonalite underlying the entire site may be an impermeable unit causing groundwater mounding. C2.6 BMPs are to be sited a minimum of 10 feet away from any foundations or retaining walls. C2.7 Other Factors: It is NOVA's judgment that this site is not suitable for permanent stormwater infiltration BMPs. NOVA recommends lining any BMPs with the required impermeable liner and be limited by siting any such structures away from property lines.			

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: Based on the geotechnical hazards associated with the relatively low infiltration rates, it is NOVA's judgment that infiltration should not be considered at this site.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: Based on the geotechnical hazards associated with the relatively low infiltration rates, it is NOVA's judgment that infiltration should not be considered at this site.			
Part 2 Result*	If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	No Infiltration	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		Worksheet C.4-1	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?** Note that it is not necessary to investigate each and every criterion in the worksheet if infiltration is precluded. Instead a letter of justification from a geotechnical professional familiar with the local conditions substantiating any geotechnical issues will be required.			
Criteria	Screening Question	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: The infiltration rates of the existing soils for locations P-5 and P-6 were calculated to be 1.5 and 1.2 inches per hour, respectively, after applying a minimum factor of safety of F=2. Infiltration rates of greater than 0.5 inches per hour imply that geologic conditions allow for full infiltration; however, infiltration rates at this site are highly variable, due to the degree of weathering of the tonalite under the site, with many of the tests resulting in rates consistent with a No Infiltration Condition. Therefore, based on the widely variable rates across the site, it is NOVA's judgment that more conservative partial infiltration rates should be utilized for design of BMPs in the location of P-5 and P-6.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: No. See Criterion 1.			

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: Based on the geotechnical hazards associated with the high infiltration rates, it is NOVA's judgment that partial infiltration rates be utilized for the design at this site in the locations of P-5 and P-6.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: Based on the geotechnical hazards associated with the high infiltration rates, and the highly variable infiltration rates across the site, it is NOVA's judgment that partial infiltration rates be utilized for the design at this site in the location of P-5 and P-6.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	Proceed to Part 2	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by County staff to substantiate findings.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 3 of 4</th></tr>
<tr><td colspan="4"><u>Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria</u>

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>5</td><td>Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
The infiltration rates of the existing soils for locations P-5 and P-6 were calculated to be 1.5 and 1.2 inches per hour, respectively, after applying a minimum factor of safety of F=2. Infiltration rates of greater than 0.5 inches per hour imply that geologic conditions allow for full infiltration; however, infiltration rates at this site are highly variable, due to the degree of weathering of the tonalite under the site, with many of the tests resulting in rates consistent with a No Infiltration Condition. Therefore, based on the widely variable rates across the site, it is NOVA's judgment that more conservative partial infiltration rates should be utilized for design of BMPs in the location of P-5 and P-6.</td></tr>
<tr><td>6</td><td>Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

C2.1 A geologic investigation was performed at the subject site.
C2.2 Settlement due to water infiltration is possible due to the loose fill/colluvium on site.
C2.3 Infiltration has the potential to cause slope failures. BMPs are to be sited a minimum of 50 feet away from any slope.
C2.4 BMPs are to be sited a minimum of 10 feet away from all underground utilities.
C2.5 Stormwater infiltration can result in damaging ground water mounding during wet periods. The dense tonalite underlying the entire site may be an impermeable unit causing groundwater mounding.
C2.6 BMPs are to be sited a minimum of 10 feet away from any foundations or retaining walls.
C2.7 Other Factors: It is NOVA's judgment that this site design should utilize partial infiltration rates for the locations tested. NOVA recommends lining any BMPs with the required impermeable liner ands be limited by siting any such structures away from property lines.</td></tr>
</table>

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Based on the widely variable infiltration rates across the site, and the geotechnical hazards associated with the high infiltration rates, it is NOVA's judgment that partial infiltration rates be utilized for the design at this site in the locations tested.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Based on the widely variable infiltration rates across the site, and the geotechnical hazards associated with the high infiltration rates, it is NOVA's judgment that partial infiltration rates be utilized for the design at this site in the locations tested.			
Part 2 Result*	If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	Partial Infiltration	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings