Crestview Apartments

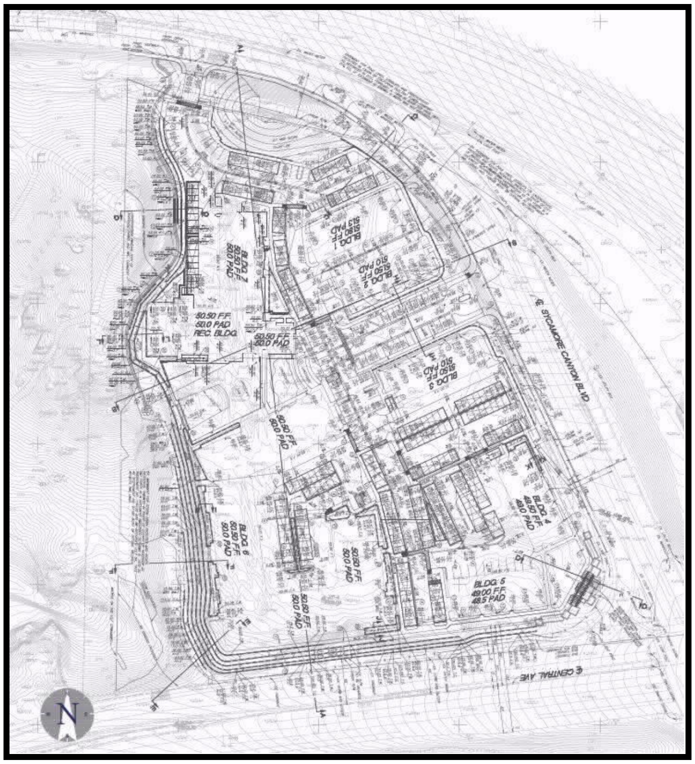
Draft Environmental Impact Report (DEIR)

Appendix F – Geotechnical Evaluation

Report
Update Geotechnical Evaluation

Proposed Crestview Apartment Complex

Sycamore Canyon Boulevard and Central Avenue,
Riverside, California



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NOVA Project 3020003

September 18, 2020

NOVA

GEOTECHNICAL

MATERIALS

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September 18, 2020
NOVA Project 3020003

Subject: Report
Update Geotechnical Evaluation
Proposed Crestview Apartment Complex
Sycamore Canyon and Central Avenue, Riverside, California

NOVA Services, Inc. (NOVA) is pleased to present herewith its update geotechnical evaluation for the proposed Crestview Apartment Complex. The work reported was completed by NOVA for KA Enterprises in accordance with NOVA's proposal dated July 28, 2020, as authorized on August 3, 2020.

Revised site development concepts were provided to NOVA in July 2020. This report updates prior reporting to address development concepts that are current as of this date.

NOVA appreciates the opportunity to be of service to KA Enterprises. Should you have any questions regarding this report or other matters, please do not hesitate to call.

Sincerely,
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REPORT
UPDATE GEOTECHNICAL EVALUATION

Proposed Crestview Apartment Complex
Sycamore Canyon Boulevard and Central Avenue, Riverside, California

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1.0 INTRODUCTION

1.1 Overview

1.1.1 Terms of Reference

The work reported herein was completed by NOVA Services, Inc. (NOVA) for KA Enterprises in accordance with the scope of work detailed in NOVA's proposal dated July 28, 2020, as authorized on August 3, 2020.

The report provides a design level, geotechnical evaluation of requirements for development of an apartment complex proposed to be sited on a vacant 9.44-acre property in Riverside, California. The project is known to NOVA as "Crestview Apartments." The Crestview Apartments property is located at the northwest corner of Sycamore Canyon Boulevard and Central Avenue, Riverside, California (hereinafter, 'the site').

Figure 1-1 depicts the site vicinity.

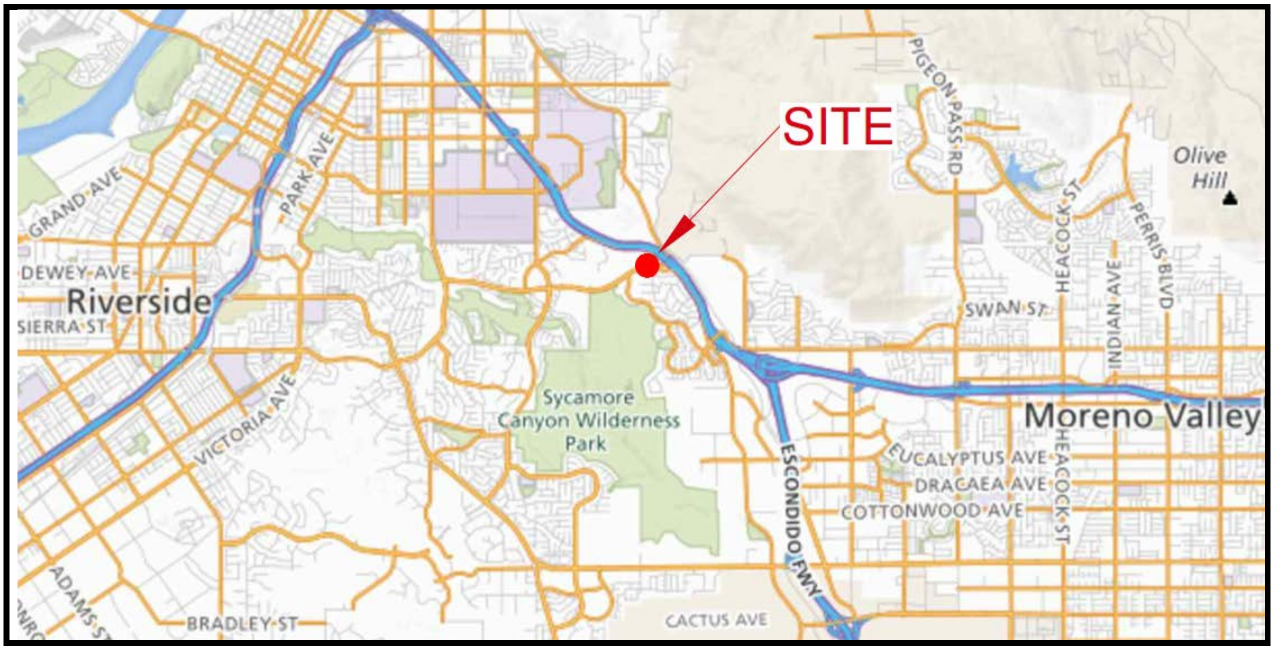

Figure 1-1. Vicinity Map

1.2 Update Report

1.2.1 General

This report updates and supersedes the findings and recommendations of a series of prior geotechnical reporting by NOVA and others. The following subsections abstract that reporting.

1.2.2 Reporting by NOVA

Prior to this report, NOVA provided the series of submittals listed below.

1. NOVA 2018. NOVA provided a pre-acquisition geotechnical review of this site in an April 2018 report entitled *Report, Pre-Acquisition Geotechnical Review of Alternatives for Ground Improvement, Proposed Apartment Complex Property, Sycamore Canyon Boulevard and Central Avenue, Riverside, CA* (NOVA Project 2018015, April 3, 2018, hereinafter, 'NOVA 2018').
2. NOVA 2020a. NOVA provided Preliminary Geotechnical Evaluation of this site in a January 2020 report entitled *Report, Preliminary Geotechnical Evaluation, Proposed Crestview Apartment Complex Property, Sycamore Canyon Boulevard and Central Avenue, Riverside, CA* (NOVA Project 3020003, January 20, 2020, hereinafter, 'NOVA 2020a').
3. NOVA 2020b. NOVA provided Response to Review Comments to NOVA 2020a in a July 2020 submittal (reference, *Response to Review Comments, Preliminary Geotechnical Evaluation, Proposed Crestview Apartment Complex Property, Sycamore Canyon Boulevard and Central Avenue, Riverside, CA*, NOVA Project 3020003, July 21, 2020, hereinafter, 'NOVA 2020b').

Design concepts were incomplete at the time NOVA 2018 and NOVA 2020a were submitted. Those concepts have since become more finalized and certain design codes have changed. This report addresses the current design and building codes. Additionally, the report addresses data gaps from prior subsurface exploration by NOVA and others.

1.2.3 Reporting by Others

This report and prior reporting by NOVA utilize upon the findings of subsurface explorations by others as a basis for understanding subsurface conditions. The reporting by others that has been utilized by NOVA is listed below.

1. AKA 2007. *"Revised" Preliminary Geotechnical Investigation, Proposed Apartment Complex, Tract 34946 (Alexan Cityscape Project), City of Riverside, California,* Albus-Keefe & Associates, Inc., Project Number 1566.00, December 11, 2007 (hereinafter, 'AKA 2007').
2. JRB 1997. AKA 2007 references previous subsurface work for the site performed by John R. Byerly, Inc. dated August 1997 (hereinafter, 'JRB 1997'). Lab testing and test pit logs by JRB 1997 as referenced within AKA 2007 are attached within Appendices B and C of this report.
3. Geocon 2018. *Supplemental Geotechnical Investigation, Crest View, Northwest Corner of Central & Sycamore Canyon, Riverside, California,* Geocon West, Inc., Project Number T2820-22-01, September 11, 2018 (hereinafter, 'Geocon 2018').

Laboratory and subsurface exploration records from the above reporting are attached within Appendices B, C and D.

1.3 Objective, Scope, and Limitations of This Work

1.3.1 Objective

The objective of the work reported herein is to utilize existing subsurface information and develop supplemental subsurface information to finalize a geotechnical investigation for the development depicted by current site planning.

1.3.2 Scope

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

1. Task 1, Pre-Mobilization Activities. Prior to initiating any fieldwork, NOVA will undertake the series of subtasks described below.

    - *Subtask 1-1, Project Document Review*. Review readily available development plans for the project. No structural information was available for review.

    - *Subtask 1-2, Utility Clearance*. Contacted underground service alert (USA) to determine the presence of underground utilities at locations planned for excavations.

    - *Subtask 1-3, Subcontracting*. A California Certified Geophysicist was retained to conduct the seismic surveys. A specialty earthwork subcontractor was retained to excavate test pits required for the subsurface exploration.

2. Task 2, Subsurface Exploration. A NOVA Certified Engineering Geologist directed a subsurface exploration that included the subtasks listed below.

    - *Subtask 2-1, Seismic Traverses.* A seismic survey was performed that included seven (7) approximately 100-foot to 300-foot seismic lines at the site. The data collected was used to provide seismic shear wave (V_s) and compression (p-wave) velocities in the underlying fill, alluvium, and bedrock material. The data was used (i) to characterize the thickness and stiffness / density of the subsurface geologic units; (ii) provide the contractor with data to determine rippability of the tonalite in the northern portion of the site, and (iii) develop a quantitative basis for site classification per Table 20.3-1 within CBC 2019

    - *Subtask 2-2, Exploratory Test Pits*. Excavated exploratory test pits at 18 locations determined by NOVA's engineering geologist to evaluate data gaps from previous subsurface explorations by others. The test pits were excavated up to 15 feet below existing grade. The data objectives were to (i) physically inspect the subsurface, correlating this inspection with the seismic data; and, (ii) to recover soil samples for inspection and laboratory testing.

    - *Subtask 2-3, Closure*. Each test pit was backfilled with uncompacted soil cuttings. NOVA understands that some settlement may occur in the area of the test pit excavations. This uncompacted soil will be densified during site development

3. Task 3, Laboratory Testing. Samples recovered by the work completed as part of Task 2 were returned to NOVA's laboratory for review. Laboratory testing was completed on representative samples to address engineering characteristics.

4. Task 4, Engineering Assessment. The findings of Tasks 1-3 were utilized to support the geotechnical-related assessments abstracted below.

   - *Subtask 4-1, Updated Mapping.* Updated mapping of the occurrence of various units of the subsurface relative to the planned site grades.

   - *Subtask 4-2, Retaining Walls.* Geotechnical-related assessments of alternatives for development of retaining walls, with particular concern for the complex requirements for retaining fill slopes near Building 6.

   - *Subtask 4-3, Ground Improvement.* Update evaluations of requirements for ground improvement for development of pavement subgrades and building pads.

   - *Subtask 4-3, Pavements and Foundations.* Update recommendations for development of pavements and foundations for structures.

5. Task 6, Reporting. Submittal of this report concludes the scope of work described by NOVA's July 28 proposal. The report provides geotechnical-related recommendations for site, structure and infrastructure development. The report includes a record of all work, including logs of test pits, records of laboratory testing and records of the geophysical testing.

1.3.3 Limitations

Assessment of the subsurface in geological and geotechnical engineering is characterized by uncertainty. Opinions relating to environmental, geologic, and geotechnical conditions are based on limited data, such that actual conditions may vary from those encountered at the times and locations where the data are obtained, despite the use of due professional care. The judgments provided in this report are based upon NOVA's understanding of the planned construction, its experience with similar work, and its judgments regarding subsurface conditions indicated by the evaluations of subsurface explorations by others that are described in the report.

Conditions exposed by construction may vary from those disclosed. NOVA should be retained for design review and for surveillance to observe subsurface conditions revealed during construction. NOVA cannot assume responsibility for the recommendations of this report if NOVA does not perform construction observation. Section 10 of this report addresses this consideration in more detail.

This report addresses preliminary geotechnical considerations only. The report does not provide any environmental assessment or investigation of the presence or absence of hazardous or toxic materials in the soil, soil gas, groundwater, or surface water within or beyond the site.

1.4 Understood Use of This Report

NOVA expects that the findings and recommendations provided herein will be utilized by KA Enterprises and its Design Team in decision-making regarding geotechnical-related design and construction.

1.5 Report Organization

The remainder of this report is organized as abstracted below.

- Section 2 reviews available project information.
- Section 3 describes the field exploration and laboratory testing by NOVA.
- Section 4 describes the field exploration and laboratory testing by others.
- Section 5 describes the site physical setting, including the geologic setting and site-specific subsurface conditions.
- Section 6 reviews geologic and soil hazards common to the Riverside region, considering each for its potential to affect the site.
- Section 7 reviews alternatives for large-scale ground improvement, concluding with recommendations for ground improvement by ‘deep dynamic compaction’ (DDC).
- Section 8 provides preliminary recommendations for earthwork and foundation design.
- Section 9 provides preliminary recommendations for pavement design.
- Section 10 provides recommendations for construction observation and testing.
- Section 11 cites the principal references used in preparation of the report.

Figures and tables that amplify the discussions in the text are embedded at their point of reference. Plates providing larger scale view of certain figures and subsurface sections are provided immediately following the text of the report. The report is supported by 8 appendices.

- Appendix A provides guidance regarding use of the geotechnical report.
- Appendix B provides logs of exploratory excavations by NOVA.
- Appendix C provides logs of exploratory excavations by others.
- Appendix D provides records of seismic traverses by NOVA.
- Appendix E provides records of seismic traverses by seismic traverses by others.
- Appendix F provides records of laboratory testing by NOVA.
- Appendix G provides records of laboratory testing by others.
- Appendix H provides records of the seismic shear wave survey by NOVA.

2.0 PROJECT INFORMATION

2.1 Location

The Crestview Apartments site is located on the northwest corner of Sycamore Canyon Boulevard and Central Avenue, Riverside, California. Figure 2-1 depicts the location and approximate limits of the site on a recent aerial image.

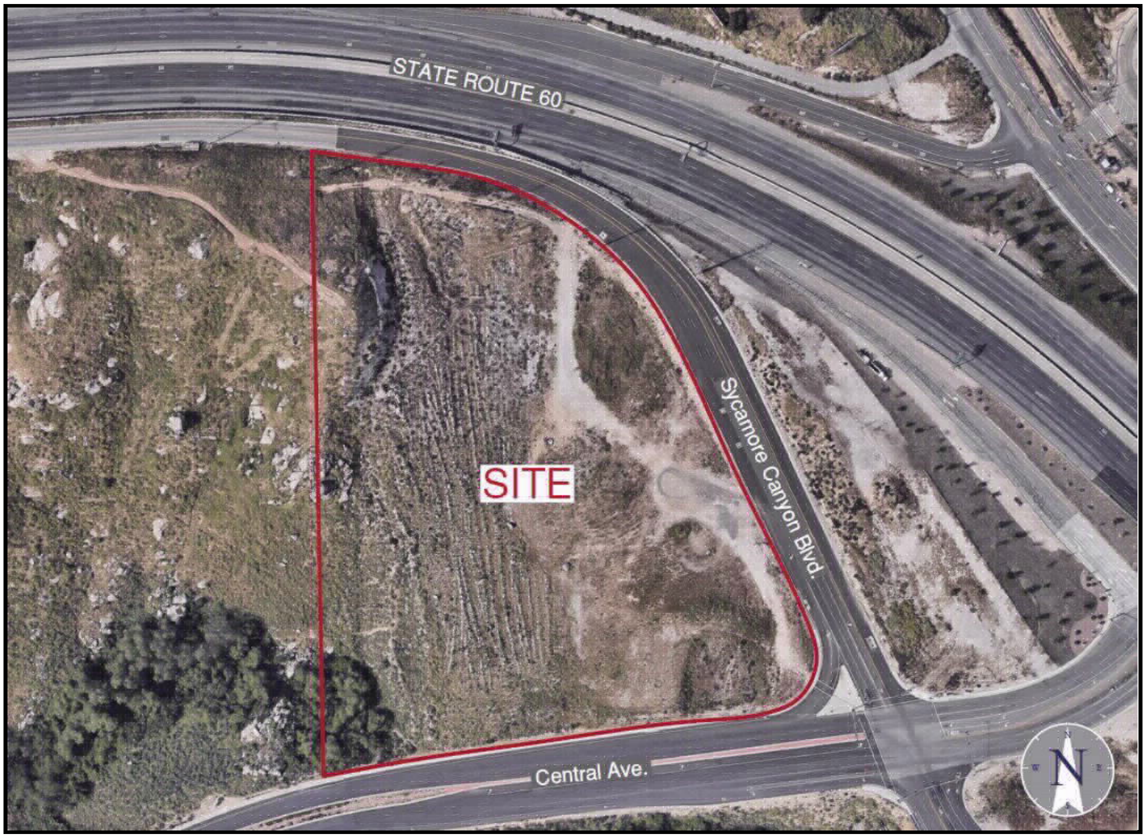

Figure 2-1. Site Limits and Location

2.2 Site Use

2.2.1 Documentation

The site is described by a 2006 Alta Survey (Rick Engineering, November 2006, hereinafter, ‘Rick 2006’).

2.2.2 Current Site Use

The review of previous site documentation (including AKA 2007, JRB 1997 and Geocon 2018), as well as observation of the current surface topography, indicates that the site has been extensively graded. NOVA is not aware whether the previous earthwork or grading at the site required permitting.

The ground surface descends in elevation from northeast to southwest, from about El +1,375 feet msl to El +1,315 feet msl. This grade occurs over a distance of about 750 feet, at a surface gradient averaging about 7%.

The site is lightly vegetated, with areas of bare soil and windrows of rocks and boulders. An approximately 38-foot high cut slope exposing granitic bedrock is located in the northwest corner of the property.

2.2.3 Historic Site Use

Review of aerial photography indicates that the site was utilized for construction staging operations and grading in 2005 through 2006, accommodating the realignment of Sycamore Canyon Boulevard. Previous to this time, the site was a hilltop cut by two southwest-trending drainage features.

Figure 2-2 depicts the site area in 1994 prior to this realignment. Figure 2-3 (following page) shows the site area in 2005, during realignment of Sycamore Canyon Boulevard.

Figure 2-2. Site Area, May 1994

Figure 2-3. Site Area, October 2005

2.3 Planned Apartment Complex

2.3.1 Reference Documentation

NOVA's understanding of the planned development is based upon review of the preliminary design architectural and civil documentation described below.

- *Conceptual Site Plan, Crestview Apartments,* Architects Orange, undated.
- *Preliminary Grading Plans, Crestview Apartments,* SDH and Associates, Inc., May 2020 (hereinafter 'SDH 2020').

2.3.2 Architectural

Documentation by Architects Orange indicates that the apartment complex will consist of seven (7) new apartment structures. Five structures will be 3-level apartments of Type I construction; a single Type II, 2-4 split story structure will be developed; and, a single Type III, 4-story building will be developed. In aggregate, the seven structures will provide 237 dwelling units, supported by 428 parking spaces.

Development plans also include a recreational building, an outdoor swimming pool and spa, a putting green, paved driveways and parking stalls, and typical hardscapes.

2.3.3 Structural

Structural design for the apartment complex development has not yet begun.

As is noted above, the seven residential apartment buildings will rise two to four levels (to approximately 49 feet) above surrounding ground level (SDH 2020).

It is expected that the four-level structure will be developed with wood framed construction and masonry bearing walls (a ‘brick-and-joist structure’). Foundation loads will be relatively light, with interior column loads less than about 300 kips and wall loads less than about 4 kips/lineal foot.

2.3.4 Earthwork

NOVA’s understanding of earthwork required for development of the apartment complex is based upon review of SDH 2020. Figure 2-4 (following page) provides a graphic depicting the planned development.

SDH 2020 indicates it is expected that significant earthwork will be required for site preparation and to create level pads for the structures. Associated with planning for large scale ground improvement. Current planning is to effect ground improvement by deep dynamic compaction (‘DDC’). Anticipating the use of DDC, NOVA expects that grading will be performed in three identifiable ‘phases’, as is abstracted below.

(i) Phase 1 grading will consist of site preparation, leveling areas for preparation for ground improvement by DDC.

(ii) Phase 2 rough grading will follow DDC operations, taking the surfaces from ground improvement elevations to near design grades as indicated on the Civil grading plans.

(iii) Phase 3 will complete precise grading of building pads and other areas supporting new improvements including surface drainage within appropriate tolerances.

2.3.5 Stormwater

Planning for permanent stormwater infiltration Best Management Practices (‘stormwater BMPs’) has been conceptually developed for the site. Based upon review of the WQMP (TRW 2020), NOVA understands that the permanent stormwater BMPs will include a variety of biofiltration solutions set throughout the development.

Subsurface exploration indicates that the existing fill locally extends to as much as ±34 feet in depth. Permanent stormwater BMP’s located in fill will be required to be designed with a ‘no infiltration’ condition.

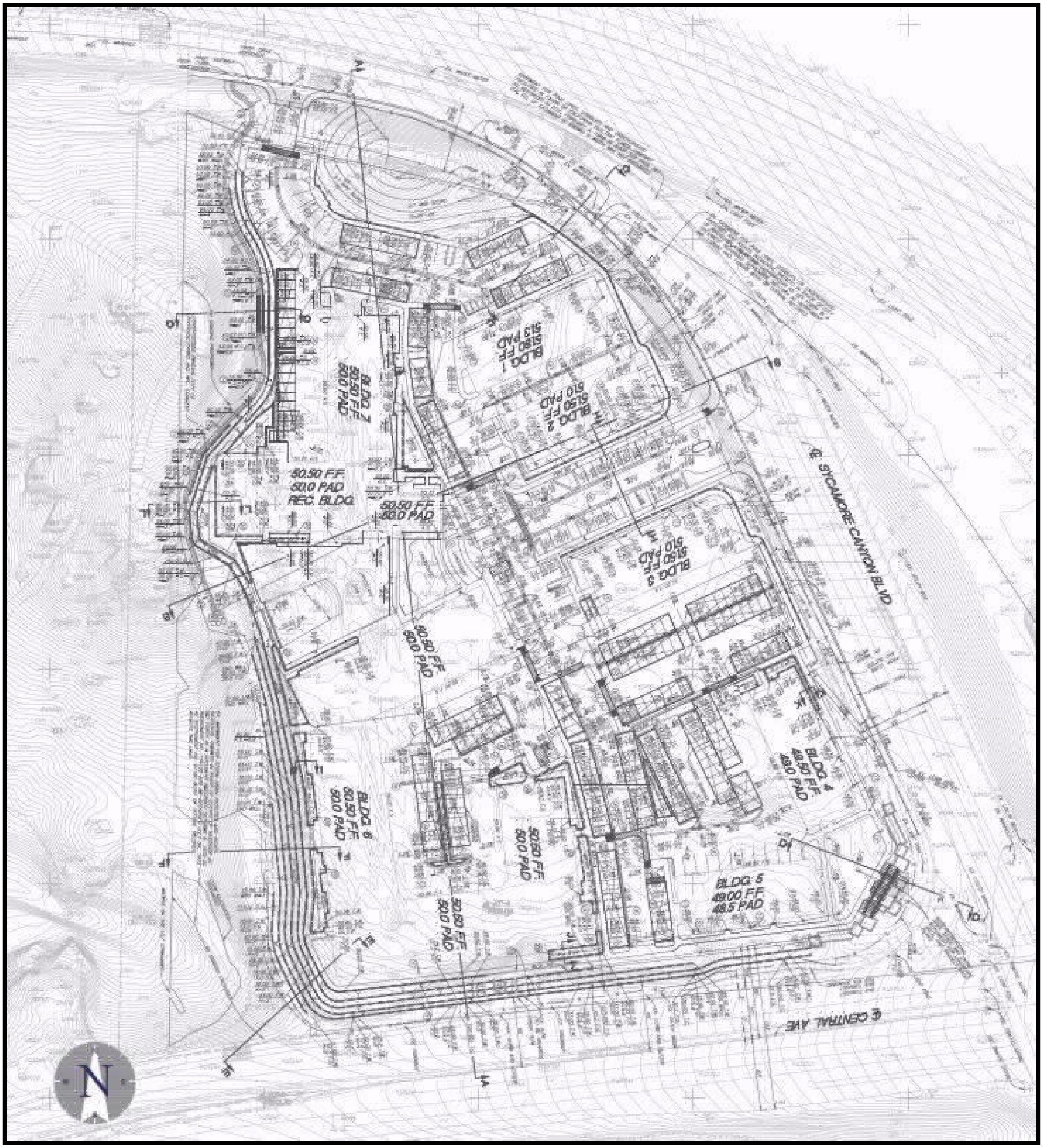

Figure 2-4. Conceptual Grading Plan
(source: SDH 2020)

3.0 SITE CHARACTERIZATION BY NOVA

3.1 Overview

3.1.1 Subsurface Exploration

The subsurface by NOVA was completed as two separate tasks in July 2020, as described below.

- Task 1, Exploratory Test Pits. Eighteen (18) test pits were completed to addresses data gaps from previous geotechnical investigations

- Task 2, Seismic Survey. A seismic survey was performed that included seven (7) approximately 100-foot to 300-foot seismic lines at the site. The data collected was used to provide seismic shear wave (V_s) and compression (p-wave) velocities in the underlying fill, alluvium, and bedrock material. The data was used (i) to characterize the thickness and stiffness / density of the subsurface geologic units; (ii) provide the contractor with data to determine rippability of the tonalite in the northern portion of the site, and (iii) develop a quantitative basis for site classification per Table 20.3-1 within CBC 2019.

Figure 3-1 and Figure 3-2 depict the field work.

Figure 3-1. Task 1, Exploratory Test Pits, July 2020

Figure 3-2. Task 2, Seismic Survey, July 2020

Figure 3-3 (following page) presents a plan view of the site indicating the locations of eighteen (18) exploratory test pits, six (6) seismic refraction survey lines, and one (1) seismic shear wave line. Plate 1 Subsurface Investigation Map, located at the end of this report, provides this figure in larger format and scaled.

The exploratory test pits (referenced as TP-1 through TP-18) and the seismic traverses (referenced as S-1 through S-6 and SW-1) were performed by specialty subcontractors retained by NOVA.

All work was completed under the surveillance of a NOVA engineering geologist. The test pits were backfilled with cuttings of excavated soil to match the existing ground surface prior to leaving the site.

3.1.2 Laboratory Testing

Soil samples recovered from the test pits 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.

Laboratory testing included both index and strength testing. Like the subsurface exploration, the laboratory testing was directed toward addressing data gaps from prior site characterization.

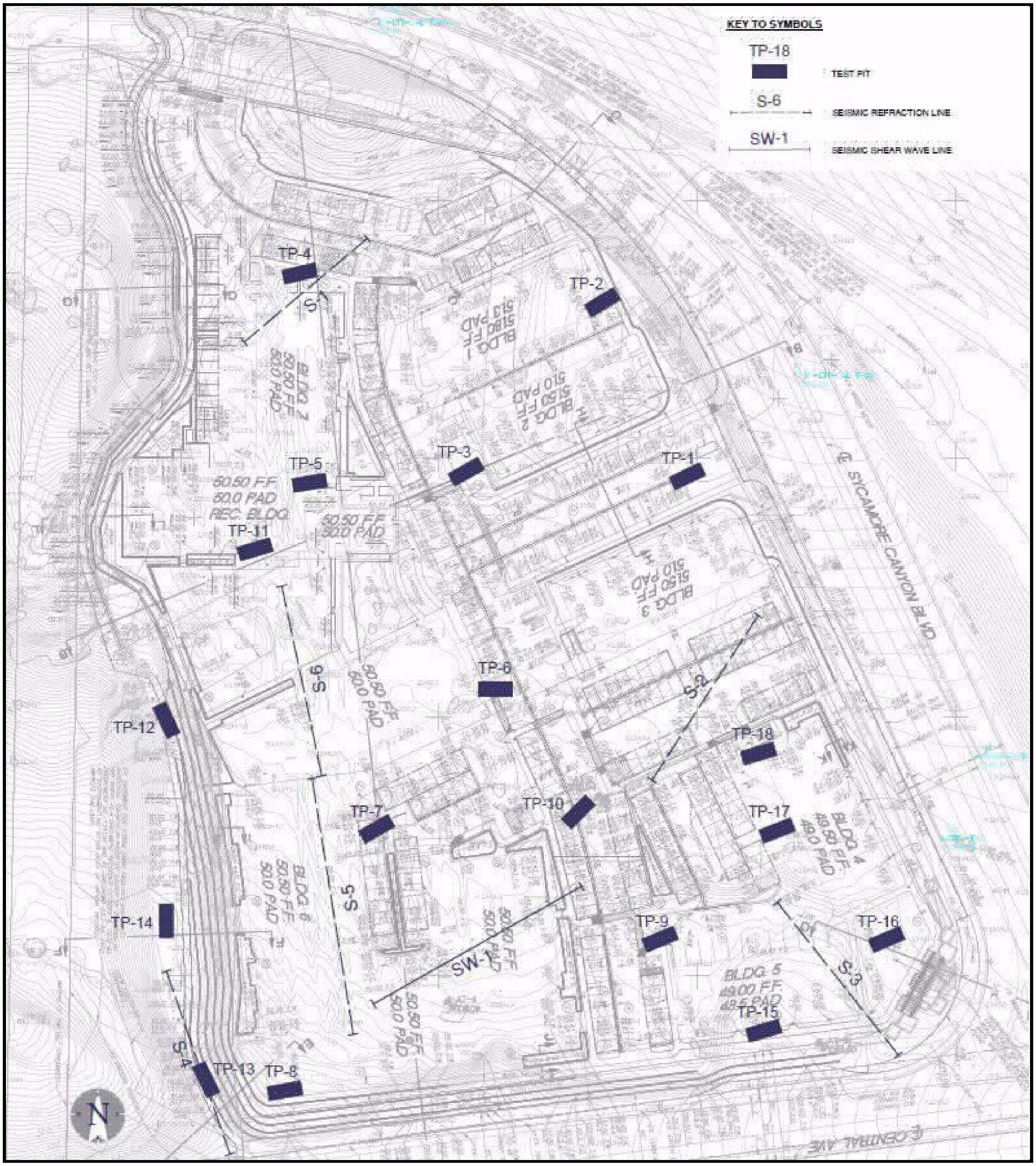

Figure 3-3. Subsurface Exploration by NOVA

3.2 Exploratory Test Pits

The exploratory test pits were completed utilizing a CAT 320 backhoe. The test pits were excavated to depths as great as 15 feet bgs and logged and sampled by the NOVA geologist.

Table 3-1 abstracts the exploratory test pits. Graphical representation of these logs is presented in Appendix B. Figure 3-4 (following page) depicts an excavated test pit.

Table 3-1. Abstract of the Exploratory Test Pits

Test Pit	Approximate Ground Surface Elevation (feet, msl)	Total Depth Below Ground Surface (feet)	Terminal Elevation (feet, msl)	Terminated in Rock ?	Thickness of Fill and Alluvium (feet)
TP-1	1,350	4	1,346	Yes	0
TP-2	1,349	13	1,336	No	>13
TP-3	1,351	8.5	1,343	Yes	3
TP-4	1,355	3	1,352	Yes	0
TP-5	1,354	9.5	1,345	Yes	6
TP-6	1,350	3	1,347	Yes	2
TP-7	1,345	15	1,330	No	>15
TP-8	1,329	13	1,316	No	>15
TP-9	1,341	9	1,332	No	>9
TP-10	1,346	4	1,342	Yes	2
TP-11	1,351	13	1,338	No	>13
TP-12	1,335	13	1,322	No	>13
TP-13	1,313	8	1,305	No	>8
TP-14	1,315	12	1,303	No	>12
TP-15	1,332	6	1,326	No	>6
TP-16	1,345	15	1,330	No	>15
TP-17	1,351	15	1,336	Yes	15
TP-18	1,348	6	1,342	No	4

Notes: 1. ‘‘>’ indicates ‘deeper than’
2. No groundwater was encountered

Figure 3-4. Test Pit, TP-1, Exposing Val Verde Tonalite

3.3 Seismic Survey

3.3.1 General

A seismic survey consisting of six (6) seismic refraction lines and one (1) seismic shear wave line was performed on August 19, 2020 by a California-certified Professional Geophysicist (PGP). The purpose of the survey was to collect data to provide seismic shear wave (V_s) and compression (p-wave) velocities in the underlying fill, alluvium, and bedrock material. The data was used (i) to characterize the thickness and stiffness / density of the subsurface geologic units; (ii) provide the contractor with data to determine rippability of the tonalite in the northern portion of the site, and (iii) develop a quantitative basis for site classification per Table 20.3-1 within CBC 2019. The seismic survey reports are attached at the end of this text in Appendices D and H.

3.3.2 Seismic Refraction Survey (“P”-Wave)

The seismic survey of the subject site included six seismic refraction survey lines (S-1 through S-6), ranging from approximately 125 to 200 feet in length. The seismic refraction lines were utilized to assess the general seismic velocity characteristics of the underlying bedrock materials with regards to rippability during grading and determine the depth of bedrock underneath the existing artificial fill and alluvial deposits.

The approximate locations of the seismic refraction survey lines are shown on Figure 3-3 and Plates 1, the Geotechnical Map. A 24-channel Geometrics StrataVisor NZXP model signal-enhancement refraction seismograph was used in conjunction with 24 14-Hz geophones spaced at regular intervals. Striking a 16-pound sledge hammer on steel plates at seven locations along each survey traverse created seismic “P”-waves. As the “P”-waves travel through the subsurface, they refract off of the contacts of subsurface materials with different velocities and travel back to the surface. The arrival times of the “P”-waves are recorded on the seismograph. Figure 3-5 presents seismic refraction geometry.

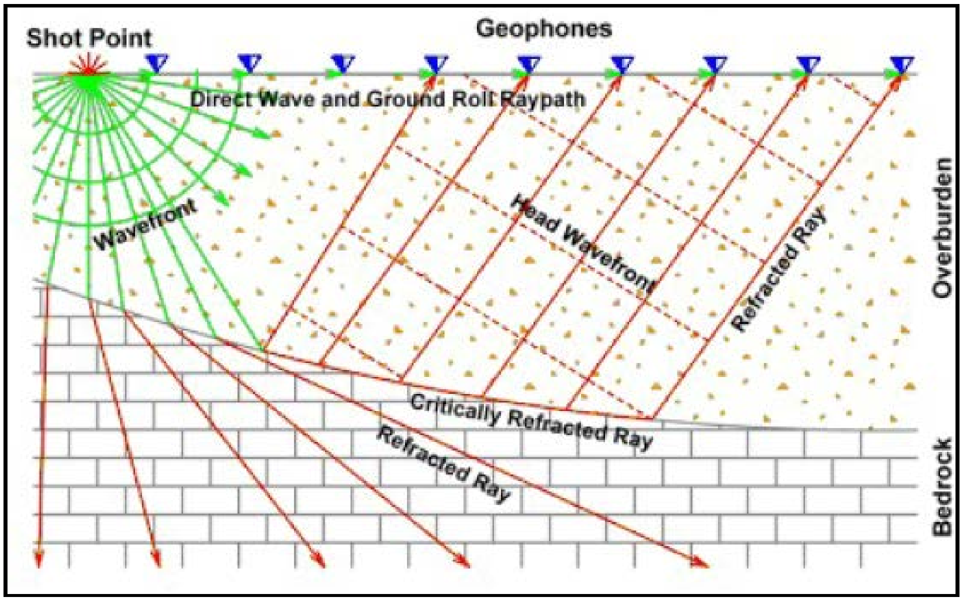

Figure 3-5. Seismic Refraction Geometry

After the field data was collected, the geophysicist analyzed the data using specialized software specific to this purpose. The line lengths ranged from 125 to 200 feet in length which resulted in a maximum obtainable depth of approximately 25 to 50 bgs. Section 8.4 will further discuss the seismic refraction data relative to rippability. Appendix C contains the Seismic Refraction Survey Report. Figure 3-6 presents the layer velocity model of S-1.

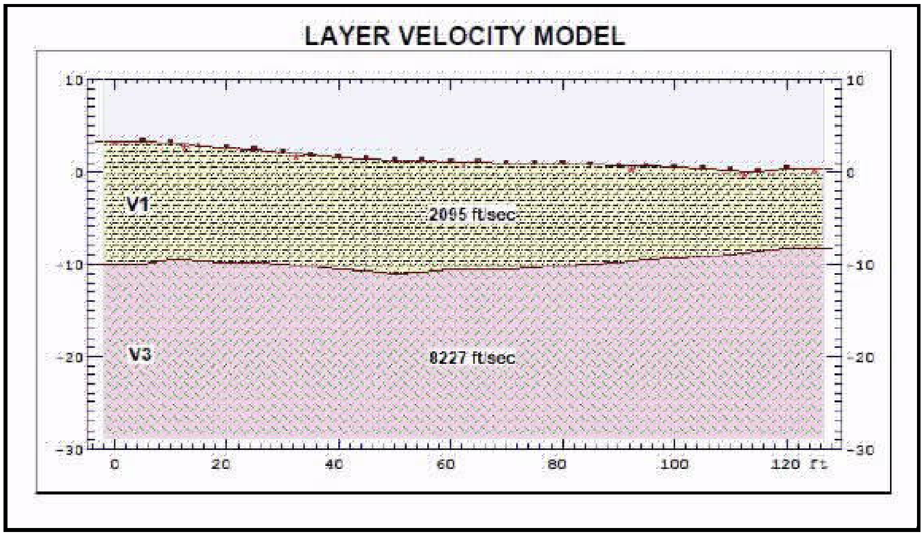

Figure 3-6. Layer Velocity Model of S-1

3.3.3 Seismic Shear Wave Survey (V_S)

The seismic survey of the subject site included one seismic shear wave survey line (SW-1), approximately 184 feet in length. The approximate location is shown on Figure 3-3 and Plate 1, the Geotechnical Map. A 24-channel Geometrics StrataVisor NZXP model signal-enhancement refraction seismograph was used in conjunction with 24 4.5-Hz geophones spaced at regular intervals. For the MASW survey, two seismic records were obtained by multiple hammer strikes of a 16-pound sledge hammer on steel plates positioned 25 feet from the end of each terminus of the seismic line. Vibrations were recorded using a one second record length at a sampling rate of 0.5 milliseconds. The MAM survey records vibrations from background and ambient noise. The ground vibrations were recorded using 32-second record length at 2-milisecond sampling rate with 30 separate records obtained for quality control purposes.

After the field data was collected, the geophysicist combined the MASW and MAM survey results using specialized software specific to this purpose. The weighted average for velocity in the upper 100 feet of the site (referred to as V_{100} or V_s30) was computed from ASCE 7-16 Equation 20.4-1. The seismic model indicates that the average shear wave velocity (weighted average) in the upper 100 feet is 2109.1 feet/sec. This average velocity classifies the underlying soils as Site Class C. Figure 3-7 presents the results of the shear wave analysis.

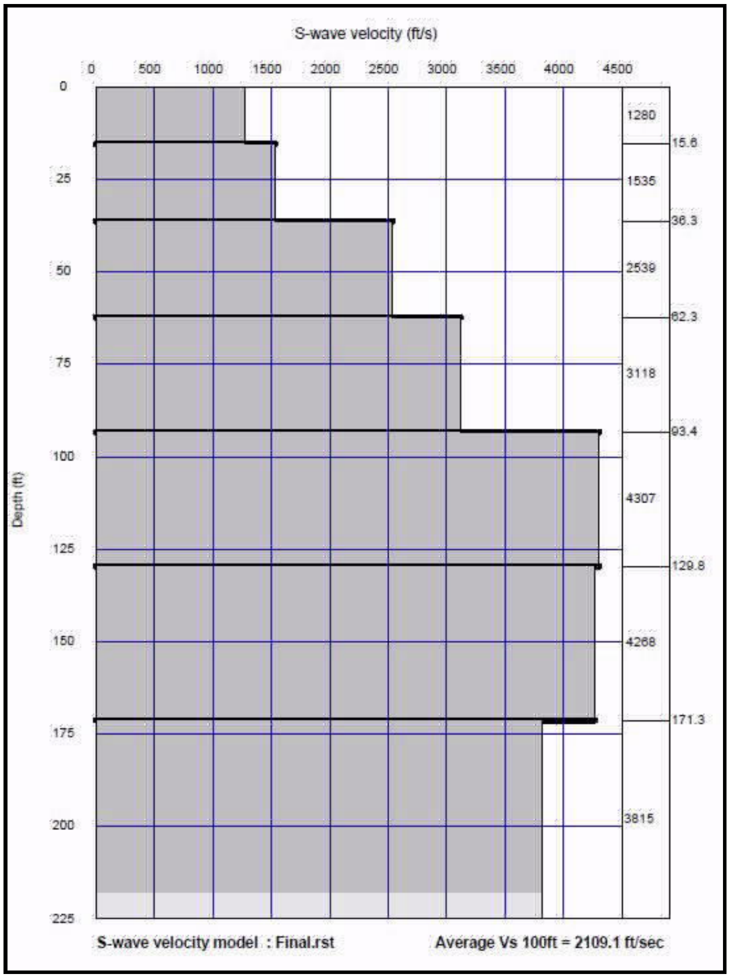

Figure 3-7. Shear Wave Model

3.4 Laboratory Testing

3.4.1 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 well as index testing in general accordance with ASTM standards. Additional representative samples were delivered to a specialty testing laboratory for determination of the potential for soils to corrode/attack embedded metals and concrete. Records of the laboratory testing are provided in Appendix D.

3.4.2 Soil Gradation

The visual classifications were further evaluated by grain size testing. Table 3-2 provides an abstract of this testing.

Table 3-2. Abstract of the Soil Gradation Testing

Sample Ref		**Percent Passing the #200 Sieve**	**Classification after ASTM D2488**
Test Pit	**Depth (feet)**		
TP-1	0 to 5	6	SP-SM
TP-2	5 to 8	29	SC
TP-7	0 to 5	17	SM
TP-11	5 to 7	9	SP-SM

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

3.4.3 Maxium Density and Optimum Moisture

Two (2) bulk samples of the near-surface soils were tested to determine maximum density and optimum moisture content, ASTM D1557 (the 'modified Proctor'). Table 3-3 abstracts this testing.

Table 3-3. Abstract of the Maximum Density and Optimum Moisture Testing

Test Pit	**Depth (feet)**	**Soil Description**	**Optimum Moisture (w, %)**	**Optimum Dry Density (γ_{dry}, lb/ft^3)**
TP-1	0 to 5	Sand to Silty Sand	8.8	127.4
TP-7	0 to 5	Silty Sand to Sand	9.7	132.5

3.4.4 Moisture Content

Soil samples were tested to determine moisture content. Table 3-4 abstracts the results of this testing.

Table 3-4. Abstract of the Moisture Content Testing

Sample Ref		Moisture Content (%)
Test Pit	**Depth (feet)**	
TP-1	0 to 5	2.0
TP-2	5 to 8	3.9
TP-7	0 to 5	4.5
TP-11	5 to 7	3.2

3.4.5 Remolded Shear

Direct shear tests were performed on two samples, which were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plots are presented in Appendix D.

3.4.6 R-Value

The purpose of this test is to determine the suitability of prospective subgrade soils and road aggregates for use in the pavement sections of roadways. The test is used by Caltrans for pavement design, replacing the California Bearing Ratio (CBR) test.

The Resistance Value (R-value) test is a material stiffness test, demonstrating a material's resistance to deformation as a function of the ratio of transmitted lateral pressure to applied vertical pressure. A saturated cylindrical soil sample is placed in a Hveem Stabilometer device and then compressed. The stabilometer measures the horizontal pressure that is produced while the specimen is under compression.

Samples representative of the subgrade soils at each phase of construction were selected for this testing and tested in NOVA's laboratory after ASTM D2844. Table 3-5 abstracts the testing.

Table 3-5 Abstract of the R-Value Testing

Sample Ref		R-Value
Test Pit	**Depth (feet)**	
TP-7	0 to 5	70

The R-value listed above are characteristic of granular non-cohesive soils and suggestive of high-quality subgrade material. Design for pavements should anticipate the listed R-values for the corresponding phase.

3.4.7 Corrosivity and Sulfate Attack

Resistivity, pH, soluble sulfates and chloride content of two (2) soil samples were determined as a basis for estimating the potential for the soils to corrode embedded metals and for sulfate attack to embedded concrete.

Section 8.3 provides discussion regarding the corrosion potential for metals and concrete embedded in the site soils. Records of the corrosivity testing are provided in Appendix D. Table 3-6 abstracts the testing.

Table 3-6. Summary of Corrosivity Testing of the Near Surface Soil

Location	Depth (feet)	pH	Resistivity (Ohm-cm)	Sulfate Content		Chloride Content	
				ppm	%	ppm	%
TP-1	0 to 5	8.0	6,700	15	0.002	11	0.001
TP-7	0 to 5	8.2	2,600	96	0.010	21	0.002

4.0 SITE CHARACTERIZATION BY OTHERS

4.1 Overview

AKA 2007 and Geocon 2018 reported the findings of subsurface explorations conducted in November 2006 and June 2018, respectively. The scope of this work is abstracted below.

- Test Pits. JRB 1997 includes seven (7) test pits excavated to a maximum depth of 15.5 feet bgs. AKA 2007 reports eleven (11) test pits excavated to 33 feet bgs. Geocon 2018 reports eighteen (18) test pits excavated to a maximum depth of 34 feet bgs.

- Geophysical. AKA 2007 performed five seismic refraction wave traverses throughout the site.

Figure 4-1 locates the test pits and seismic lines by others on a plan view of the proposed development. The exploratory locations by others is shown on Plate 1, the Geotechnical Map, following the text of this report.

4.2 Test Pits

4.2.1 JRB 1997

Seven (7) test pits were referenced by AKA 2007 were excavated at the site. Table 4-2 abstracts the indications of the test pits. Elevations of the ground surface at the test pit locations are not provided in JRB 1997.

Table 4-2. Abstract of the Test Pits (JRB 1997)

Test Pit	Total Depth Below Ground Surface (feet)	Terminated in Rock ?	Thickness of Fill and Alluvium (feet)
T-1	10.5	Yes	7.5
T-2	13.5	Yes	13
T-3	16	Yes	15
T-4	16.5	Yes	15.5
T-5	16	Yes	15
T-6	8	No	> 8
T-7	8	Yes	7.5

Figure 4-1. Locations of the Previous Site Explorations

4.2.2 AKA 2007

Eleven (11) test pits reported in AKA 2007 were excavated utilizing a track-mounted long stick backhoe and logged by an AKA geologist.

Table 4-2 (following page) abstracts the indications of the test pits. Elevations of the ground surface at the test pit locations are not provided in AKA 2007. Elevations are estimated by NOVA.

Table 4-2. Abstract of the Test Pits (AKA 2007)

Test Pit	Approximate Ground Surface Elevation (feet, msl)	Total Depth Below Ground Surface (feet)	Terminal Elevation (feet, msl)	Terminated in Rock ?	Thickness of Fill and Alluvium (feet)
T-1	1,355	16	1,339	Yes	16
T-2	1,352	21	1,331	Yes	19
T-3	1,350	26	1,352	Yes	24
T-4	1,351	4	1,351	Yes	0
T-5	1,350	33	> 1,317	No	> 33
T-6	1,349	10	1,339	Yes	0
T-7	1,352	28	> 1,324	No	> 28
T-8	1,340	19	1,321	Yes	18
T-9	1,340	18	> 1,322	No	> 18
T-10	1,345	21	1,324	Yes	20
T-11	1,340	20	> 1,320	No	> 20

Notes:
1. '>' indicates 'deeper than'
2. Table developed by review of AKA 2007

4.2.3 Geocon 2018

Eighteen (18) test pits reported in Geocon 2018 were excavated utilizing a track-mounted excavator and logged by a geologist. Table 4-3 (following page) abstracts the indications of these test pits.

4.3 Seismic Refraction Survey

AKA 2007 reports the findings of five (5) seismic refraction lines that were completed by a specialty subcontractor as a part of the field exploration.

The refraction data provided in AKA 2007 show the variation of subsurface 'P-waves' (i.e., compressional waves) with horizontal distance and depth. P-waves travel through rock in a manner analogous to sound waves traveling through air. The speed by which a P-wave propagates through the subsurface depends on the physical properties (i.e. stiffness, density, saturation) and degree of homogeneity. Higher P-wave velocities, $P \geq 5,000$ feet/sec, are indicative of rock.

Seismic refraction traverses provide some indication of the occurrence of the rock surface along the traverses investigated. Though difficult to interpret (and provided in the report without interpretation), the traverses appear to support the findings of the test pits, broadly indicating depths to rock.

Table 4-3. Abstract of the Test Pits (Geocon 2018)

Test Pit	Approximate Ground Surface Elevation (feet, msl)	Total Depth Below Ground Surface (feet)	Terminal Elevation (feet, msl)	Terminated in Rock ?	Thickness of Fill and Alluvium (feet)
T-1	1,347	31.5	1,315.5	Yes	26
T-2	1,341	34	1,307	Yes	34
T-3	1,344	27	1,317	Yes	27
T-4	1,339	31	1,308	Yes	31
T-5	1,315	24	1,291	Yes	22
T-6	1,350	34	1,316	Yes	34
T-7	1,350	25	1,325	No	> 25
T-8	1,352	22	1,330	Yes	20
T-9	1,351	12	1,339	Yes	3
T-10	1,352	23	1,329	Yes	21.5
T-11	1,356	7	1,349	Yes	3
T-12	1,364	5	1,359	Yes	< 1
T-13	1,369	3	1,366	Yes	0
T-14	1,366	6.5	1,359.5	Yes	0
T-15	1,359	1	1,358	Yes	0
T-16	1,356	22	1,334	Yes	18
T-17	1,358	4.5	1,353.5	Yes	1
T-18	1,356	2.5	1,353.5	Yes	1.5

Notes:
1. ‘‘>’ indicates ‘deeper than’
2. Table developed by review of Geocon 2018

4.4 Laboratory Testing by Others

4.4.1 General

AKA 2007 reports limited scope laboratory testing of soil samples recovered from the exploratory test pits. Records of this testing are provided in Appendix A.

4.4.2 Moisture-Density

Moisture-density testing after ASTM D1557 Method A (the ‘Modified Proctor’) was undertaken to project the behavior of the soil when used in earthwork. Testing of a composite bulk sample of sandy soil recovered from near surface indicated an optimum dry density (γ_{dmax}) of γ_{dmax} = 131 lb/ft^3 at an optimum moisture content (w_{opt}) of w_{opt} = 7 %.

4.4.3 Direct Shear

A representative sample from the surface soils was tested in direct shear after ASTM D3080. Table 3-3 abstracts the indications of this testing.

Table 4-4. Abstract of the Direct Shear Testing

Sample Reference		Apparent Cohesion (c, psf)	Angle of Internal Friction (ϕ, degrees)
Sample	Depth (feet)		
S-2	0’ – 1’	300	31

4.4.4 Chemical Testing

Resistivity, sulfate content and chloride content testing of a representative sample of the near-surface soils was used to address the potential for the soils to corrode unprotected metals and the potential for sulfate attack to embedded concrete. Table 3-4 abstracts the chemical testing. Indications of this testing are discussed in more detail in Section 7.3.

Table 4-5. Abstract of Chemical Testing

Sample Ref		pH	Resistivity (Ω-cm)	Sulfates		Chlorides	
Sample	Depth (feet)			ppm	%	ppm	%
S-2	0 - 1	7.6	2,400	10	0.001	42.5	0.004

4.4.5 Expansion Index

The expansion index (EI) of selected materials was evaluated in general accordance with ASTM D4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (±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.

Sample S-2 from the upper 1 foot of the surface soils was tested and found to have EI = 1. This low EI is expected from the sandy soils at this site.

5.0 SITE CONDITIONS

5.1 Geologic Setting

The project area is located within in the southern part of the Los Angeles Basin in the Peninsular Ranges Geomorphic Province. The Peninsular Ranges Geomorphic Province is a series of mountain ranges separated by northwest-trending valleys, which characterizes the southwest portion of California. This geomorphic province encompasses an area that extends about 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 surface topography is characterized geomorphically by eroded and dissected mesa terrain (CGS 2002).

The site itself is set in an area of widely varying topography with Cretaceous-aged, Val Verde Tonalite occurring at the near surface (USGS 2001). Alluvial soils locally infill some scattered lower lying areas.

Figure 5-1 reproduces geologic mapping of the site vicinity.

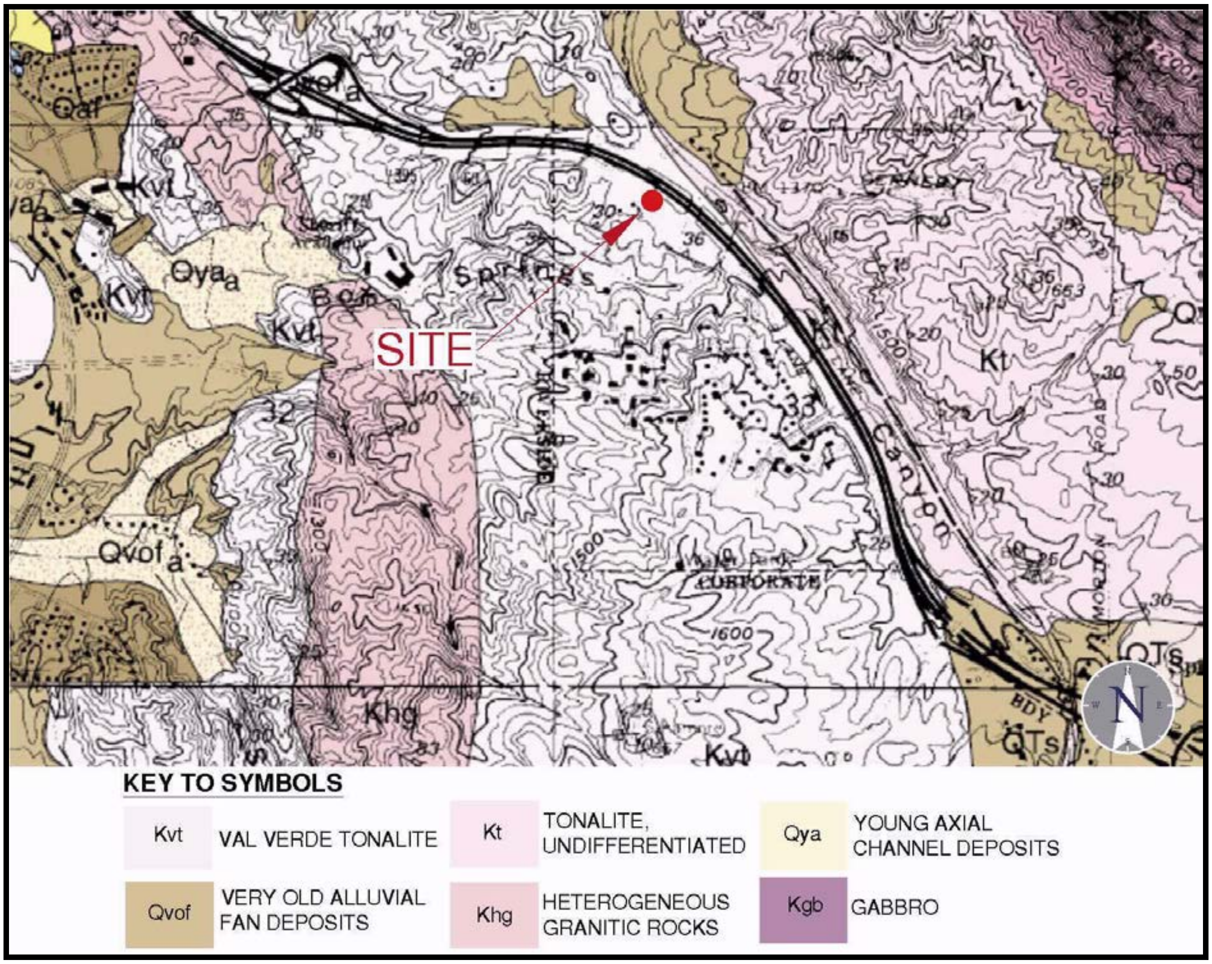

Figure 5-1. Geologic Mapping of the Site Vicinity

5.2 Surface, Subsurface and Groundwater

5.2.1 Surface

The ground surface descends in elevation from northeast to southwest, from about El +1,375 feet msl to El +1,315 feet msl. This grade occurs over a distance of about 750 feet, at a surface gradient averaging about 7%.

As may be seen by review of Figure 2-1, the site is lightly vegetated, with areas of bare soil and windrows of rocks and boulders. An approximately 38-foot high cut slope exposing granitic bedrock is located in the northwest corner of the property.

Figure 5-2 depicts current surface conditions.

Figure 5-2. Surface Conditions, Looking NW Near Sycamore Canyon Boulevard and Central Avenue Intersection

5.2.2 Subsurface

For the purposes of this report, the subsurface may be generalized to occur as the sequence of soils described below. The limits of the subsurface units are presented on the Geotechnical Map and Geotechnical Cross-Sections presented at the end of this text as Plates 1 through 3.

1. <u>Unit 1, Undocumented Fill (map symbol – afu)</u>. Much of the site is covered with undocumented fill consisting of dry to slightly moist silty sand to sand with abundant cobbles and boulders. The fill ranges in thickness from a few feet to greater than 35 feet in the southwestern portion of the site.

   The fill is heterogeneous, widely varying in quality and consistency over short horizontal and vertical distances. Some wood was encountered in test pits, though it appears that the amounts of wood or other organics is not of a scale that would affect the mechanical behavior of the fill. Boulders up to 11 feet in maximum dimension were commonly encountered during the subsurface investigations.

2. <u>Unit 2, Quaternary Alluvial Deposits (map symbol – Qal)</u>. Quaternary Alluvial Deposits were encountered below the fill in the central and southern portions of the site. This unit was deposited in old drainage channels that have since been filled in with the artificial fil. The alluvial deposits generally consisted of slightly moist to moist clayey sand to sand with clay. The alluvial deposits encountered by NOVA and others ranged in thickness from about two feet to thirteen feet thick.

3. <u>Unit 3, Cretaceous Val Verde Tonalite (map symbol – Kvt)</u>. The entire site is underlain by Cretaceous-aged Val Verde Tonalite, a type of plutonic rock. Tonalite was encountered across the site from the surface, at the northern portion of the site, to depths up to 26 feet below ground surface in the central and southern portions of the site. The upper few feet of this unit are generally weathered, and excavated to sand and cobbles with larger scale construction equipment. Below the upper few feet of this unit, this rock is sound and less weathered and may be difficult to excavate with heavy equipment.

5.2.3 Groundwater

Groundwater was not encountered to the maximum depths explored of about 34 feet below ground surface in the test pits by NOVA or others.

6.0 REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

6.1 Overview

This section provides a review of geologic, soil and siting-related hazards common to this region of California, considering each for its potential to affect the planned apartment buildings and other structures. The primary hazards identified by this review are abstracted below.

1. Seismic. The site is at risk for moderate-to-severe ground shaking in response to large-magnitude earthquakes during the lifetime of the planned development.

2. Undocumented Fill. As is discussed in Section 3 and Section 4, much of the site is mantled by undocumented fill that ranges to about 34 feet in thickness. The fill is predominately sandy, but includes gravel, cobble and boulder-sized rock. The boulder-sized rock ranges to 11 feet in size. Unmanaged by design, this fill has the potential to effect damaging total and differential settlements to structures and infrastructure.

   As is discussed in this section and in Section 6 and Sections 7, this risk will be managed by large-scale ground improvement.

The following subsections describe NOVA's review of geologic, soil and siting hazards.

6.2 Geologic Hazards

6.2.1 Strong Ground Motion

The site will be subject to relatively intensive ground shaking resulting from earthquakes sourced from any of several major faults in the area. Faults in the site vicinity are capable of generating large magnitude seismic events.

The peak ground acceleration adjusted for site effects (PGA_M) for the risk-targeted Maximum Considered Earthquake (MCE_R) is PGA_M ~ 0.6 g.

6.2.2 Faulting and Surface Rupture

California is known to contain active faults that can potentially cause significant damage during earthquakes. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the construction of urban developments across the trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements. Special Publication 42 was most recently revised in 2018 (CGS 2018). According to the State Geologist, an "active" fault is defined as one which has had surface displacement within Holocene time (roughly the last 11,700 years). Regulatory Earthquake Fault Zones have been delineated to encompass traces of known, Holocene-active faults to address hazards associated with surface fault rupture within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering geologists can identify the locations of active faults and recommend setbacks from locations of possible surface fault rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during NOVA's site evaluation or during investigations by others. In addition, the site is not located within a fault zone as designated by the County of Riverside (County of Riverside 2017). The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site. The closest known active faults are associated with the San Bernardino Valley section of the San Jacinto Fault, located approximately 5.8 miles northeast of the subject site, and the Glen Ivy section of the Elsinore Fault Zone, located approximately 16.3 miles southwest of the subject site. The site is located approximately 0.19 miles west of an unnamed fault in the San Jacinto Fault Zone. The fault is not mapped by the State of California (CGS 2010) but it designated as a fault by Riverside County (Riverside County 2017).

Figure 6-1 and Figure 6-2 (following page) depict regional fault maps off the site vicinity.

Figure 6-1. Faulting in the Site Vicinity (CGS 2010)

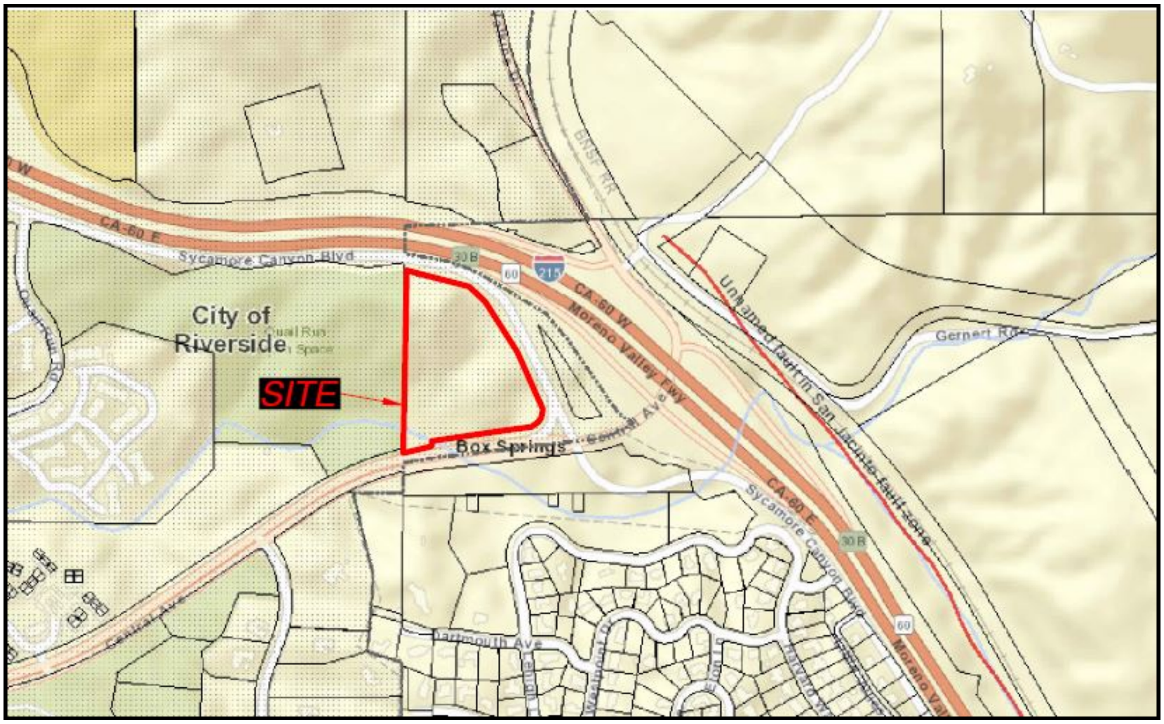

Figure 6-2. Faulting in the Site Vicinity (Riverside County 2017)

6.2.3 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 may be greater than about 10 feet thick and larger than 300 feet across. Landslides typically may 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 more narrowly confined modes such as rock topples, ‘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.).

Clues to the landslide hazard for an area can also be obtained by review of mapping that depicts both historic landslides and landslide prone geology/topography. Figure 6-3 (following page) reproduces such mapping for the site area. The mapping indicates that the site is in an area judged to be at lower relative risk for landsliding.

No indication of large scale landsliding was observed at the time of NOVA’s investigation. Minor amounts of slope erosion and surficial sloughing was observed along the southwestern boundary of the site in the existing fill slope. Neither AKA 2007 nor Geocon 2018 report evidence of active landsliding at this site.

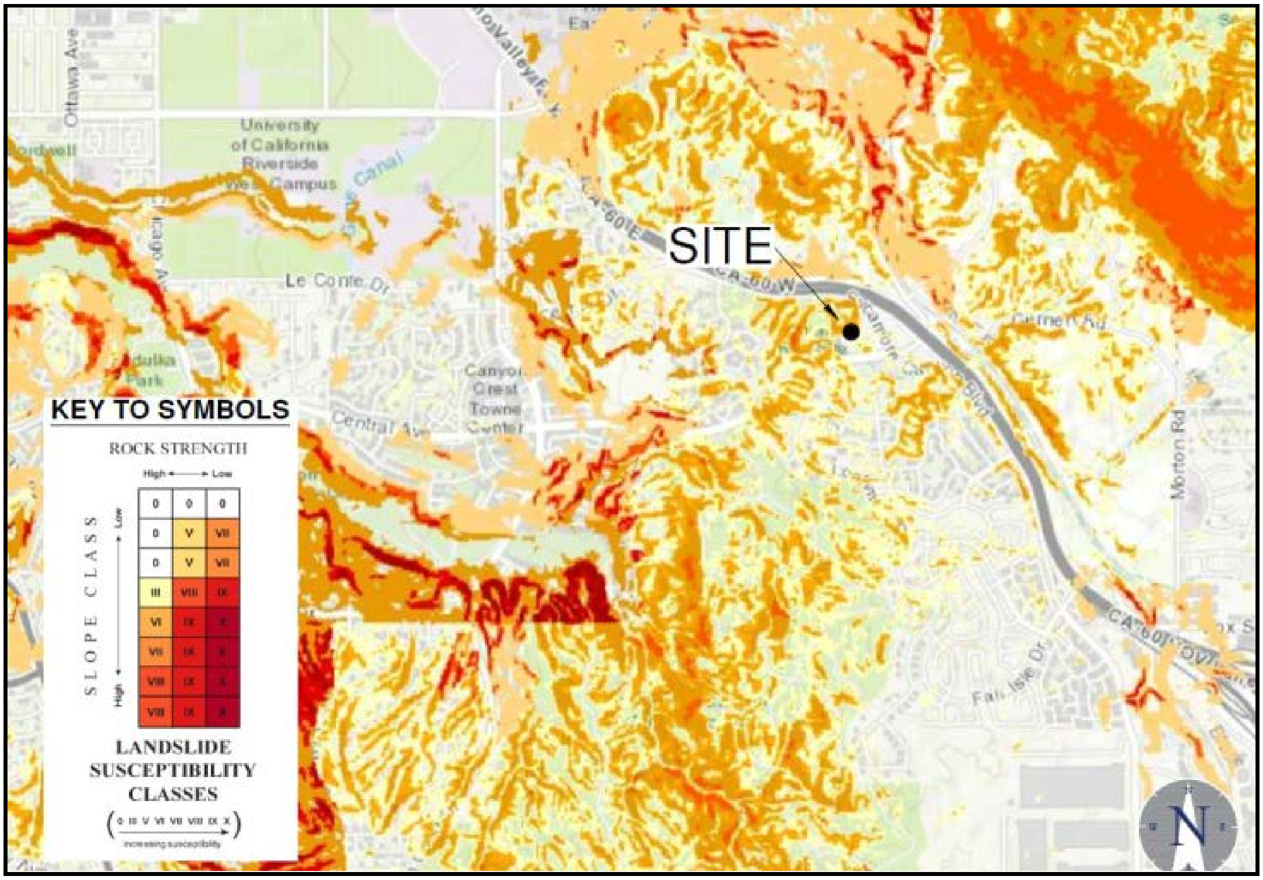

Figure 6-3. Landslide Susceptibility Mapping of the Site Area (USGS 2020a)

In consideration of the indications of the geologic reconnaissance by NOVA and others, review of published mapping, and review of aerial photography, NOVA considers the landslide hazard at the site to be ‘low’ for the site and the surrounding area in their current condition.

6.3 Soil Hazards

6.3.1 Embankment Stability

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

As is discussed in more detail in Section 2, the site includes some relatively steep cuts in rock. However, by virtue of the strength of the granitic rock exposed in these cuts, the embankments are not at risk for collapse.

Development of the site will include development of numerous smaller embankments across the limits of the site. The stability of these various embankments should be reviewed on the basis of development design and evaluated during grading.

6.3.2 Seismic

Liquefaction

'Liquefaction' refers to the loss of soil strength during a seismic event. Seismic ground motions increase soil pore water pressures, decreasing grain-to-grain contact among the soil particles, causing the soils to 'liquefy' and lose strength. 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 depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

In consideration of the lack of groundwater and related lack of the potential for saturated soils to occur in the near surface, the site is not at risk for liquefaction and related soil phenomena (i.e., lateral spreading, ground lurching, etc.). The site is not in area designated by the County of Riverside to be susceptible to liquefaction (County of Riverside 2017).

Seismically Induced Settlement

Apart from liquefaction, a strong seismic event can induce settlement within loose to moderately dense, unsaturated granular soils. The unsaturated sandy soils of Unit 1 are presently of sufficiently loose consistency so that these soils will be prone to measurable seismic settlement. However, as is discussed in the subsequent text of this report, it is intended that this ground be improved by densification such that this risk is removed.

6.3.3 Expansive Soil

Expansive soils are clayey soils characterized by their ability to undergo significant volume changes (shrinking or swelling) due to variations in moisture content, the magnitude of which is related to both clay content and plasticity index. These volume changes can be damaging to structures. Nationally, the annual value of real estate damage caused by expansive soils is exceeded only by that caused by termites.

The dominantly sandy soils that comprise Unit 1 and Unit 2 are not at risk for volume change on wetting and drying.

6.3.4 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, 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. Human activities that can facilitate soil collapse include irrigation, water impoundment, changes to the natural drainage, disposal of wastewater, etc.

The loosely placed sands of Unit 1 are at some risk of soil collapse upon wetting. However, as is discussed in the subsequent text of this report, it is intended that this soil unit be improved by densification such that this risk is removed. The site is not within an area designated by the County of Riverside to be susceptible to subsidence (County of Riverside, 2017).

6.3.5 Undocumented Fill

The site is covered by undocumented fill- predominantly sandy soils of loose to medium dense consistency that range to more than 35 feet in thickness. Records regarding placement of the Unit 1 fill are not available for review, such that the fill is considered ‘undocumented,’ subject to wide variations in quality and consistency.

A common concern in this regard is the occurrence of unanticipated highly compressible soils. As is discussed in the preceding sections, the soils are also at risk for both hydro-collapse and settlement during a seismic event.

As is discussed in the subsequent text of this report, the risks to related to the undocumented fill will be removed by large-scale ground improvement.

6.3.6 Corrosivity

The near-surface soils were tested to determine levels of sulfates and chlorides. The testing indicates that (i) the potential for sulfate attack to embedded concrete is negligible; and (ii) the potential for corrosion of embedded metals is relatively low. The indications of this testing are discussed in more detail in Section 7.

6.4 Siting Hazards

6.4.1 Effect on Adjacent Properties

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

6.4.2 Flood

The site is within an area designated as Riverside County Unincorporated Areas. Flood Map No. 06065C0733G, (not printed 8/28/2008) designates the area as “Zone D,” an area of undetermined flood hazard. The site is not in an area designated by the County of Riverside as having a potential for flooding (County of Riverside, 2017). Figure 6-4 reproduces flood mapping by FEMA of the site area.

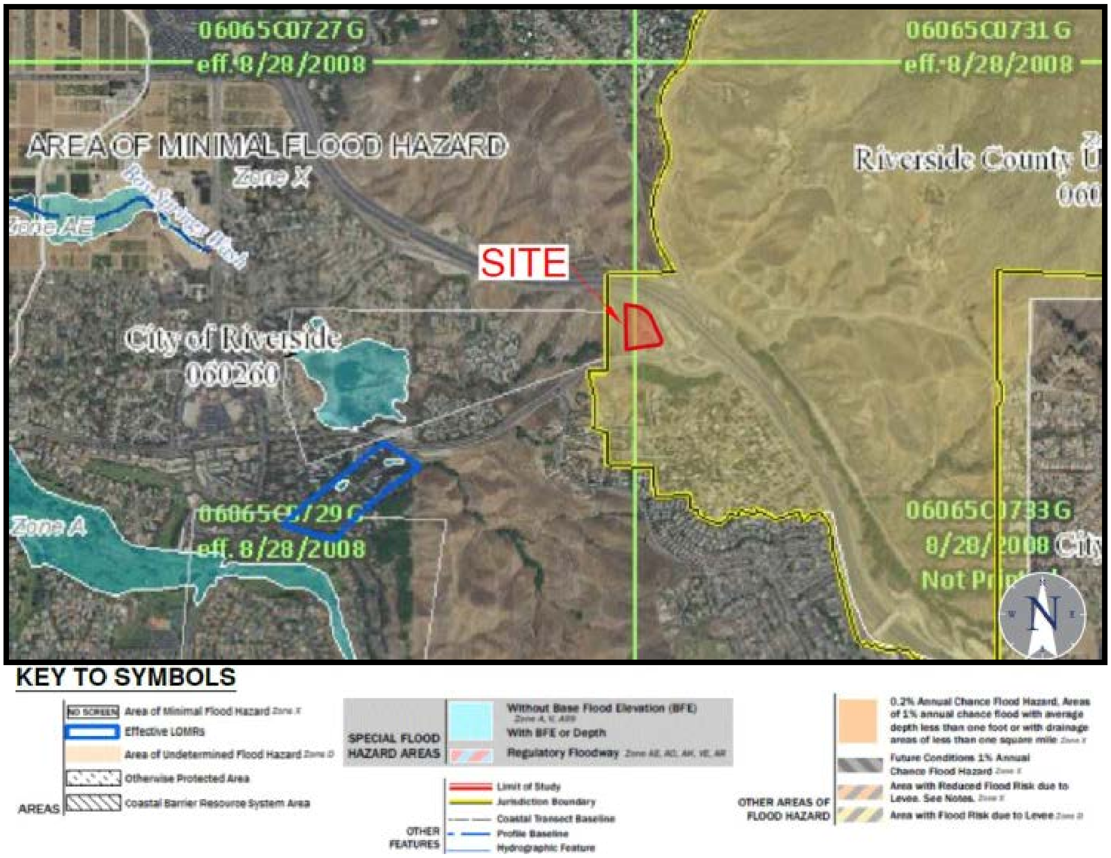

Figure 6-4. Flood Mapping of the Site Area
(source: adapted from FEMA Panel 06065C0733G, not printed 8/28/2008)

6.4.3 Inundation

The site is not located near a surface body of water (dam, levee, canal, etc.) which could lead to inundation of the project site if overtopping or an impoundment breach were to occur.

6.4.4 Tsunami and Seiche

The altitude of the site and distance from the ocean preclude potential for being affected by a tsunami.

Seiches are standing waves that develop in enclosed or partially enclosed bodies of water such as lakes or reservoirs. Harbors or inlets can also develop seiches. Most commonly caused by strong winds and rapid atmospheric pressure changes, seiches can be affected by seismic events and tsunamis. The altitude of the site and distance from enclosed bodies of water preclude the potential for seiche effects.

7.0 GROUND IMPROVEMENT

7.1 Overview

This section provides review of the Unit 1 undocumented fill that covers much of the site, identifying the need for ground improvement to densify and homogenize this soil. Such improvement will mitigate (i) longer-term settlement potential and (ii) the potential for differential settlement of heterogeneous soils.

As is previously discussed, the undocumented fill at this site ranges to greater than 35 feet in thickness. To this end, alternatives that may be considered for ground improvement must be adaptable to both the nature of the fill and its thickness.

This section reviews ground improvement technologies that meet these criteria, identifying deep dynamic compaction (‘DDC’) as preferred on a basis of expected performance and cost.

7.2 The Need for Ground Improvement

7.2.1 General

The structures proposed for development at this site are relatively light. As is well developed by AKA 2007, subsurface conditions pose two concerns for development, as abstracted below.

1. Concern 1, Undocumented Fill. Much of the site is covered with uncontrolled sand/cobble/ boulder fill. This fill ranges in thickness from a few feet to greater than 35 feet. The fill is characteristically loose/compressible and extremely heterogeneous (i.e., widely varying in quality and consistency over short horizontal and vertical distances).
2. Concern 2, Near Surface Rock. Sound granitic bedrock occurs in the near surface in several areas of the site. This rock will be difficult to excavate, posing a challenge to site development and related foundation design.

Concern 1 drives this assessment - the need for large-scale ground improvement. As is discussed in Section 1, this report is intended to address only planning for improvement of the undocumented fill. Near surface rock will be encountered during various phases of grading. Rock will need to be removed from the upper portions of building pads and streets, and within utility corridors. Rock will need to be either re-utilized for landscaping features, hauled away off-site or can placed within approved areas meeting criteria and recommendations as described in Section 8.4.12.

7.2.2 Near Surface Rock

Foundations for structures will be at risk for differential movement if founded partially on rock and partially on fill. Development of foundations for the separate apartment units will require undercutting/removal of near-surface rock to create stable foundations. Absent planning to manage the near surface rock, grading to develop site pads can become costly.

An earthwork contractor experienced in work of this nature can readily identify the requirements and related cost for loosening and removal of near-surface rock.

7.2.3 Undocumented Fill

While the fill is of an age that most settlement has already occurred, the performance of foundations for the separate apartment buildings can be affected by (i) the differential thickness (ranging to about 35 feet) of fill beneath structures and (ii) longer-term collapse of zones of loosely placed fill.

Though the existing fill extends to depths greater than 30 feet, NOVA does not believe that the entire fill thickness needs to be densified. The new structures will not materially change the state of stress for soils below a depth of about 25 feet. Site improvements outside the limits of structures - roadways, utility lines, common areas, landscaped areas, etc. will not require improvement. As is discussed in AKA 2007, longer-term settlements of unimproved fill will be relatively small. A relatively homogenous, well-densified cap of soils above this level will be suitable for excellent long-term performance of structures.

With the foregoing perspective, the objectives of any ground improvement at the separate apartment building structures should be as described below.

1. <u>Objective 1, Depth of Improvement</u>. Densify the undocumented fill as it may occur beneath the finished pad levels of structures to depths of up to 25 feet.
2. <u>Objective 2, Foundation Bearing</u>. Densify the near-surface soils sufficient to develop an allowable bearing pressure (i.e., contact stress) as great as about 6,000 psf for shallow foundations.

7.3 Alternatives for Ground Improvement

A variety of alternatives is available for improvement of the fill. Table 7-1 abstracts ground improvement alternatives that are potentially applicable at this site.

Table 7-1. Overview of Alternatives for Larger-Scale Ground Improvement

Description	Applicable To Site? [1]	Notes
Excavate & Replace	Yes	Labor, equipment and energy intensive
Deep Dynamic Compaction ('DDC')	Yes	Strong track record of success at similar sites. Numerous competitors
Rapid Impact Compaction ('RIC')	Yes	Strong track record with fills less than 20 feet thickness
Rolling Dynamic Compaction	Partial	Limited U.S. experience. Applicable to roadways, but cannot reach 15 feet depth
Compaction by Explosives	No	Great track record in the literature, poor track record in practice
Compaction Grouting	No	Boulder fill will limit effectiveness
Vibrocompaction	No	Boulder fill will limit effectiveness

Notes:

1. "Applicable to Site" means applicable to reliable densification of the range of soil/cobbles/boulders at the site to a depth of about 25 feet.

Alternatives considered by Table 7-1 are those that improve largely “cohesionless” (i.e., sandy) soils, to which the term “densification” or “compaction” are conventionally applied.

Alternatives not considered by Table 7-1 are those that improve largely “cohesive” (i.e., clayey) soils, to which the terms “consolidation” or “compression” are conventionally applied. For example, ground improvement that involves preloads (i.e., “pre-compression”) or analogous efforts are not considered.

7.4 Review of Potentially Applicable Approaches to Ground Improvement

7.4.1 General

As may be seen by review of Table 7-1, ‘deep dynamic compaction’ and ‘rapid impact compaction’ are technologies that, on the basis of expected performance and cost, appear most applicable to this site. The following subsections provide more detailed review of each of these ground improvement technologies.

7.4.2 Deep Dynamic Compaction (‘DDC’)

General

DDC utilizes a heavy tamper (or ‘pounder’) that is repeatedly raised and dropped with a single cable from varying heights to impact the ground. The mass of the tampers generally ranges from 5 to 20 tons, while drop heights range from 30 feet to 80 feet. While specialty equipment can be developed for larger sites in order to effect higher drops of greater weights, the usual project employs a tracked crane to lift and drop the tamper. Figure 7-1 shows a typical operation.

Figure 7-1. Crane Lifting the Tamper

The pounding energy from DDC is generally applied in ‘phases’ (or ‘passes’) on a grid pattern over the entire area using either single or multiple passes. Following each pass, the impact craters formed by the tampers are either leveled with a dozer or filled with granular fill material before the next pass of energy is applied. A final pass - intended to densify the upper several feet of soil loosened by previous work and sometimes called an ‘ironing pass - consists of overlapping short drops of the tamper. DDC has been successfully used to improve a great variety of loose/weak ground deposits including those listed below.

- Loose naturally occurring soils such as alluvial, flood plain, or hydraulic fill deposits.
- Municipal solid waste (MSW) landfills.
- Building rubble and construction debris deposits.
- Strip mine spoil.
- Collapsible soils (i.e., soils that may settle as they become wetted).
- Loose sands and silts with high liquefaction potential.

At least 900 DDC projects have been completed in the U.S., mostly for commercial purposes. Dynamic compaction is practiced by a number of specialty contractors across the U.S., such that it should be expected that this project will draw several qualified competitors for the work.

Depth of Improvement

Estimates of the depth of improvement by dynamic compaction are empirical. Both the degree of improvement and the depth of improvement is a function of the energy applied: i.e., the mass of the tamper, the drop height, the grid spacing, and the number of drops at each grid point location.

Lighter tampers and smaller drop heights result in lesser depths of improvement, while heavier tampers and greater drop heights result in improvements to a significant depth.

The depth of improvement is generally estimated by the following expression:

$D = n\,(WH)^{1/2}$, where D = Depth of improvement, in meters
W = Weight of the Tamper, in metric tons
n = reduction coefficient based on soil type
= 0.5 to 0.6 for unsaturated granular soils

Utilizing the above equation, NOVA estimates that a 14-ton tamper dropped from about 26 feet can effect improvement to about 15-feet depth of improvement. This same pounder dropped from about 50 feet can effect improvement to 25 feet depth.

Applicability at This Site

DDC can meet all of the densification at this site, including the need to effect densification of the uncontrolled fill to depths ranging from about 10 feet to 25 feet. DDC has a well-established track record in densification of uncontrolled and heterogeneous fills of this genre to these depths. Areas of near-surface rock will not require improvement by DDC.

Loss of Ground, Vibrations, and Noise

DDC to 25 feet will effect a volume change on the order of 8% to 10% over the interval treated, resulting in a loss of about 2 to 2.5 feet of ground. This ground loss must be accounted for in planning for earthwork.

At 100 feet, peak particle velocities at the ground surface will be on the order of 2 to 3 5mm/second. Vibrations will vary with material type and will increase as the degree of compaction achieved increases. Results to date indicate that without site-specific testing, a safe working distance to structures on the order of 20 feet may be appropriate. Planning to implement DDC must consider its effects on any nearby construction, particularly effects on concrete that is curing. A variety of effective methods have been employed to limit potential damage from surface waves.

The technique is not particularly loud. Noise levels will be about 90 decibels at 20 feet.

7.4.3 Rapid Impact Compaction

General

Developed in the early 1990's by the British Military, "rapid impact compaction" (RIC) is a form of dynamic compaction, utilizing a hydraulic hammer to lift and drop a 7.5-ton weight from a controlled height of about 1 m (3 feet). The pounding rate is high, typically 40 to 60 blows per minute. Designed for use in granular deposits, benefits have also been noted in random fills and mine wastes. Figure 7-2 depicts a larger scale application of this procedure.

Figure 7-2. Large-Scale Application of Rapid Impact Compaction

NOVA has recent, successful experience utilizing RIC with a similar fill (i.e., a deep, sandy fill with rubble and boulders). The hammer utilized at that site was mounted on a Cat 345 mobile carrier. The densification process is completed with a high degree of control, allowing the machine to be used in difficult locations, and for a variety of applications. The drop height, number of blows, and penetration per blow are monitored and/or controlled by a data acquisition system. The high energy impacts densify cohesionless soils by a combination of displacement and vibration.

Figure 7-3 depicts the mechanics of the RIC process.

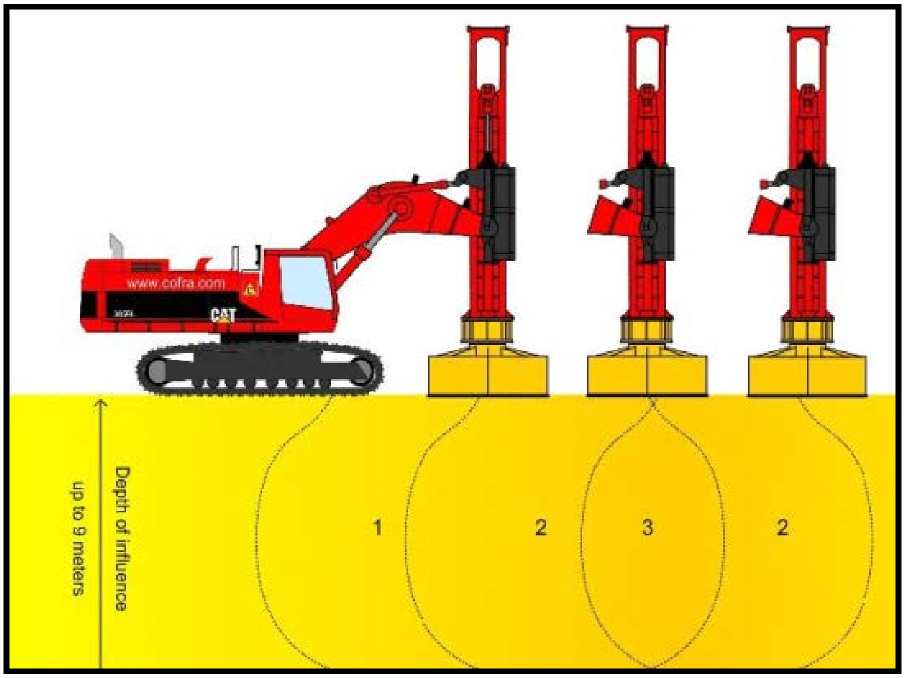

Figure 7-3. Overlapping Influence of Points of Rapid Impact Compaction
(source: Cofra Ltd, found at http://cofra.com/activities/rapid-impact-compaction/)

Limited Applicability at This Site

The potential that significant obstructions (e.g., nested boulders and rock) may occur below a depth of about 20 feet is of concern. Though RIC is relatively fast and potentially cost-effective, as applied to this site, the technology could not be relied upon to achieve densification beyond a depth of about 15 feet. However, much of the work at the site will be conducted at this depth and shallower.

Loss of Ground, Vibrations, and Noise

RIC to 15 feet will effect a volume change (enforced ground settlement) on the order of 7% to 10% over the interval treated, resulting in ground loss of about 1 to 1.5 feet.

At 100 feet, peak particle velocities may vary from about 1 to 2.3 mm/second. Vibrations will vary with material density, increasing as compaction increases.

The technique is relatively loud. Noise levels will be about 140 decibels at 20 feet.

7.5 Recommended Ground Improvement by DDC

In consideration of expected performance, deep dynamic compaction ('DDC') is recommended as the ground improvement technique at this site.

Rapid impact compaction ('RIC') does not have the potential to achieve the depths of densification that does DDC. However, the technology can be competitive with DDC in areas where densification is required to depths of 15 feet or less. Additional site characterization and evaluation may identify a basis for limiting the depth of ground improvement. Alternatively, development of the site in separate construction phases may raise instances where multiple mobilizations for ground improvement are necessary and lesser depths of densification are required. In these or analogous instances, RIC would compete well with DDC.

7.6 Implementing DDC

7.6.1 Expected Response of the Fill to Pounding

Areas of Boulders

Densification by DDC will adapt to the of heavy boulders within the interval to be densified. Subsurface materials in such areas could densify less readily, requiring more energy input. The energy from the impact of the pounder coupled with the nested boulders may distribute more of the pounder impact in the upper portion, transmitting less energy to deeper soils. NOVA does not believe this circumstance will materially affect the overall DDC product.

Loose Surficial Soils

Loose to very loose surficial soils could present a problem that is the converse of the above. Loose/soft soils near the ground surface may initially limit the energy that is transmitted to greater depths. The tamper will tend to bury itself as it is dropped; i.e., the initial drops may result in crater depths greater than the height of the tamper. Allowing this to occur is undesirable for a number of reasons including:

- Operational Effects. Extracting the tamper from a deep crater is difficult and can result in cable breakage as loose debris falls in on top of the tamper, increasing the extraction loads.
- Efficiency Effects. After the tamper is extracted from a deep crater, the sides may cave into the crater, providing a cushioning effect for the next impact. Similarly, the caving that occurs can cause the tamper to strike the base irregularly with some of the energy being absorbed as the tamper strikes the side walls of the crater.

The specialty contractor who performs the work will have several alternatives to manage this circumstance. For example, the Unit 1 soils can be stabilized by adding granular soil that is driven into the loose soil during impact.

7.6.2 Test Sections

The specialty contractor employed for this work will be required to complete test sections in the representative areas to evaluate the depth and degree of improvement that can be attained.

In instances where such testing shows that DDC cannot achieve the degree/depth of improvement necessary, the specialty contractor will be required to adapt to this finding by either increasing the applied energy (see Section 7.4.2) or otherwise take measures to demonstrate adequate densification.

7.6.3 Estimated Enforced Settlement

Planning for implementation of DDC should anticipate a loss of ground on the order of 2 feet in areas of deeper densification. This loss of ground will be less in shallower areas. NOVA expects that the volumetric loss of ground will be on the order of 10% of the thickness of the column of fill being densified.

7.6.4 Placement of Site Fill

NOVA's preliminary evaluation of earthwork indicates that some portions of the site will be receiving several feet of fill at the building pad location. A common question from earthwork contractors regards the wisdom of placing fill required for the site within the areas to be densified, allowing this fill to be in part densified by dynamic compaction.

NOVA expects that such an approach would work well, as it is expected that most of the surficial soils are relatively looser. These looser soils may benefit by the stabilizing effect of a new fill, enhancing the stability of the ground surface and improving the efficiency of dynamic compaction operations.

Placing fill will abet development of a working 'mat' that will stabilize equipment. The most favorable type of material to use for a working mat is a coarse-grained granular deposit such as gravel or crushed rock.

7.7 Recommended Approach to Contracting

7.7.1 General

Two basic types of specifications are used for dynamic compaction projects:

- Method Specifications. The Owner provides the Contractor a proscriptive approach to the work needed to obtain the desired improvement, specifying the size of tamper; drop height; energy that needs to be applied; area that is needed to be densified; number of passes to be made plus the delay time, if any, between passes; plus consideration of off-site vibration or displacement as a result of dynamic compaction.
- Performance Specifications. The Owner provides performance criteria expressed as improvement in the strength and compressibility of the material that is treated. Performance criteria are often associated with requirements for a minimum energy that needs to be applied.

The decision as to which type of specification to use will depend on a variety of factors, including the complexity of the job, the proximity of specialty and non-specialty contractors to the site, the time available for test sections, etc.

The following subsections provide brief review of each approach.

7.7.2 Method Specification

In this type of contract, the contractor assumes very little, if any, risk related to the improvement that occurs as a result of dynamic compaction. The contractor's primary duties under this type of contract are:

- to provide a tamper of the prescribed size and with the proper contact pressure at the base and
- to provide the proper equipment to raise and drop the tamper

The advantage of a method specification is that different types of contractors can bid the work. DDC has sometimes been completed by earth moving contractors, wrecking contractors, or specialty contractors. The local contractors would have the advantage of lower mobilization and general knowledge of the area and can be very competitive. This advantage accrues in smaller jobs for which execution of the work is relatively simple and the potential for changes are limited.

7.7.3 Performance Specification

More complex projects that resource the expertise of specialty contractors with broad-based expertise in dynamic compaction are amenable to performance contracts. In this method, the Owner's Engineer specifies the required degree and depth of improvement and the contractor selects the proper equipment to achieve this goal.

The responsibilities of the Owner's Geotechnical Engineer of Record (GEOR) will include:

- providing subsurface information including the geotechnical report to the bidders,
- defining the extent of the area to be improved, and
- specifying the end product to be achieved.

The Contractor assumes a greater risk with this type of a contract. If the equipment selected to do the work does not achieve the desired end product, the contractor must alter his field procedures; for example, use a heavier tamper or a larger drop height to achieve the goals. Normally, this work is undertaken on a lump sum basis and the contractor absorbs the additional costs.

7.8 Control of DDC

7.8.1 Pre-Densification Data

It is important that the GEOR develop a sound base of developed pre-densification data using *in situ* geotechnical testing (e.g., pressure meter test (PMT) or the cone penetrometer test (CPT)) and geophysical testing, as appropriate.

The standard penetration test (SPT) is often used to gauge ground improvement. Of concern in this regard is that at sites such as this (i.e., heterogeneous, with boulders and debris), the SPT values after dynamic compaction are frequently the same order of magnitude as SPT values before dynamic compaction. At these same sites, PMT has frequently shown significant improvements. The PMT measures increase in the stiffness of the soil deposit, which is one of the primary reasons for dynamic compaction. The increase in stiffness results in the reduction in compressibility of the soil mass. The SPT is insensitive to the stiffness because of the remolding of the soil as the sampler is being driven. With this perspective, the PMT and geophysical testing will be used to develop a pre-densification data base.

7.8.2 Control During Densification

PMT and monitoring of ground subsidence should be undertaken during densification. Ground subsidence will typically reach a threshold value (for example, settlement of 5 to 10% of the original thickness of the formation), after which added pounding has little settlement effect, indicating that densification has taken place.

Measurements of the velocity of the tamper (using a radar gun) should also be undertaken to assure that near free fall is achieved (i.e., fall relative to the theoretical velocity for a tamper falling in a vacuum). The results are fairly consistent for different size tampers and different geographic locations.

Geophysical testing completed prior to densification can be reproduced following densification, showing an increase in seismic wave velocity as an indicator of increased soil stiffness.

8.0 EARTHWORK AND FOUNDATIONS

8.1 Overview

8.1.1 Review of Site Hazards

As is discussed in Section 5, the site is affected by two principal soil/geologic hazards, as abstracted below.

1. Seismic. The site is at risk for moderate-to-severe ground shaking in response to large-magnitude earthquakes during the lifetime of the planned development. Section 7.2 provides seismic design parameters

2. Undocumented Fill. The site is mantled by undocumented fill that ranges to about 34 feet in thickness. The fill is predominately sandy, but includes gravel, cobble and boulder-sized rock. As is discussed in Section 6, the Unit 1 undocumented fill will be improved by large scale densification, removing this hazard prior to developing structures.

8.1.2 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 opinion of NOVA that the site is suitable for development utilizing shallow foundations, 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 right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

8.1.3 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 work related to site and foundation development should be completed under the observation of NOVA as Geotechnical Engineer-of-Record (GEOR). Section 10 addresses this consideration in more detail.

8.1.4 Preliminary Recommendations

The remainder of this section provides preliminary recommendations for earthwork and foundations. These recommendations are provided well in advance of design and, very significantly, well in advance of any detailed planning for ground improvement and final grading for development of the structures and infrastructure across the site. In consideration of these factors, the recommendation should be understood to be preliminary, subject to change.

8.2 Seismic Design Parameters

8.2.1 Site Class

The shear wave testing described in Section 3 and Section 4 shows that the average shear wave velocity of the upper 100 feet of the subsurface (V_{100}) averages 2,109.1 feet per second. This average shear wave velocity meets the criterion for Site Class C per ASCE 7-16 (Table 20.3-1).

8.2.2 Seismic Design Parameters

Table 8-1 provides seismic design parameters after ASCE 7-16 utilizing the on-line resource provided by the USGS and SEAOC for this determination.

Table 8-1. Site Class C, Risk Category II, ASCE 7-16 and 2019 CBC

Parameter	Value
Site Soil Class	C
Site Latitude (decimal degrees)	33.959 °N
Site Longitude (decimal degrees)	- 117.313 °W
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
Mapped Short Period Spectral Acceleration, S_S	1.5 g
Mapped One-Second Period Spectral Acceleration, S_1	0.6 g
Short Period Spectral Acceleration Adjusted for Site Class, S_{MS}	1.8 g
One-Second Period Spectral Acceleration Adjusted for Site Class, S_{M1}	0.84 g
Design Short Period Spectral Acceleration, S_{DS}	1.2 g
Design One-Second Period Spectral Acceleration, S_{D1}	0.56 g
Modified Peak Ground Acceleration, PGA_M	0.613 g

Source: Site Class parameters obtained from *SEAOC Hazard Tool* (found at: https://seismicmaps.org/)

8.3 Corrosivity and Sulfates

8.3.1 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. AKA 2007 reports this testing on a representative sample of the near-surface soils.

The results of the testing are tabulated in Table 8-2 (following page).

Table 8-2. Summary of Corrosivity Testing

Parameter	Units	Value
pH	standard unit	7.6
Resistivity	Ω-cm	2,400
Water-Soluble Chloride	ppm	42.5
Water-Soluble Sulfate	%	0.001

8.3.2 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 E 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 below.

Table 8-3. 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

The resistivity testing summarized on Table 8-3 suggests that design should consider that the soils may be corrosive to embedded 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 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.

8.3.3 Sulfate Attack

As shown in Table 8-2, the soil sample tested indicated water-soluble sulfate (SO_4) content of 0.001% by weight. With SO4 < 0.10 percent 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 8-4 reproduces the Exposure Categories considered by ACI.

Table 8-4. Exposure Categories and Requirements for Water-Soluble Sulfates

Exposure Category	Class	Water-Soluble Sulfate (SO_4) In Soil (percent by weight)	Cement Type (ASTM C150)	Max Water-Cement Ratio	Min. f'_c (psi)
Not Applicable	S0	$SO_4 < 0.10$	-	-	-
Moderate	S1	$0.10 \leq SO_4 < 0.20$	II	0.50	4,000
Severe	S2	$0.20 \leq SO_4 \leq 2.00$	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

8.3.4 Limitations

Testing to determine several of the chemical parameters that indicate a potential for soils to be corrosive to construction materials are 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.

8.4 Earthwork

8.4.1 General

Following the large-scale ground improvement described in Section 6, NOVA expects that earthwork will include (i) finish grading to create the new ground form for the separate structures, and (ii) excavations for foundations and 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*."

8.4.2 Select Fill

Materials

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

- maximum particle size of 4-inches;
- classified as GM, GC, SC, SM or SP-SM after ASTM D 2487; and,
- expansion index (EI) of less than 10 (i.e., EI < 10, after ASTM D 4829).

Most of the Unit 1 fill that is now in place will conform to the above criteria.

Placement

All fill and backfill should be compacted to a minimum of 90% relative compaction after ASTM D1557 (the 'modified Proctor') following moisture conditioning to slightly over the optimum moisture content.

The fill must be densified using vibratory compaction methods. Fill should be placed in loose lifts no thicker than the ability of the vibratory 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) will be limited to about 4-inches or less.

Oversized Material

Oversized materials in excess of 6 inches in maximum dimension should be removed from fills and properly disposed of off-site. If feasible, crushing oversized materials onsite and incorporating them into "rock fills" (windrows, rock blankets, or individual rock burial) may be considered. Another alternative is to export the oversized materials. Isolated core stones consisting of slightly weathered to fresh bedrock may be encountered buried in a more weathered matrix in the tonalite bedrock. Specially recommendations should be provided on a case-by-case basis, if core stones are encountered

8.4.3 Site Preparation

Prior to the start of any land-disturbing activities, the Contractor should establish construction Best Management Practices ('BMPs') to control erosion of graded/excavated areas. BMPs must be maintained until permanent stormwater infiltration BMPs are operable.

At the outset of work, the site should be cleared of vegetation and related root systems, and existing pavement. The deleterious materials should be disposed of in approved off-site locations.

As is discussed in Section 2, the site has not been used other than for car parking. It is unlikely that site preparation will expose relic foundations, utilities, etc.

8.4.4 Site Grading

Based upon review of the planning described SDH 2020, it is expected that significant earthwork will be required for ground improvement, site preparation and to create level pads for the structures. Grading should be performed in 3 separate phases for the development of the site.

(i) Phase 1 grading will consist of site preparation, leveling areas for preparation of DDC techniques. DDC areas should be considered for all building pads, paved areas and retaining walls where existing fills extend beyond about 5 feet thick.

(ii) Phase 2 rough grading will follow DDC operations, taking the surfaces from ground improvement elevations to near design grades as indicated on the Civil grading plans.

(iii) Phase 3 grading will be precise grading building pads and other areas supporting new improvements including appropriate drainage within appropriate tolerances.

8.4.5 Excavation Characteristics

The Unit 1 fill will be readily excavated by earthwork equipment usual for developments of this nature. No heavy ripping or special excavation techniques will be required. Unbraced slopes less than 4 feet in height will stand for short periods (less than 2 weeks) at slopes as steep as 0.5H:1V. The handling of rock and boulders should be planned for, as Geocon 2018 indicated boulders as great as 11 feet in diameter were encountered during site exploration.

The Unit 3 granitic rock occurs in the near surface in areas of the site. This rock unit will be difficult to excavate potentially requiring mid-size to heavy size equipment to excavate this material. Planning for loosening and removal of near-surface rock is outside the scope of this report. An earthwork contractor experienced in work of this nature can readily identify the requirements and related cost for loosening and removal of near-surface rock.

8.4.6 Rippability

Rippability refers to the ability of a subsurface unit to be excavated with by conventional heavy-duty earthwork equipment. There are a variety of rippability performance charts prepared by Caterpillar, Inc., Caltrans, and Santi. The different rippability classifications in this report (rippable, moderately rippable, and non-rippable) are based on the Caterpillar D-9R Ripper Performance Chart (Caterpillar 2018). The seismic refraction survey data was used with Caterpillar 2018 to estimate depths to the different rippability classifications at the site. The seismic refraction data is summarized blow and presented in Appendix C.

- Velocity Layer V1 contained average seismic velocities ranging from approximately 1,948 feet per second (fps) to 2,370 fps. These velocities are typical of undocumented artificial fill, alluvial deposits, and/or completely weathered and fractured tonalite bedrock. Velocity Layer V1 is considered rippable with conventional equipment.

- Velocity Layer V2 contained average seismic velocities ranging from approximately 3,340 fps to 3,504 fps. These velocities are typical of highly weathered tonalite bedrock and of older alluvial deposits, which may be locally present in this velocity layer. Velocity Layer V2 is considered rippable with conventional equipment.

- Velocity Layer V3 contained average seismic velocities ranging from approximately 5,789 fps to 8,246 fps. These velocities are typical of moderately to slightly-weathered bedrock. Velocity Layer V3 is considered rippable to non-rippable with conventional equipment.

Generally, the undocumented artificial fill and alluvial deposits are anticipated to be rippable utilizing conventional heavy-duty earthwork equipment. The upper portions of the site bedrock are anticipated to be rippable to non-rippable utilizing conventional heavy-duty earthwork equipment. Based on the data from the seismic refraction lines, the excavation difficulty of the bedrock increases with depth. Proposed design grades and recommended over-excavation depths have been plotted on relevant layer velocity models and the tomographic models from the seismic refraction survey performed by NOVA and by AKA 2007. These graphs are presented at the read of this text on Plates 4A through 4E, Plate 5A, and Plate 5B.

It is important to note that the velocity ranges of the classifications are approximate and that rock characteristics, including fracturing, spacing, and orientation, are a major factor in determining rippability. Localized zones of potentially non-rippable bedrock (such as core stones) should be anticipated to be encountered above the estimated non-rippable depths. It is recommended that prospective contractors review the provided subsurface data and independently estimate potential heavy ripping / blasting quantities based on their experience.

A seismic refraction study was performed by AKA 2007 and consisted of five (5) seismic refraction lines. The seismic refraction data by others is summarized below and presented in Appendix C.

- Velocity Layer V1 contained average seismic velocities ranging from approximately 2,014 fps to 3,303 fps.
- Velocity Layer V2 contained average seismic velocities ranging from approximately 5,080 fps to 7,473 fps.

AKA 2007 only included black and white tomographic images of the seismic refraction data. The subconsultant used for the seismic refraction survey was able to provide NOVA with the layer velocity models for the previous seismic refraction survey. However, no seismic refraction survey report was prepared and no interpretations of the data were made.

8.4.7 Blasting

Where ripping with heavy equipment is difficult, explosives or blasting techniques may be necessary to remove dense bedrock or to breakdown oversize boulders.

8.4.8 Trenching and Backfilling for Utilities

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. 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 D 1557 (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.

8.4.9 Retaining Walls

Based on the planned configuration for planned MSE retaining walls, ground improvement will be required. Leveling of the existing terrain within the area at the base of planned retaining walls will be required for DDC. A Phase 1 grading plan should include excavating a +60 foot-wide horizontal bench at the base of planned walls into sloping conditions to allow for DDC equipment access.

8.4.10 Building Pad Undercuts

General

Building pads that expose weathered Unit 3 granitic rock and/or shallow fill should be over excavated to a depth of 5 feet below the final pad grade, or to at least 3 feet below the bottom of footing, whichever is deeper, and this excavation backfilled with Select Fill.

This removal and replacement should extend at least 5 feet beyond the perimeter building footings.

Building Pad 6

Based on the configuration and siting of Building 6 including proximity to planned retaining walls, ground improvement will require leveling of the existing sloping terrain within the area. A Phase 1 grading plan will be required for DDC. Building pad undercuts may require excavation of about +20 from existing site grades.

8.4.11 Street Undercuts

Like the building pads described above, granitic bedrock that is within a few feet of the finish grades of streets should be undercut at least 3 feet or 1 foot below the lowest utility whichever is deeper, and backfilled with Select Fill. The intent of this recommendation is to allow placement of utility lines within the streets.

Planning for street development should consider future utility hookups and the potential problems created by the near surface occurrence of rock.

8.4.12 Placement of New Fills

All new fills should be placed in a manner that conforms with the requirements for Select Fill that are described in Section 8.4.2. Placement of such fill will require management of boulder-sized rocks that are now present within the Unit 1 soil mass.

The GEOR will work with the Contractor to develop planning for control in placement of oversized rock. At a minimum, such control will involve the actions listed below.

1. <u>No 'Nesting.'</u> The mass of fill soils should contain sufficient finer grained granular soils such that the 'nesting' (i.e., close accumulation or close spacing) of rocks larger than 8 inches is avoided.
2. <u>Boulders.</u> No rock larger than 3 feet in diameter may be placed in a new fill. However, rocks larger than 12 inches and less than 3 feet in diameter may be placed within engineered fill if the rock is placed greater than 10 feet below proposed finish grades and setback at least 15 feet from the face of newly constructed fill slopes. Placement of rock of this dimension range will require specialty seating and placement, and will require the surrounding fill soils to consist of granular soils compacted to at least 90 percent relative compaction. No rock greater than 3 feet in diameter may be used in any fill.
3. <u>Keyways and Benching</u>. Fill slopes equal to or greater than 5 feet in height should be constructed with a keyway having a minimum width of at least 5 feet and a minimum embedment of at least 2 feet into competent bedrock. New fill placed against ground sloping more steeply than 5H :1V (horizontal : vertical) should include vertical benches excavated into the adjacent slope.
4. <u>Fill Slope Construction</u>

   Fill placed on surfaces greater than 5V:1H should include a keyway cut into firm and competent soils. Fills should be placed as described in the Fill Placement and Compaction (below). Benching of the existing soils at temporary back-cut slopes should be performed during fill placement operations.

   Fill slope faces should also be compacted to minimum project recommendations. This may be accomplished by overbuilding the fill slope and cutting back to proposed grades, or by tracking the slope face at 5-foot vertical intervals during fill slope construction.

   Any oversized rock should be setback and placed at least 15 feet from the slope face as recommended within item 2 above.

   Upon completion of fill slope construction, landscaping and stormwater BMPs should be installed.
5. <u>Existing Fill Slopes</u>

   Along the western portion of the site, the existing fill slope extends outside the limits of proposed grading. This slope is composed of undocumented, potentially uncompacted artificial fill. NOVA observed erosional rills and minor surficial sloughing within this slope. It is recommended that these areas be reconstructed in accordance with the fill slope construction recommendations and that the areas be maintained and protected from further erosion and sloughing. As the soils are relatively granular in nature, and the proposed grading plans indicates drainage towards some of these areas, it is recommended that drainage areas include slope armor or energy dissipating structures to reduce erosion potential or be relocated further west, off of the fill slope. If requested, NOVA can provide additional recommendation for surface erosion mitigation.

8.5 Foundations

8.5.1 General

The structures can be supported on shallow foundations following ground improvement as described in Section 7 and/or site preparation as described in Section 8.4. The following subsections provide recommendations for shallow foundations.

8.5.2 Ground Supported Slabs

The ground level slabs may employ conventional on-grade (ground-supported) slabs, designed using a modulus of subgrade reaction (k) of 140 pounds per cubic inch (i.e., k = 140 pci).

The actual slab thickness and reinforcement should be designed by the Structural Engineer. NOVA recommends the slab be a minimum 5 inches thick, reinforced by at least #4 bars placed at 16 inches on center each way 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 exceeding 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 ¼ the slab thickness, as depicted in Figure 8-1.

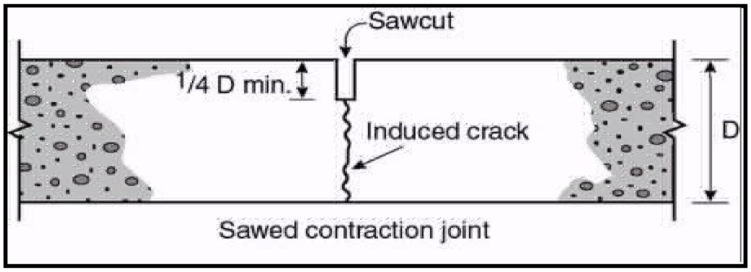

Figure 8-1. Sawed Contraction Joint

8.5.3 Isolated and Continuous Foundations

The densified Unit 1 fill will provide high-capacity foundation support for shallow foundations.

Shallow spread footings established in densified Unit 1 soil may be used to support the new buildings where designed to the parameters listed below.

1. *Minimum Dimensions*. Isolated footings should be at least 30 inches wide. Continuous footings should be at least 18 inches wide.
2. *Embedment*. Shallow foundations should be embedded a minimum of 24 inches below lowest adjacent grade. Foundations located near slopes should be embedded such that the horizontal distance to ‘daylight’ at the face of the slope is at least 7 feet from the face of the footing.
3. *Contact Stress*. An allowable bearing capacity (q_{allow}) of q_{allow} = 5,000 psf can be used for footings supported on densified Unit 1 soil or on new fill placed in conformance with the requirements of this report. These values apply to combined dead and sustained live loads (DL + LL). The allowable contact stress may be increased by one-third when considering transient loads, such as seismic and wind.
4. *Lateral Resistance*. Resistance to lateral loads will be provided by a combination of friction between the soil and foundation interface and passive pressure acting against the vertical portion of the footings. For calculating allowable lateral resistance, a passive pressure of 250 psf per foot of depth and a frictional coefficient of 0.35 may be used. No reduction is necessary when combining frictional and passive resistance.

8.5.4 Building Settlement

Structures supported on spread footings as recommended above will settle on the order of ½ -inch to 1-inch, with about 80 percent of this settlement occurring during the construction period. The differential settlement between adjacent, unevenly columns will be on the order of ½ inch or less over a horizontal distance of 40 feet.

8.5.5 Settlement Monitoring

NOVA recommends a program of settlement monitoring at the southwestern portion of the site where fills are greater than about 30 feet thick.

8.6 Retaining Walls

8.6.1 Lateral Pressures

Lateral earth pressures on retaining walls are related to the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. Table 8-5 provides soil loading on retaining walls with level and sloping backfill for varying conditions of wall yield.

Table 8-5. Lateral Earth Pressures to Retaining Walls

Condition	**Equivalent Fluid Pressure (psf/foot), Approved Backfill** Notes A, B	
	Level Backfill	**2:1 Backfill Sloping Upwards**
Active	35	60
At Rest	55	90
Passive	350	300

Note A: site-sourced Select Fill or similar imported soil.
Note B: assumes wall includes appropriate drainage and no hydrostatic pressure.

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.

8.6.2 Seismic Increment

Walls taller than 6 feet should include a seismic load increment, should be calculated as a uniform 16H psf (with H the height of the wall in feet).

8.6.3 Drainage

Design for permanent walls should include drainage to limit accumulation of water behind the wall. Figure 8-2 provides guidance for such design. Note that the guidance provided on Figure 8-2 is conceptual. A variety of options are available to drain permanent below grade walls.

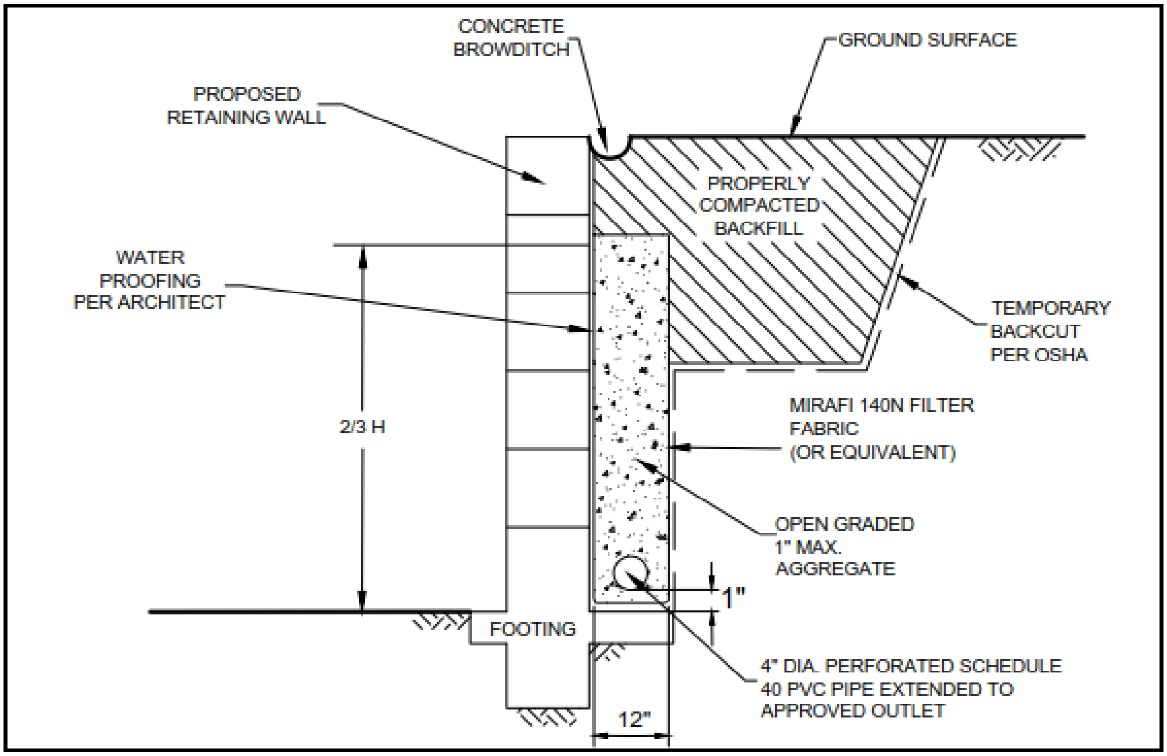

Figure 8-2. Conceptual Design for Permanent Wall Drainage

8.6.4 Elevator Pits

Elevator(s) may be used for the three and four level structures. Design for the elevator pit walls should consider the circumstances and conditions described below.

1. Wall Yield. NOVA expects that proper function of the elevator pit should not allow yielding of the elevator pit walls. As such, walls should be designed to resist ‘at rest’ lateral soil pressures and seismic pressures provided above, also allowing for any structural surcharge.
2. Construction. Design of the elevator pit walls should include consideration for surcharge conditions that will occur during and after construction.

8.7 MSE Retaining Walls

The referenced preliminary grading plans indicate terraced mechanically stabilized earth (MSE) retaining walls will be constructed along the southern and western boundaries of the site. As an example, at the southwestern corner near Building No. 6, plans indicate a series of four to six walls about 5 feet in height with a 2H:1V slopes between walls. Similarly, the terraced walls extend along the southern and western boundary adjacent to other planned buildings and structures.

Planned foundations adjacent to MSE walls should be reviewed upon completion of design plans for MSE walls. Retaining walls should be designed to include both surcharged dead load and live loads from structures and vehicular traffic.

8.8 Wall Backfill Strength Parameters

Based on the NOVA's experience with similar circumstances, Table 8-6 recommends geotechnical parameters for the design of the MSE retaining walls. NOVA expects that the onsite select granular soil will meet or exceed the strength parameters presented in Table 8-6.

Table 8-6. Soil Strength Parameters for MSE Retaining Walls

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Internal Friction Angle, ϕ'	32	32	32
Cohesion, psf	0	0	0
Wet Unit Weight, pcf	130	130	130

8.8.1 Select Granular Wall Backfill

The on-site granular soils are generally acceptable for re-use as backfills for MSE retaining walls. Backfill should be comprised of a select granular soil that meets the parameters listed below:

- at least 40 percent of the material less than ¼-inches in size;
- a maximum particle size of 4 inches; and,
- an expansion index (EI) of less than 20 (as determined by ASTM D4829).

8.8.2 Limits of Backfill

Select fill 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 8-3 (following page).

As may be seen by review of Figure 8-3, the select materials should extend below the foundation of the planned wall a minimum of 3 feet below the foundation and areas to be reinforced. The retained area extends backward from the top of wall an equivalent distance to the height of the wall.

> Construction Quality Assurance
> Prior to importing the wall backfill soil, the select material should be sampled and tested to verify conformance with the minimum soil strength design parameters presented on Table 8-6.

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

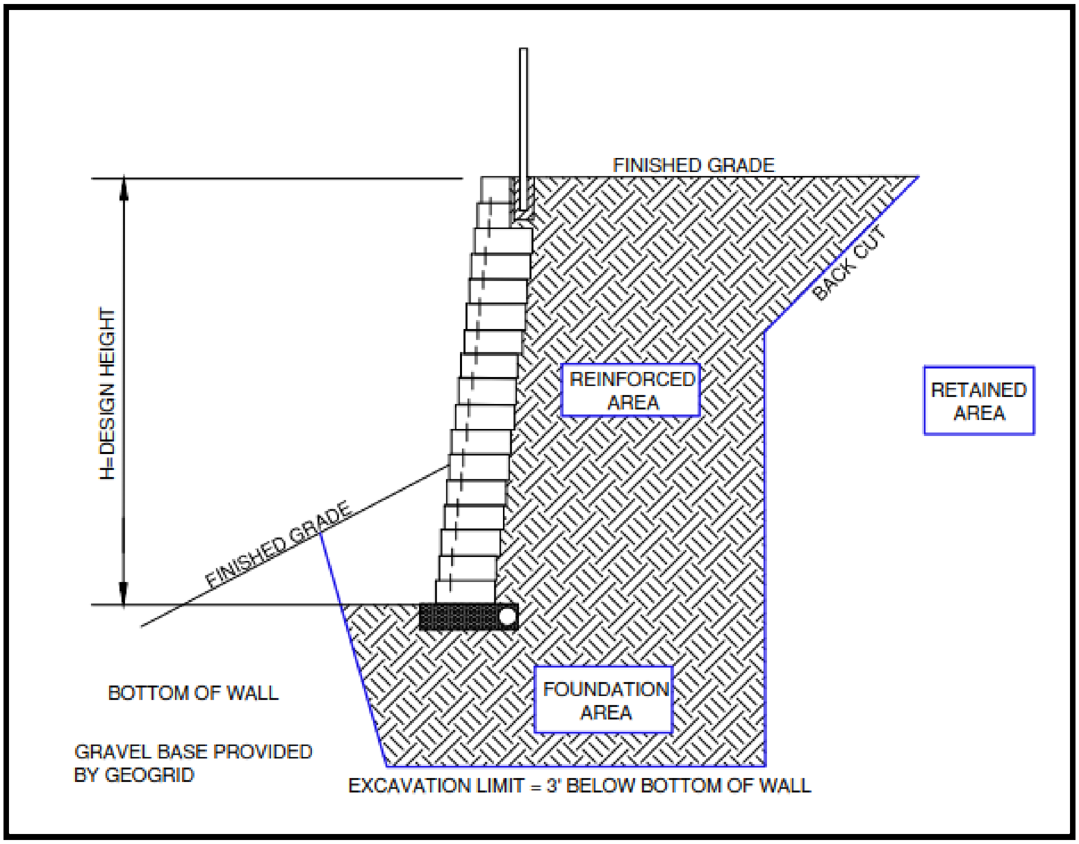

Figure 8-3. Typical Areas of Select Backfill for MSE Walls

8.8.3 Design Review

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

It is not the intent of NOVA to review the plans, calculations and documents to verify whether the design engineer has adequately utilized the design parameters. NOVA's review of the planned documents is to verify the plans are consistent with provided geotechnical recommendations and to determine if additional recommendations are necessary. Responsibility for wall design will remain with Geogrid Retaining Walls Systems, Inc.

8.9 Flatwork

Prior to casting exterior flatwork, the upper 12 inches of subgrade soils should be moisture conditioned and recompacted. The subgrade soils should be kept moist prior to casting exterior flatwork. Exterior concrete slabs for pedestrian traffic or landscaping should be at least four (4) inches thick. Weakened plane joints should be located at intervals of about 6 feet. Control of the water/cement ratio can limit shrinkage cracking due to excess water or poor concrete finishing or curing.

8.10 Temporary Slopes

8.10.1 Conformance with OSHA and Cal/OSHA

Temporary slopes may be required for excavations during grading. All temporary excavations should comply with federal, state and local safety ordinances. The safety of all excavations is the responsibility of the contractor and should be evaluated during construction as the excavation progresses.

Based on the data interpreted from the borings, the design of temporary slopes in the Unit 1 fill and Unit 2 alluvium may assume California Occupational Safety and Health Administration (Cal/OSHA) Soil Type C for planning purposes.

8.10.2 Excavation Planning and Control

The face of temporary excavations 4 feet deep or less in the Unit 1 fill or Unit 2 alluvium should not be steeper than 1:1 (horizontal : vertical).

Surcharge loads to temporary slopes should not be permitted within a distance equal to the height of the excavation measured from the top of the excavation. Excavations (i) steeper than those recommended; or, (ii) closer than 15 feet from an existing service improvement, should be shored in accordance with applicable OSHA regulations and codes.

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 materials should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation.

8.11 Infiltration Feasibility

8.11.1 Design Recommendation for Infiltration

It is the judgement of NOVA that storm water infiltration is not feasible within any of the proposed BMP locations due to unfavorable geologic conditions and the potential to create geotechnical hazards. NOVA does not recommend any infiltration into the existing fill due to geotechnical concerns about the performance of the fill, if saturated. A number of the proposed BMP locations are located very near the top of proposed slopes. Infiltration basins should not be sited within 50 feet of a slope because of the negative impacts infiltrating water will have on slope stability. Areas of the site that are not underlain by artificial fill are composed of very dense, practically impermeable, Tonalite bedrock. The tonalite is not anticipated to have infiltration rates that can support infiltration.

9.0 PAVEMENTS

9.1 Design Basis

The structural design of pavement sections depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. NOVA has assumed a Traffic Index (TI) of 5.0 for passenger car parking, and 6.0 for the driveways. These traffic indices should be confirmed by the Civil Engineer prior to final design.

9.2 Drainage and Moisture Control

Similar to the requirements for control of moisture beneath floor slabs and flatwork, control of surface drainage is important to the design and construction of pavements for this site.

Moisture must be controlled around and beneath pavements. Moreover, where standing water develops either on the pavement surface or within the base course, softening of the subgrade and other problems related to the deterioration of the pavement can be expected. Furthermore, good drainage should minimize the risk of the subgrade materials becoming saturated and weakened over a long period of time.

The following 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;
- 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 twelve inches below the base of the curb).

9.3 Preventative Maintenance

Preventative maintenance should be planned and provided for. Preventative maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment.

A plan for preventative maintenance should be comprised of both localized maintenance (e.g., crack sealing and patching) and global maintenance (e.g., surface sealing).

9.4 Subgrade Preparation

9.4.1 Grading

Preparation of subgrades for paved areas should include: (i) moisture conditioning the upper 12-inches of subgrade to about 2% above the optimum moisture content, and (ii) densification of the upper 1 foot of subgrade to at least 95% relative compaction after ASTM D 1557.

9.4.2 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.

9.4.3 Timely Base Course 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.

9.5 Flexible Pavements

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Table 9-1 provides preliminary flexible pavement sections assuming an R-value of 70. Additional R-value testing should be performed on actual soils at the design subgrade levels to confirm the pavement design.

Table 9-1. Preliminary Recommendations for Flexible Pavements, R = 70

Area	Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Auto Parking	70	5.0	3.0	6.0
Roadways	70	6.0	3.0	6.0
Heavy Traffic/Fire Lane	70	7.0	4.0	6.0

The above sections assume properly prepared subgrade consisting of at least 12-inches of subgrade densified to a minimum of 95% relative compaction at about 2% above the optimum moisture content.

The aggregate base course should also be placed at a minimum of 95% relative compaction. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction (‘Green Book’).

9.6 Rigid Pavements

9.6.1 General

Concrete pavement sections should be developed in the same manner as undertaken for all other slabs and pavements: removal of the upper 12-inches of the Unit 1 fill and replacement of that material in an engineered manner as described in Section 9.2.

Concrete pavement sections consisting of 7-inches of Portland cement concrete over a base course of 6-inches and a properly prepared subgrade support a wide range of traffic indices.

Where rigid pavements are used, the concrete should be obtained from an approved mix design with the minimum properties of Table 9-2.

Table 9-2. Recommended Concrete Requirements

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 I Portland
Concrete Aggregate	ASTM C33 and Caltrans Section 703
Aggregate Size	1-inch maximum
Maximum Water Content	0.50 lb/lb of cement
Maximum Allowable Slump	4-inches

9.6.2 Jointing and Reinforcement

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.

Load transfer devices, such as dowels or keys are recommended at joints in the paving to reduce possible offsets. 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.

10.0 CONSTRUCTION REVIEW, OBSERVATION, AND TESTING

10.1 Overview

As is discussed in Section 1, the recommendations contained in this report are based upon an evaluation of the previous subsurface explorations and an assumption of general continuity of subsurface conditions between test pits.

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 construction, and to provide construction monitoring in the form of testing and observation.

10.2 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.

10.3 Construction Observation and Testing

10.3.1 General

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 Geotechnical Engineer-of-Record (GEOR).

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

- Surveillance during site preparation, grading, and foundation excavation.
- Surveillance of the ground improvement described in Section 6.
- Soil special inspection during grading and ground densification by deep dynamic compaction.

A program of quality control should be developed prior to the beginning of earthwork. 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.

10.3.2 Continuous Soils Special Inspection

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

- Site grading, including scarification and re-compaction an fill placement.
- Ground improvement as described in Section 6.
- Pavement subgrade preparation and base course compaction.

10.3.3 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.
- Observation of foundation excavations.

10.3.4 Testing During Inspections

A preconstruction conference among representatives of the Owner, Contractor and/or Construction Manager and Geotechnical Engineer is recommended to discuss the planned construction procedures and quality control requirements.

The locations and frequencies of compaction testing 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.

11.0 REFERENCES

11.1 Site Specific

11.1.1 Architectural

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11.1.2 Civil

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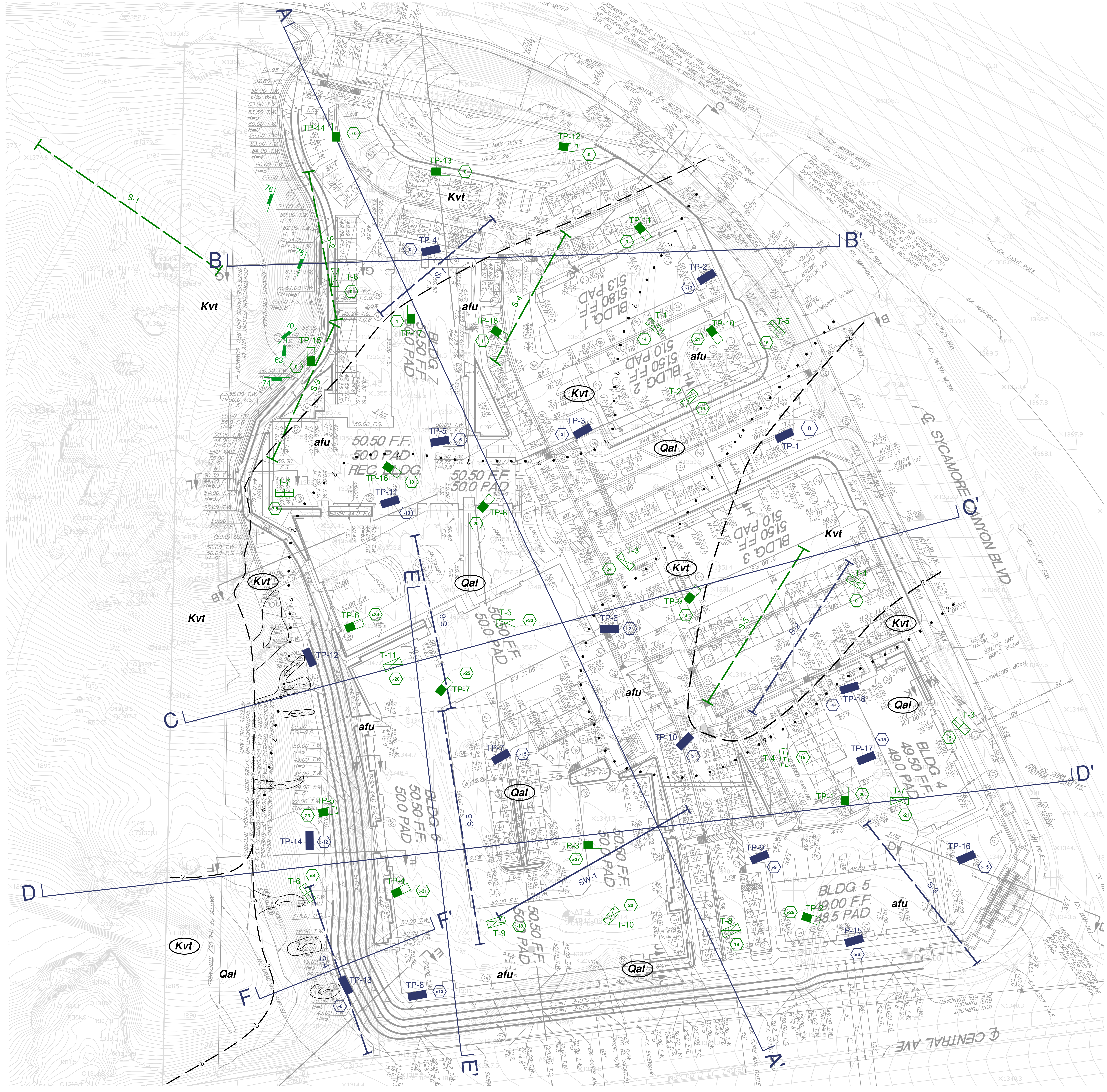
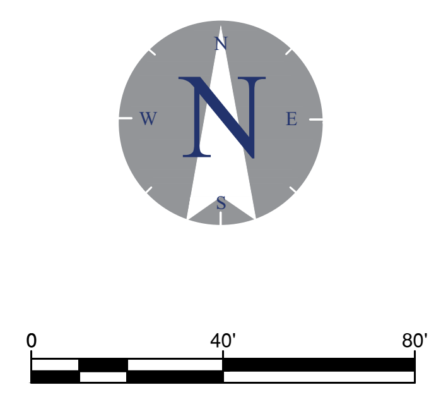
PLATES





KEY TO SYMBOLS

afu	UNDOCUMENTED ARTIFICIAL FILL
Qal	QUATERNARY ALLUVIAL DEPOSITS
Kvt	CRETACEOUS VAL VERDE TONALITE
• • ? — —	GEOLOGIC CONTACT, DOTTED WHERE BURIED, QUERIED WHERE UNCERTAIN
TP-18 ■	TEST PIT
TP-18 ■	TEST PIT (GEOCON, 2018)
T-11 ☒	TEST PIT (ALBUS-KEEFE & ASSOCIATES, INC, 2007)
T-7 ⊞	TEST PIT (BYERLY, 1997)
75	JOINT ATTITUDE (ALBUS-KEEFE & ASSOCIATES, INC, 2007)
S-6	SEISMIC REFRACTION LINE
SW-1	SEISMIC SHEAR WAVE LINE
S-5	SEISMIC REFRACTION LINE (ALBUS-KEEFE & ASSOCIATES, INC, 2007)
F F'	ALIGNMENT OF GEOTECHNICAL CROSS-SECTION
>15	APPROXIMATE DEPTH TO BEDROCK

*NOTE: T-1 & T-2 (BYERLY, 1997) ARE LOCATED OFFSITE.

SEAL:
REGISTERED PROFESSIONAL ENGINEER
DANE A. SOMMERS
No. 90433
Exp. 9-30-21
CIVIL
STATE OF CALIFORNIA

SDH & ASSOCIATES INCORPORATED
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14060 Meridian Parkway 102
Riverside, California 92518
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CRESTVIEW APARTMENTS
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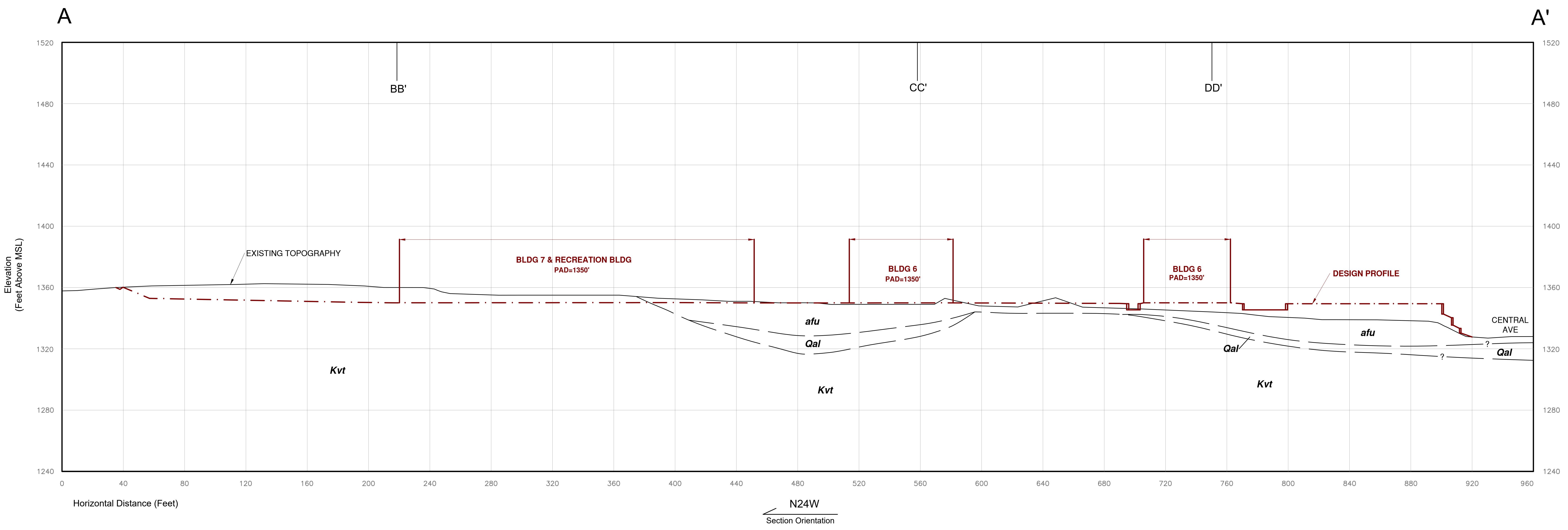
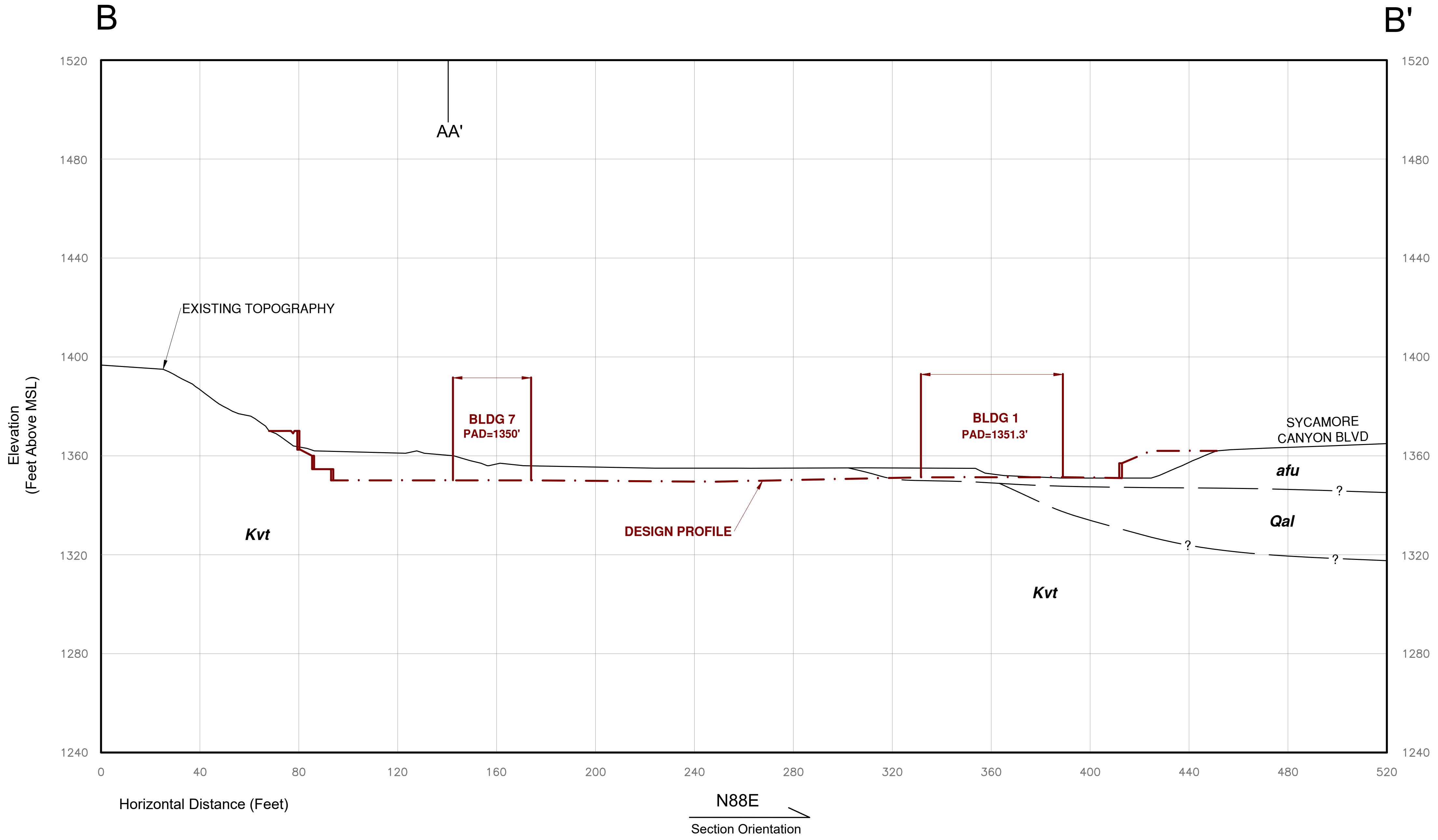
PROJECT NO.:	3020003
DATE:	SEPTEMBER 2020
DRAWN BY:	DTW
REVIEWED BY:	CNJ
SCALE:	1"=40'
DRAWING TITLE:	

GEOTECHNICAL MAP

PLATE NO. 1 OF 5





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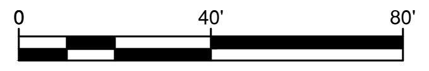

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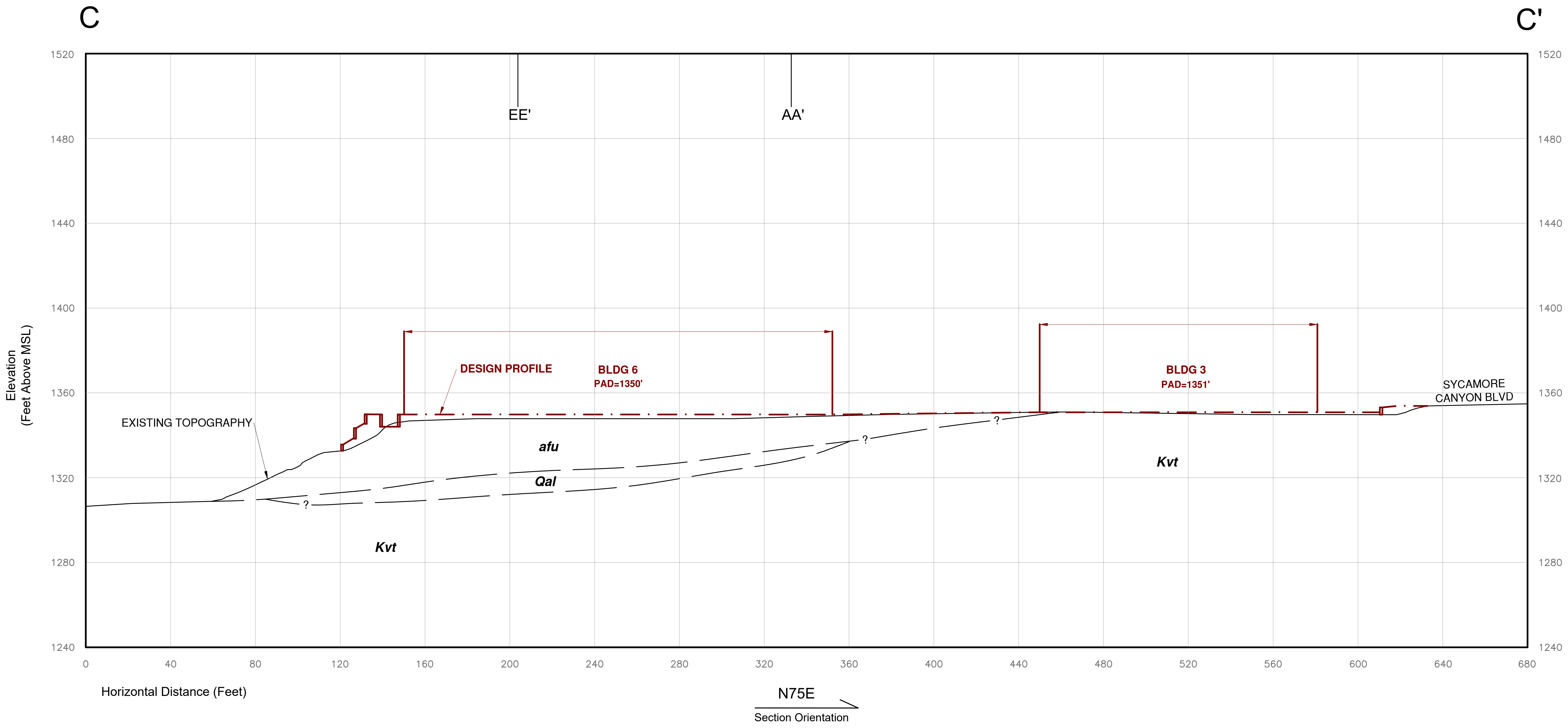
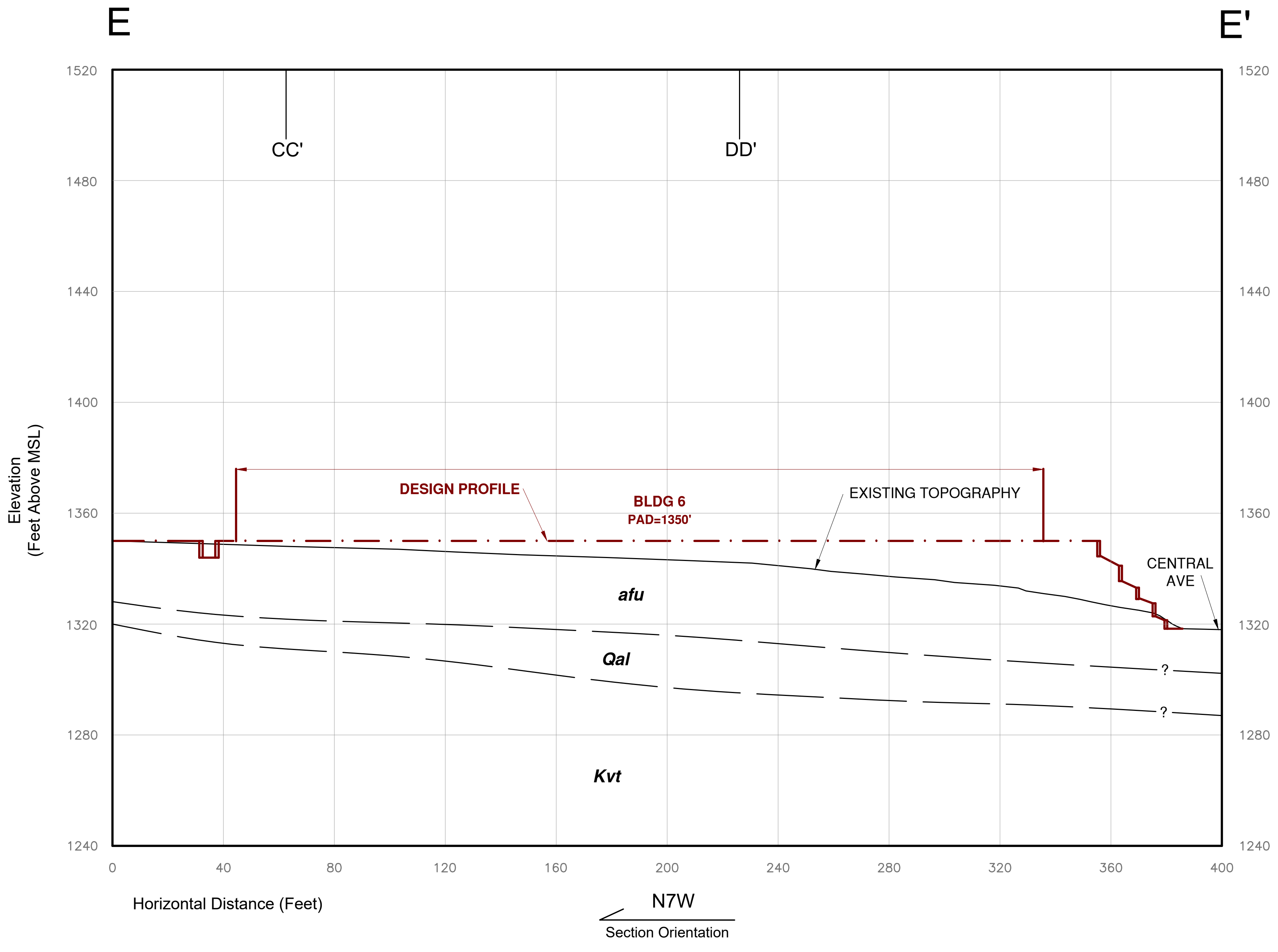
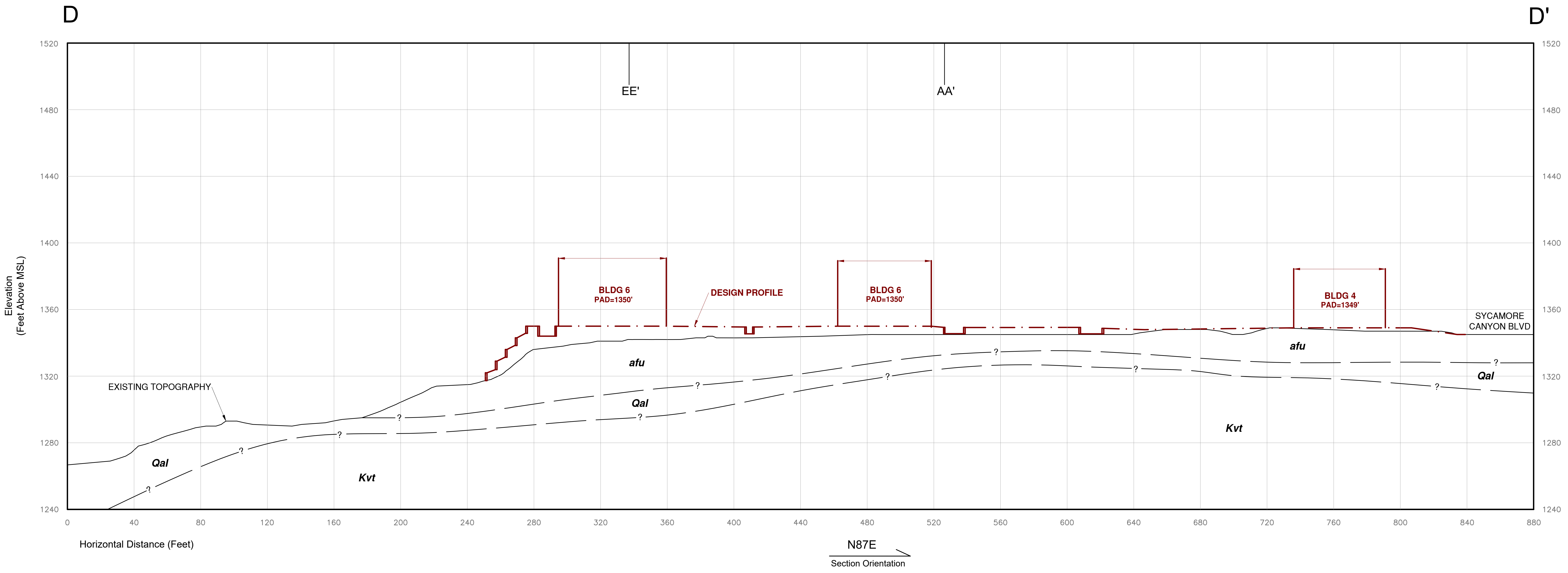
PROJECT NO.: 3020003
DATE: SEPTEMBER 2020
DRAWN BY: DTW
REVIEWED BY: CNJ
SCALE: 1"=40'
DRAWING TITLE:

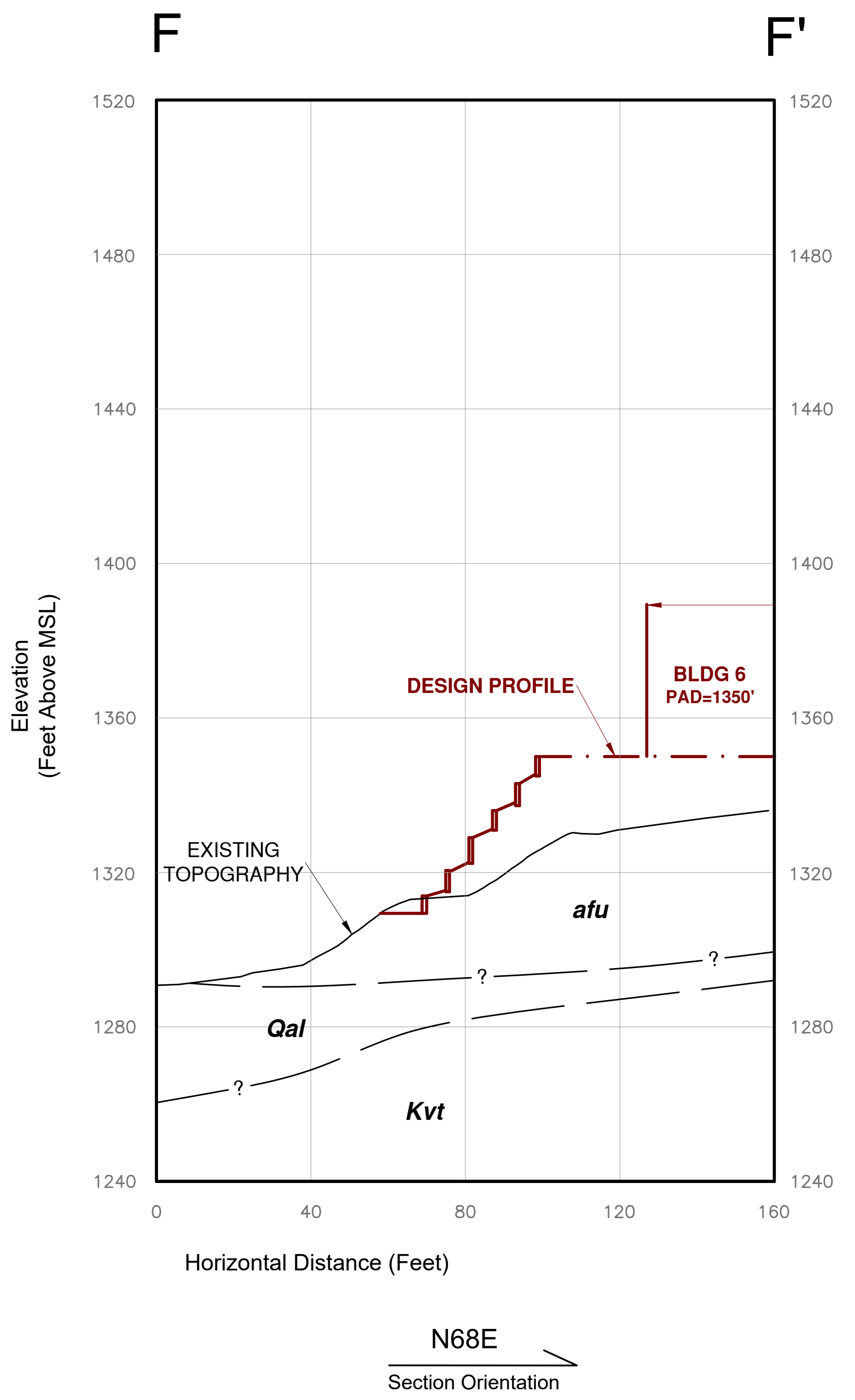
GEOTECHNICAL
CROSS-SECTIONS AA' & BB'

PLATE NO. 2 OF 5









KEY TO SYMBOLS

afu	UNDOCUMENTED ARTIFICIAL FILL
Qal	QUATERNARY ALLUVIAL DEPOSITS
Kvt	CRETACEOUS VAL VERDE TONALITE
—?—	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

0 40' 80'

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DATE:	SEPTEMBER 2020
DRAWN BY:	DTW
REVIEWED BY:	CNJ
SCALE:	1"=40'
DRAWING TITLE:	

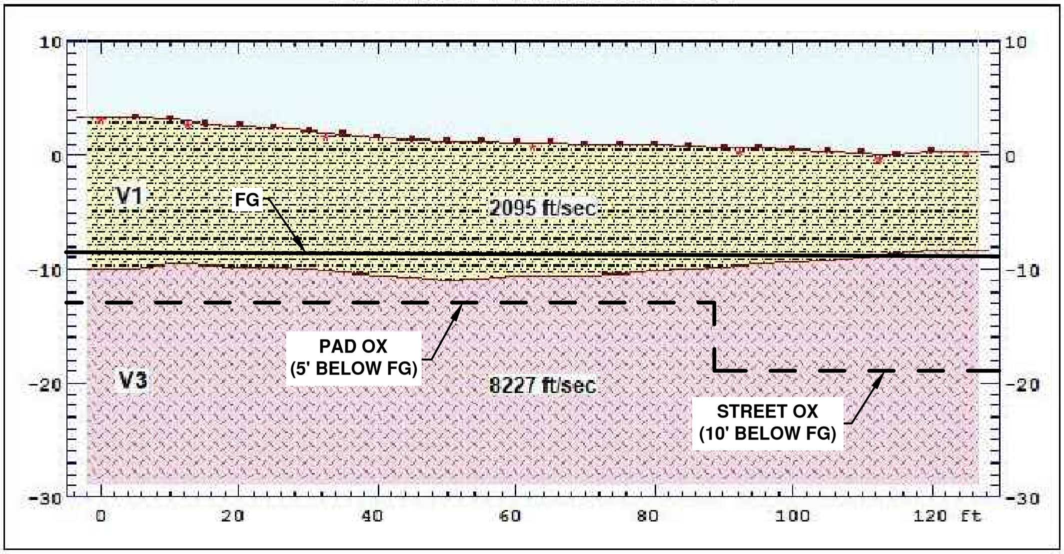
GEOTECHNICAL
CROSS-SECTIONS CC', DD',
EE', & FF'

PLATE NO. 3 OF 5

SEISMIC LINE S-1

North 51° East >

LAYER VELOCITY MODEL



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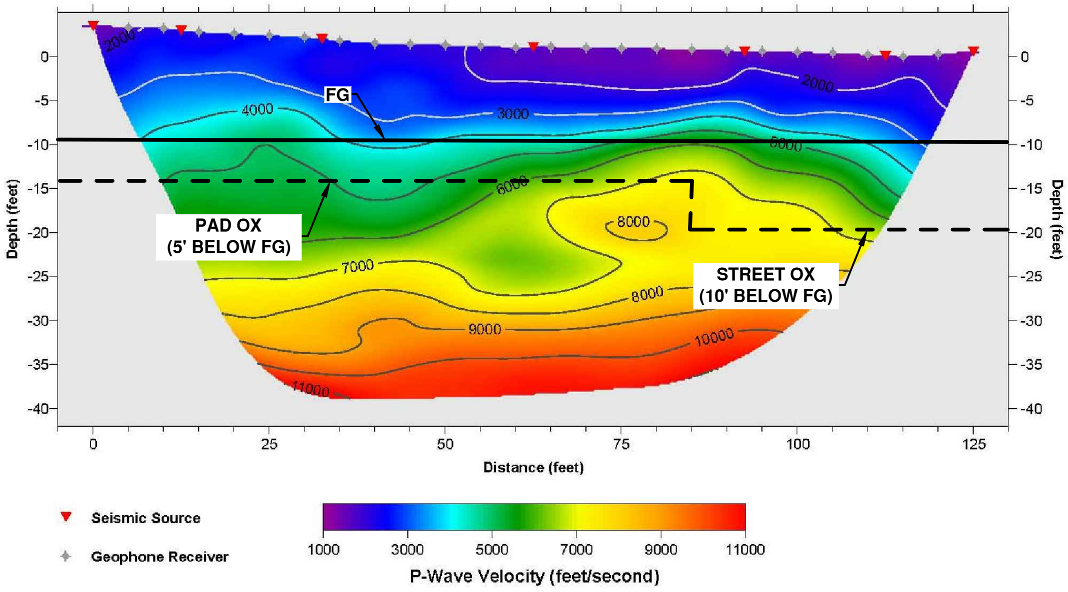

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SEISMIC REFRACTION LINE S-1

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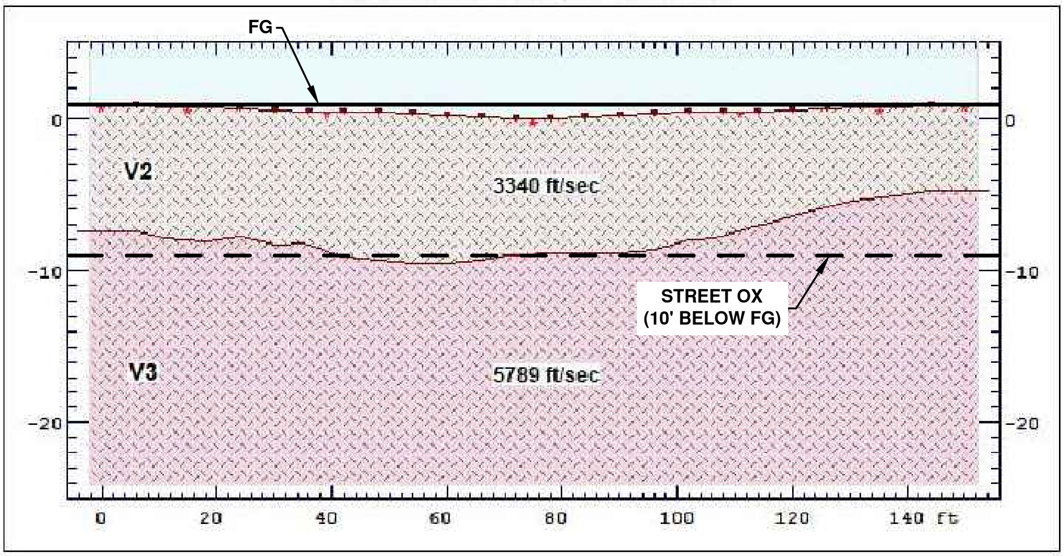
PLATE NO. 4A

SEISMIC LINE S-2

North 33° East >

LAYER VELOCITY MODEL



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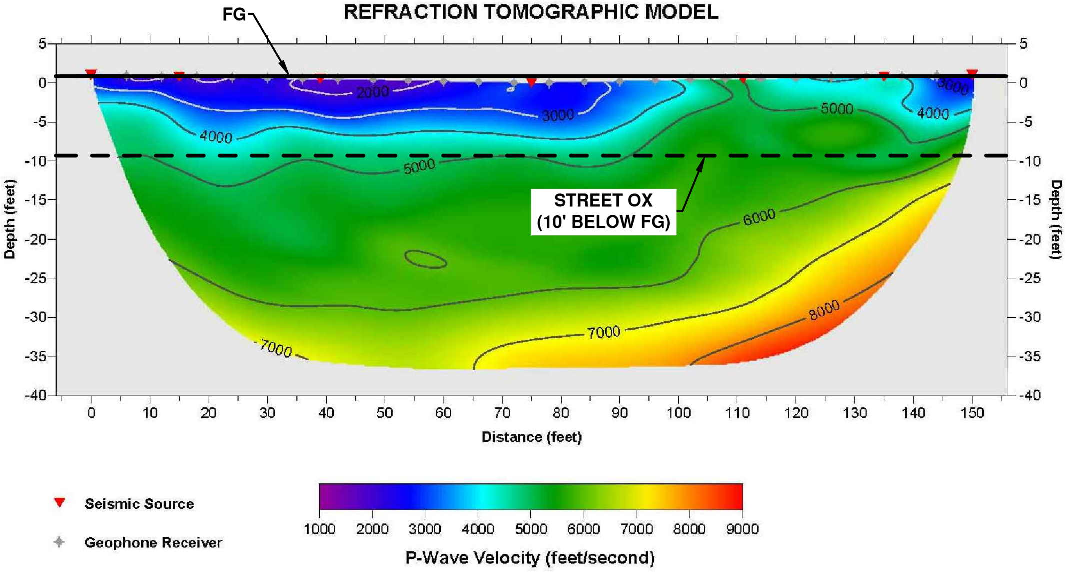

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SEISMIC REFRACTION LINE S-2

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PROJECT: 3020003

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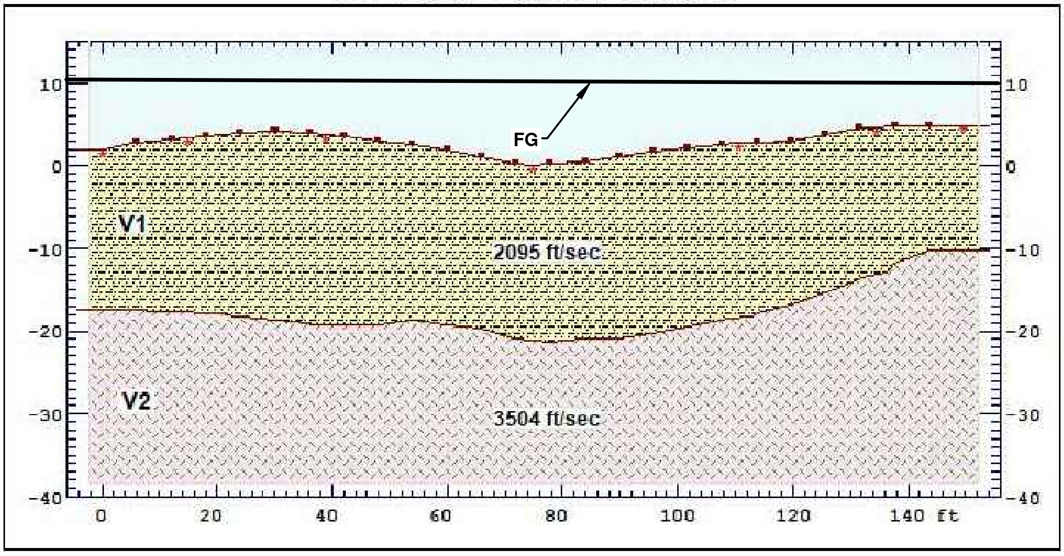
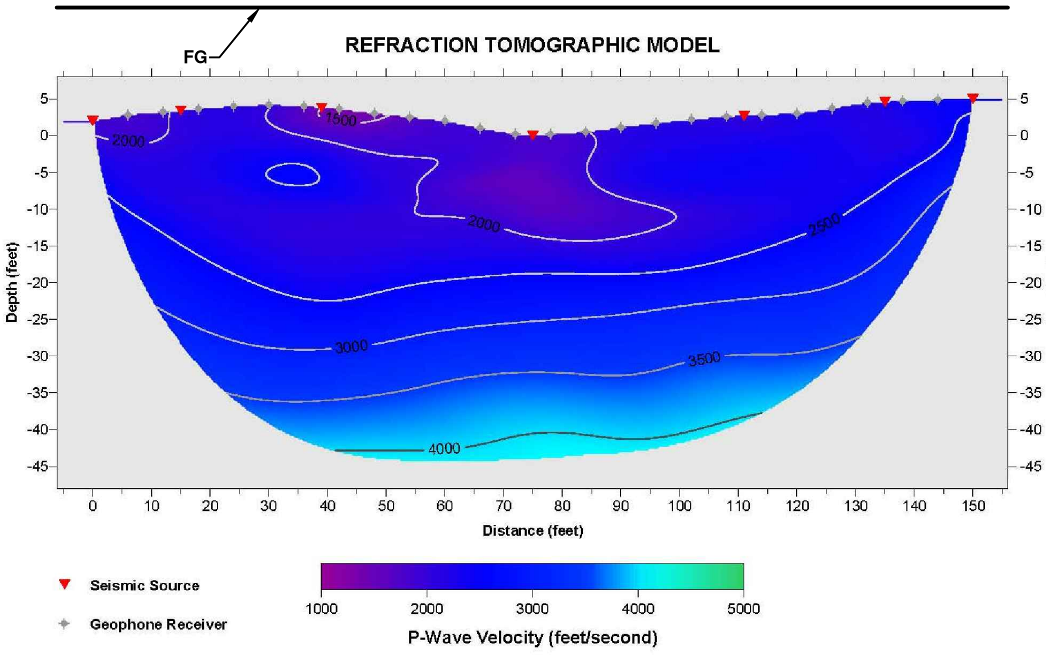
PLATE NO. 4B

SEISMIC LINE S-3

North 38° West >

LAYER VELOCITY MODEL





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SEISMIC REFRACTION LINE S-3

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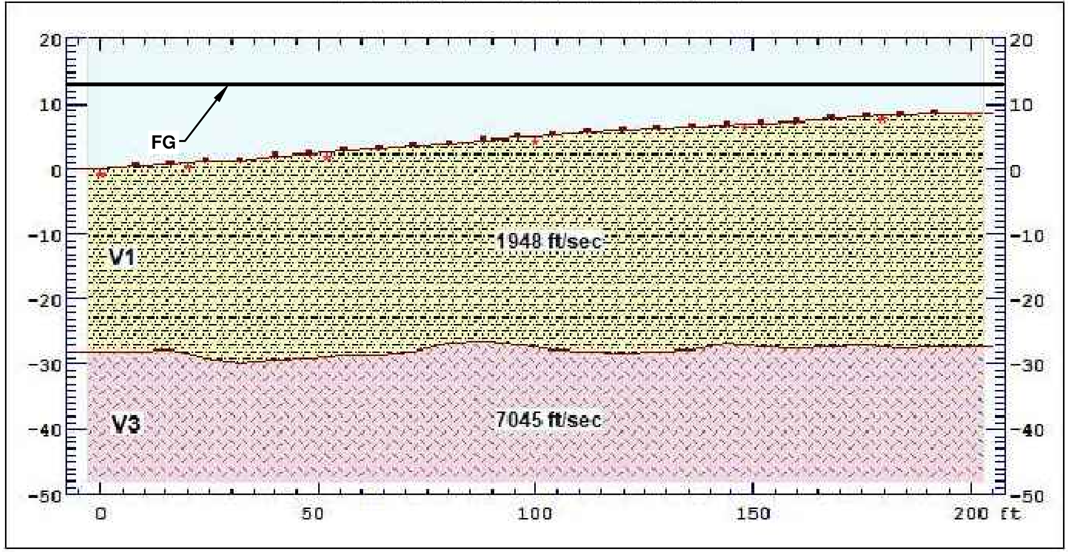
PROJECT: 3020003	DATE: SEPTEMBER 2020	PLATE NO. 4C

SEISMIC LINE S-5

North 9° West >

LAYER VELOCITY MODEL



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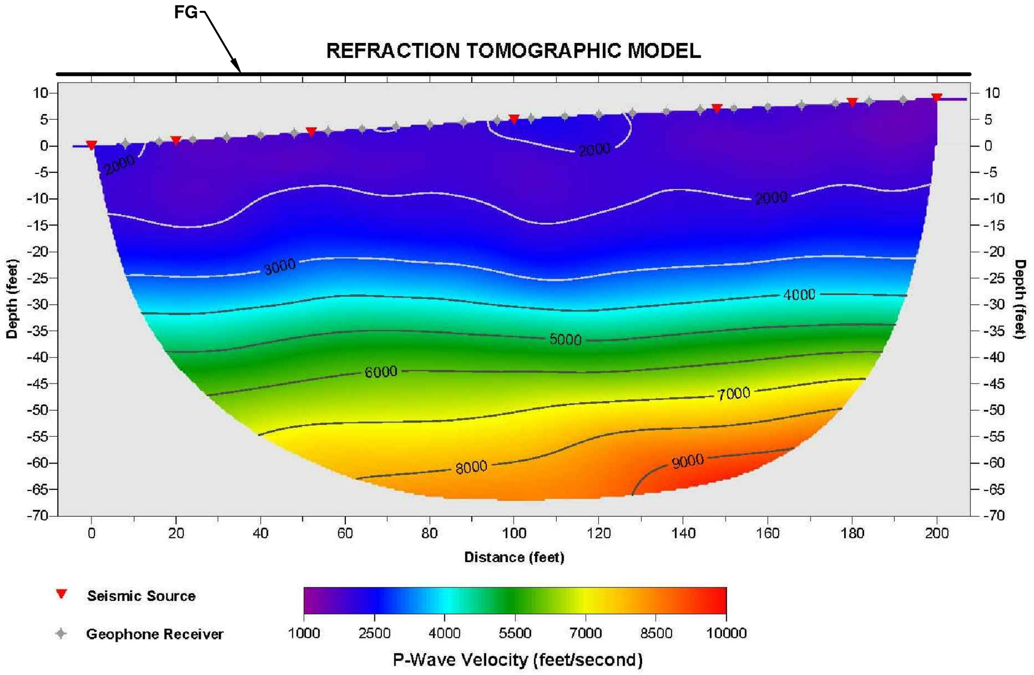

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SEISMIC REFRACTION LINE S-5

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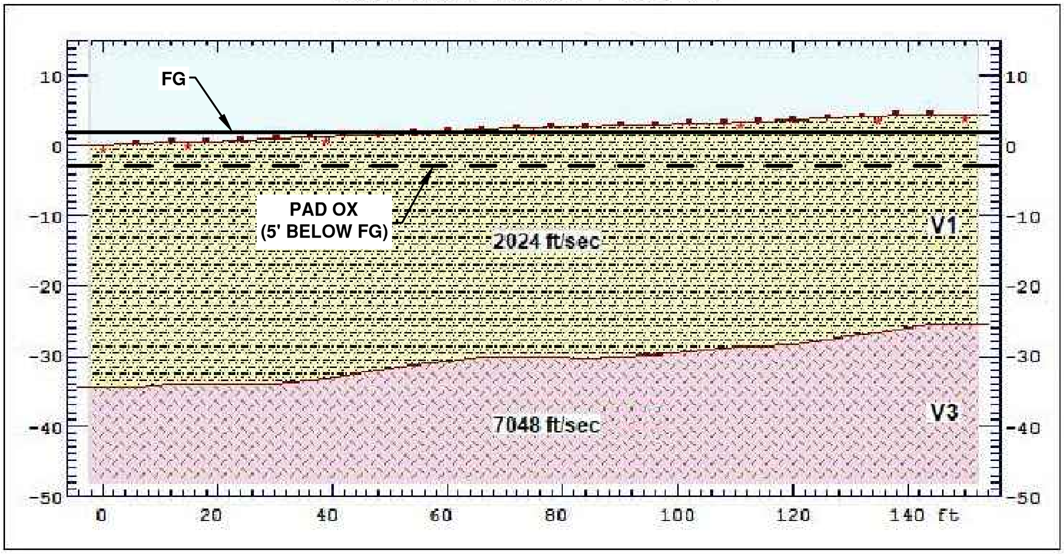
DATE: SEPTEMBER 2020

PLATE NO. 4D

SEISMIC LINE S-6

North 12° West >

LAYER VELOCITY MODEL



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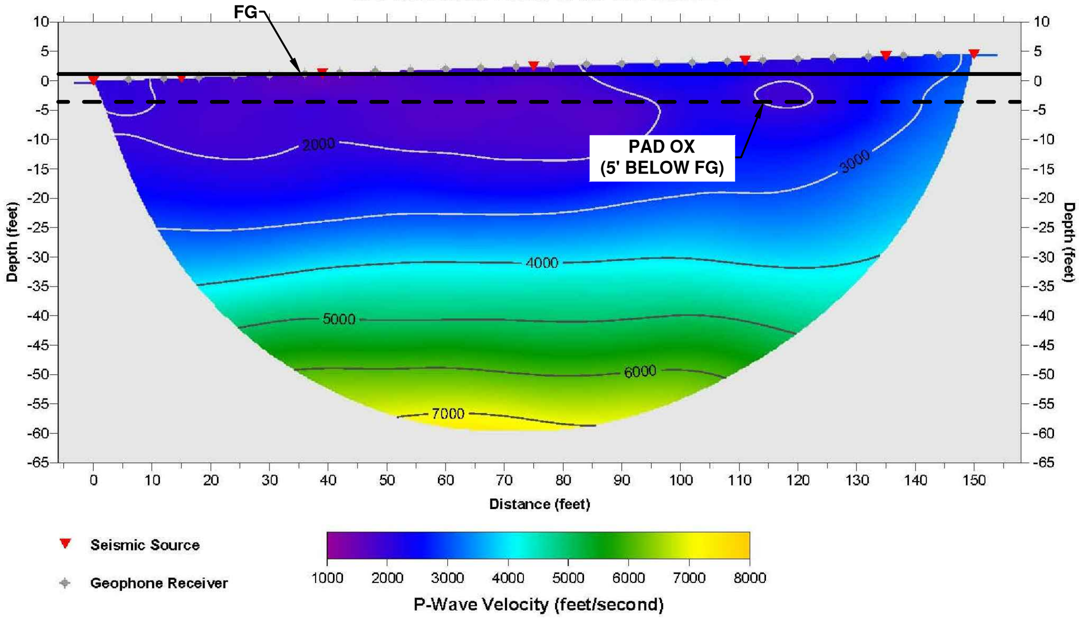

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SEISMIC REFRACTION LINE S-6

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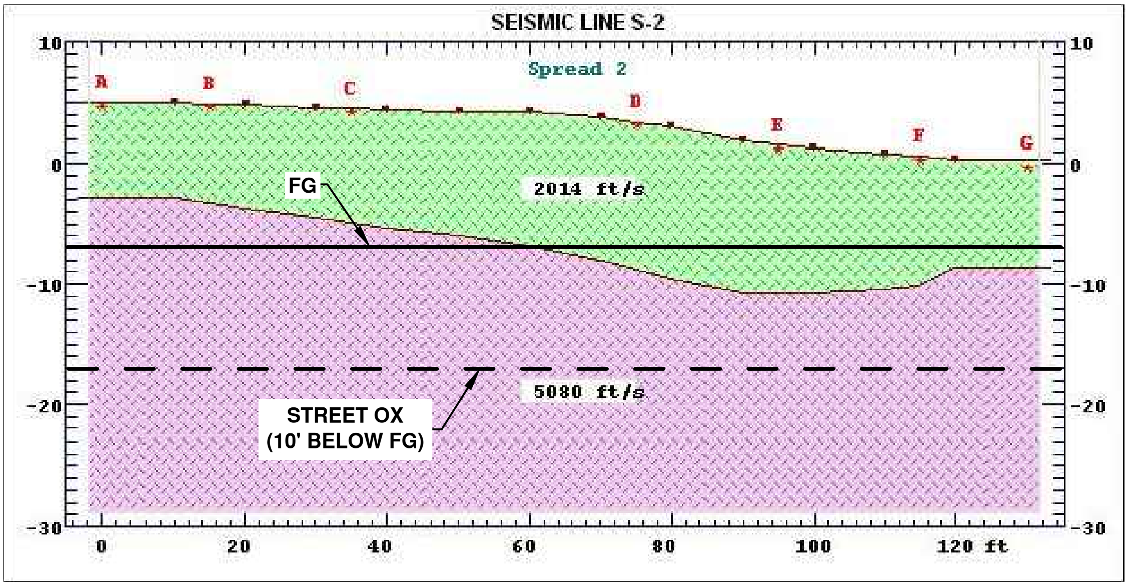
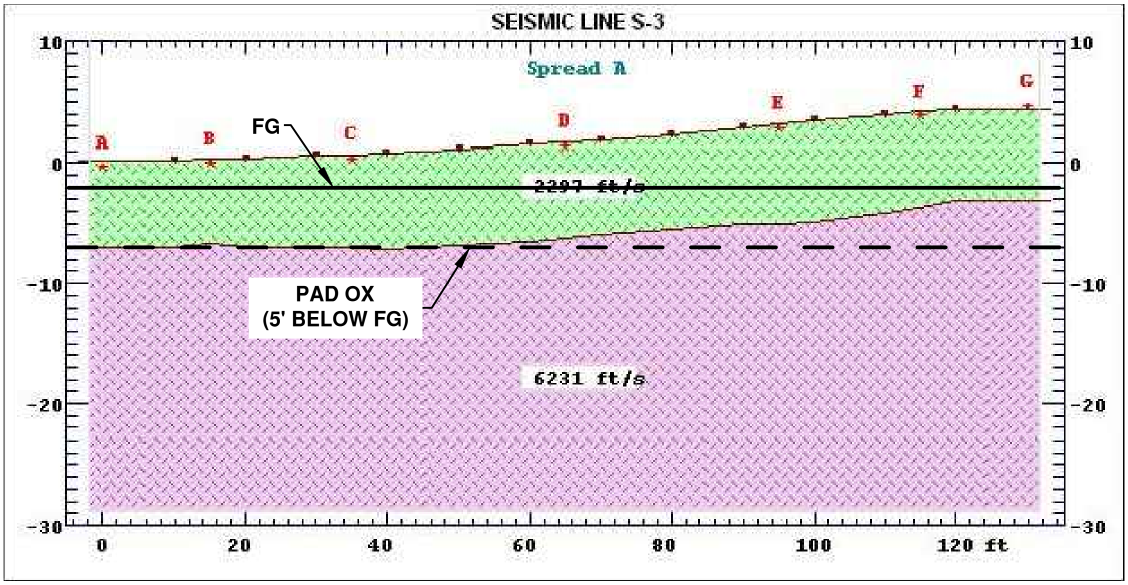
PROJECT: 3020003	DATE: SEPTEMBER 2020	PLATE NO. 4E





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SEISMIC REFRACTION LINES S-2 & S-3 (AKA 2007)

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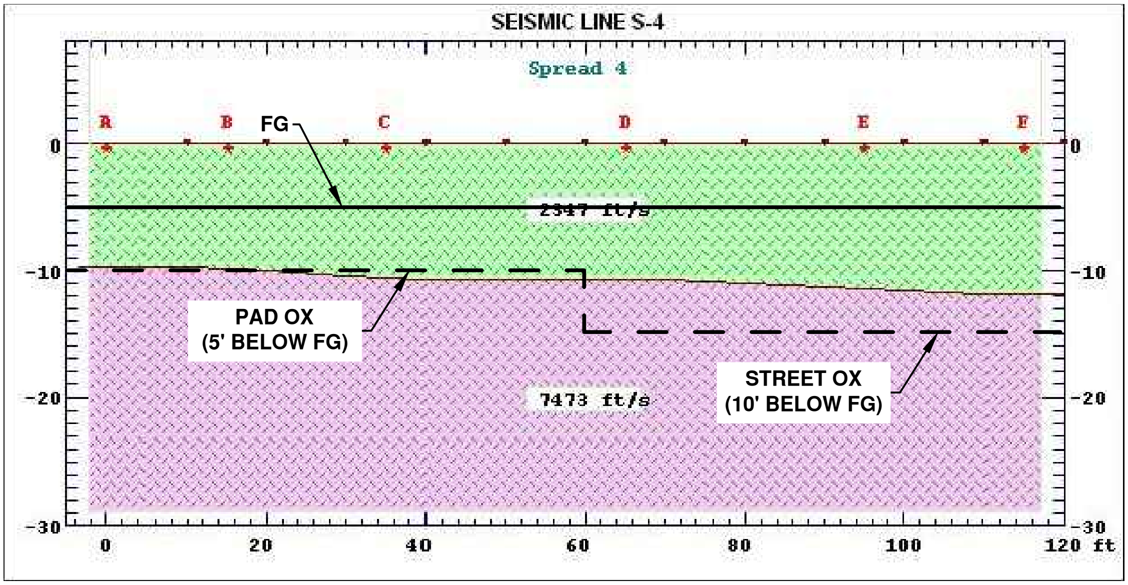
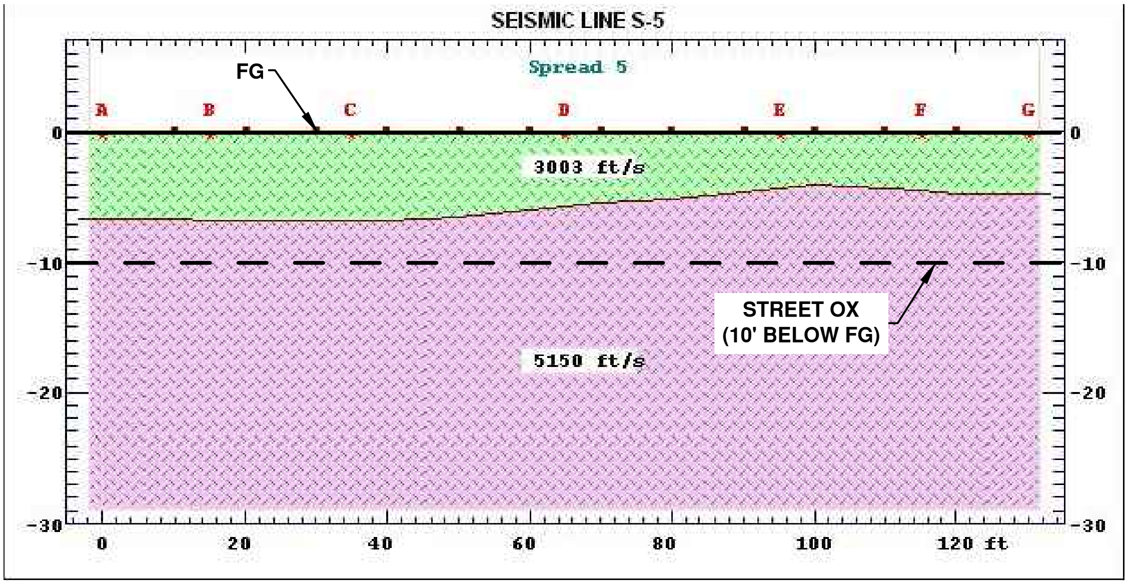
PROJECT: 3020003

DATE: SEPTEMBER 2020

PLATE NO. 5A





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SEISMIC REFRACTION LINES S-4 & S-5 (AKA 2007)

CRESTVIEW APARTMENTS
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RIVERSIDE, CALIFORNIA

PROJECT: 3020003	DATE: SEPTEMBER 2020	PLATE NO. 5B

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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Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

APPENDIX B

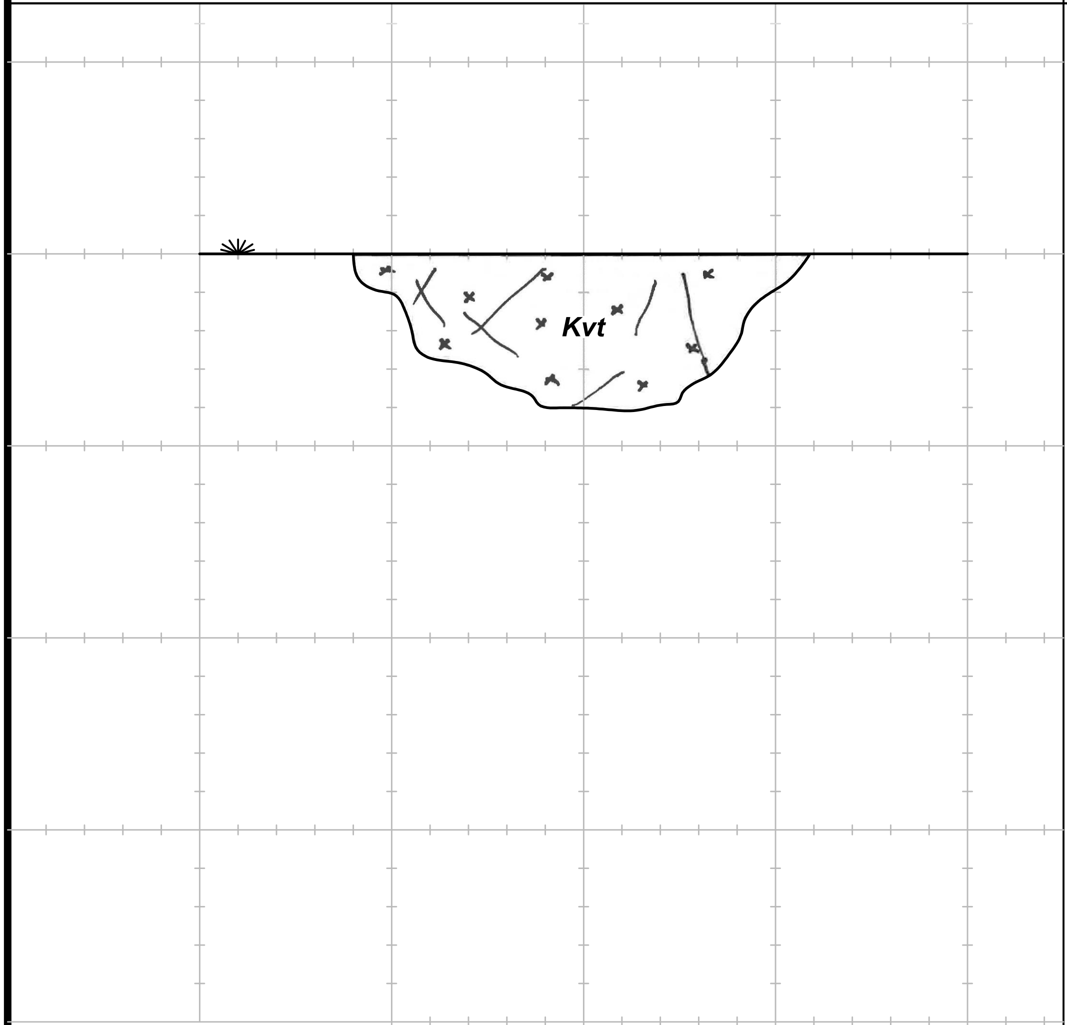
LOGS OF EXPLORATORY EXCAVATIONS

PROJECT NAME: CRESTVIEW APARTMENTS
DATE: 8/18/2020
TEST PIT NO.: TP-1
SURFACE SLOPE: 0°

PROJECT NO.: 3020003
LOGGED BY: CNJ
ELEVATION: 1350'
GROUNDWATER: NONE

EQUIPMENT: CAT 320 EXCAVATOR
REVIEWED BY: CNJ
TREND: N15E
SCALE: 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
CRETACEOUS VAL VERDE TONALITE (Kvt): *@0' TONALITE EXCAVATES TO SAND WITH SILT AND GRAVEL AND COBBLES UP TO 12" IN MAXIMUM DIMENSION, DRY, VERY DENSE, MODERATELY WEATHERED TO FRESH BY 4', JOINTED* *@3' JOINT ATTITUDE: N42E,82S* *@4' REFUSAL*	SP-SM	B-1 @ 0'-5'

TOTAL DEPTH: 4'

BACKFILLED: YES

COMPACTED: NO

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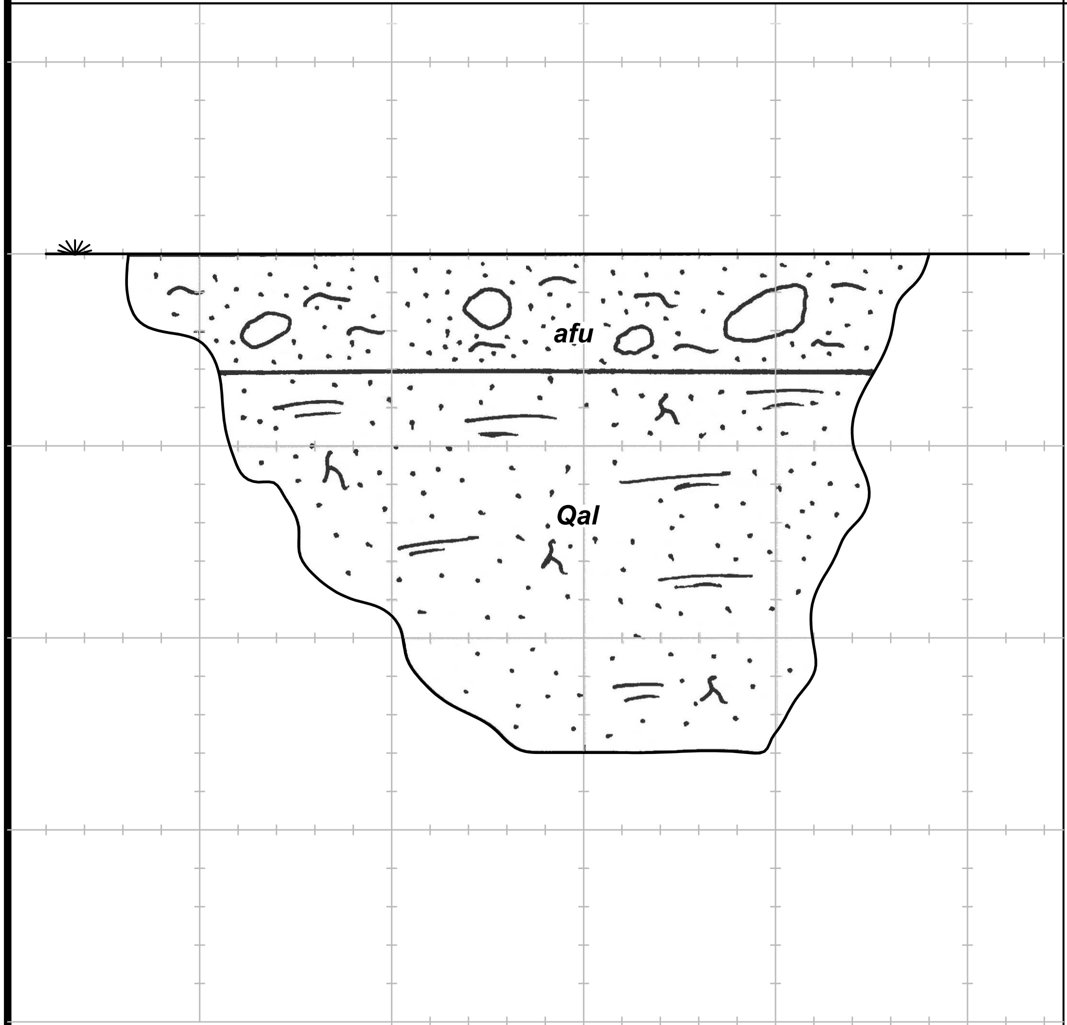
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-2	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1349'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N40W	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH DECOMPOSED TONALITE COBBLES AND BOULDERS UP TO 2' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, MEDIUM DENSE*	SM	
QUATERNARY ALLUVIAL DEPOSITS (Qal): *@3' CLAYEY SAND; REDDISH BROWN, SLIGHTLY MOIST TO MOIST, MEDIUM DENSE, ROOTLETS, ROOTS, AND PINHOLE POROSITY*	SC	B-1 @ 5'-8'
@7' DECREASE MOISTURE TO SLIGHTLY MOIST		

TOTAL DEPTH: 13'

BACKFILLED: YES

COMPACTED: NO

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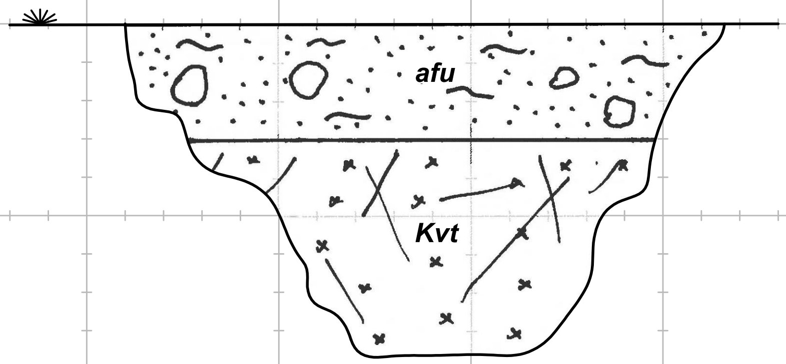
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-3	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1351'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N55E	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH COBBLES AND BOULDERS UP TO 1' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, LOOSE TO MEDIUM DENSE*	SM	
CRETACEOUS VAL VERDE TONALITE (Kvt): *@3' TONALITE EXCAVATES TO SAND WITH SILT; LIGHT GRAY, DRY, VERY DENSE, JOINTED, MODERATELY WEATHERED TO FRESH BY 8'*	SP	
@8.5' REFUSAL		

TOTAL DEPTH: 8.5'

BACKFILLED: YES

COMPACTED: NO

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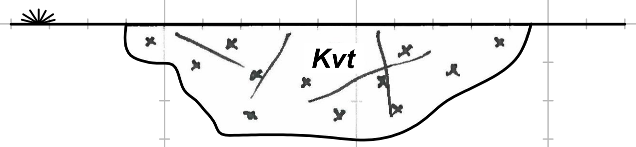
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-4	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1355'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N80W	**SCALE:** 1"=5"

GRAPHICAL REPRESENTATION BELOW:	**SOIL DESCRIPTION:**	**USCS**	**SAMPLE NO.**
	**CRETACEOUS VAL VERDE TONALITE (Kvt):** *@0' TONALITE EXCAVATES TO COBBLES AND BOULDERS UP TO 2.5' IN MAXIMUM DIMENSION; ANGULAR, DRY, VERY DENSE, JOINTED, MODERATELY WEATHERED TO FRESH* *@3' REFUSAL*	GR	

TOTAL DEPTH: 3'

BACKFILLED: YES

COMPACTED: NO

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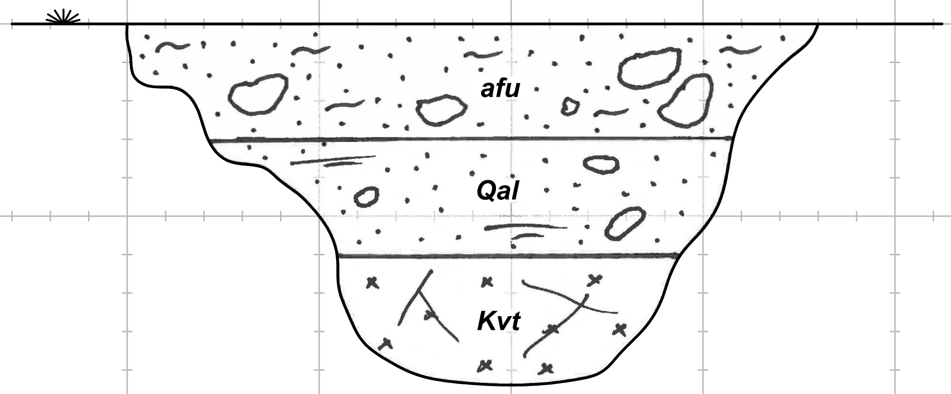
PROJECT NAME: CRESTVIEW APARTMENTS
DATE: 8/18/2020
TEST PIT NO.: TP-5
SURFACE SLOPE: 0°

PROJECT NO.: 3020003
LOGGED BY: CNJ
ELEVATION: 1354'
GROUNDWATER: NONE

EQUIPMENT: CAT 320 EXCAVATOR
REVIEWED BY: CNJ
TREND: N10W
SCALE: 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH COBBLES AND BOULDERS UP TO 1.5' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, LOOSE TO MEDIUM DENSE*	SM	
QUATERNARY ALLUVIAL DEPOSITS (Qal): *@3' SAND WITH SOME CLAY AND SCATTERED COBBLES; REDDISH BROWN, MOIST, MEDIUM DENSE*	SP	
CRETACEOUS VAL VERDE TONALITE (Kvt): *@6' TONALITE EXCAVATES TO GRAY SAND AND COBBLES; SLIGHTLY MOIST, VERY DENSE, JOINTED, VERY WEATHERED*	SP	
@9.5' REFUSAL		

TOTAL DEPTH: 9.5'

BACKFILLED: YES

COMPACTED: NO

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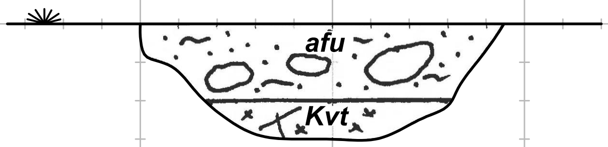
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-6	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1350'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N45W	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH COBBLES AND BOULDERS; REDDISH BROWN, DRY, MEDIUM DENSE TO DENSE, INDURATED*	SM	
CRETACEOUS VAL VERDE TONALITE (Kvt): *@2' TONALITE EXCAVATES TO SAND WITH SILT; LIGHT GRAY, DRY, VERY DENSE, MODERATELY WEATHERED*	SP	
@3' REFUSAL		

TOTAL DEPTH: 3'

BACKFILLED: YES

COMPACTED: NO

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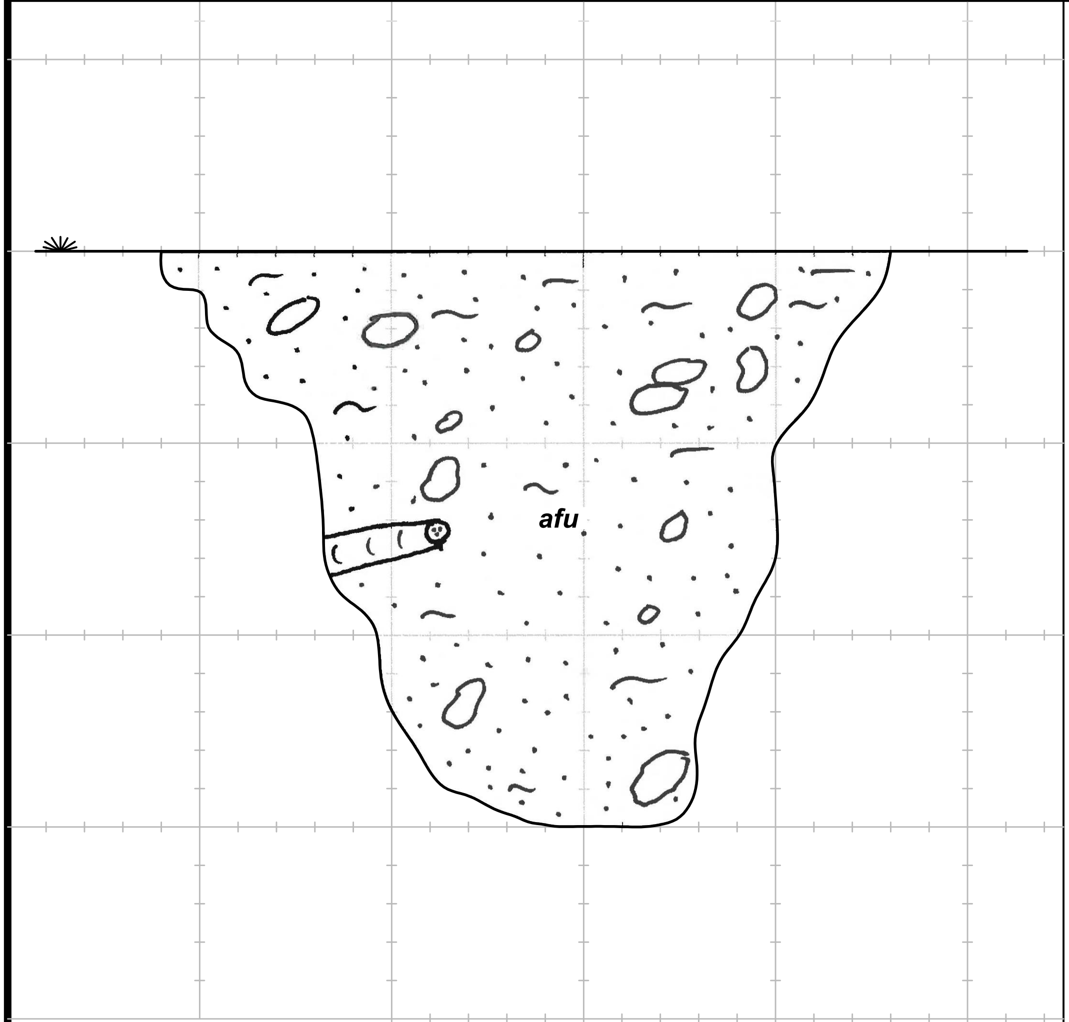
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-7	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1345'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** EW	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



TOTAL DEPTH: 15'

BACKFILLED: YES

COMPACTED: NO

SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH COBBLES AND BOULDERS UP TO 2' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, LOOSE TO MEDIUM DENSE*	SM	
@2' SAND WITH SILT AND COBBLES AND BOULDERS; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE TO DENSE	SP-SM	B-1 @ 0'-5'
@7' BROKEN, GREEN, PLASTIC, CORRUGATED PIPE, 12" DIAMETER, FILLED IN WITH SOIL		

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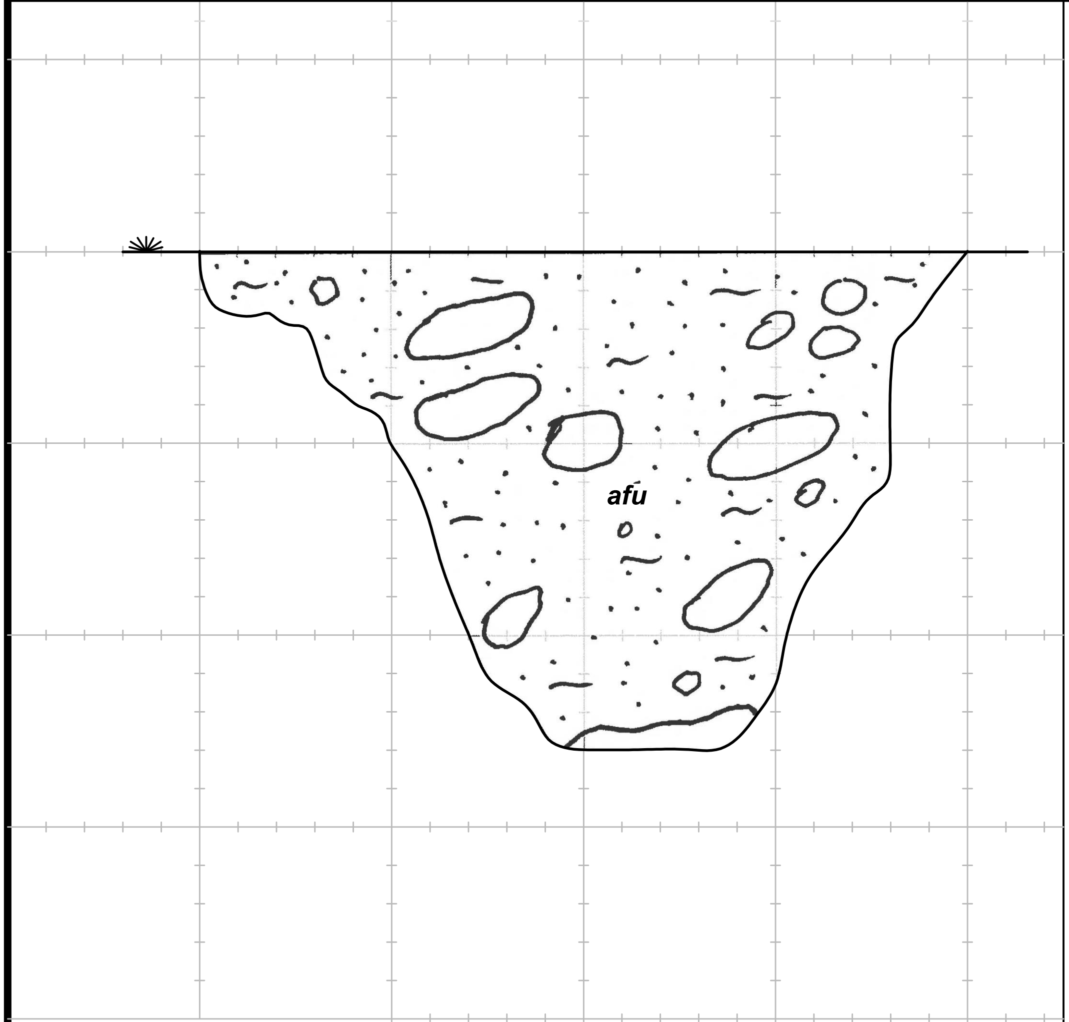
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-8	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1329'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** NS	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND; LIGHT BROWN, DRY, MEDIUM DENSE, WITH BOULDERS UP TO 4' IN MAXIMUM DIMENSION* *@4' INCREASED MOISTURE TO SLIGHTLY MOIST* *@13' REFUSAL DUE TO BOULDER*	SM	

TOTAL DEPTH: 13'

BACKFILLED: YES

COMPACTED: NO

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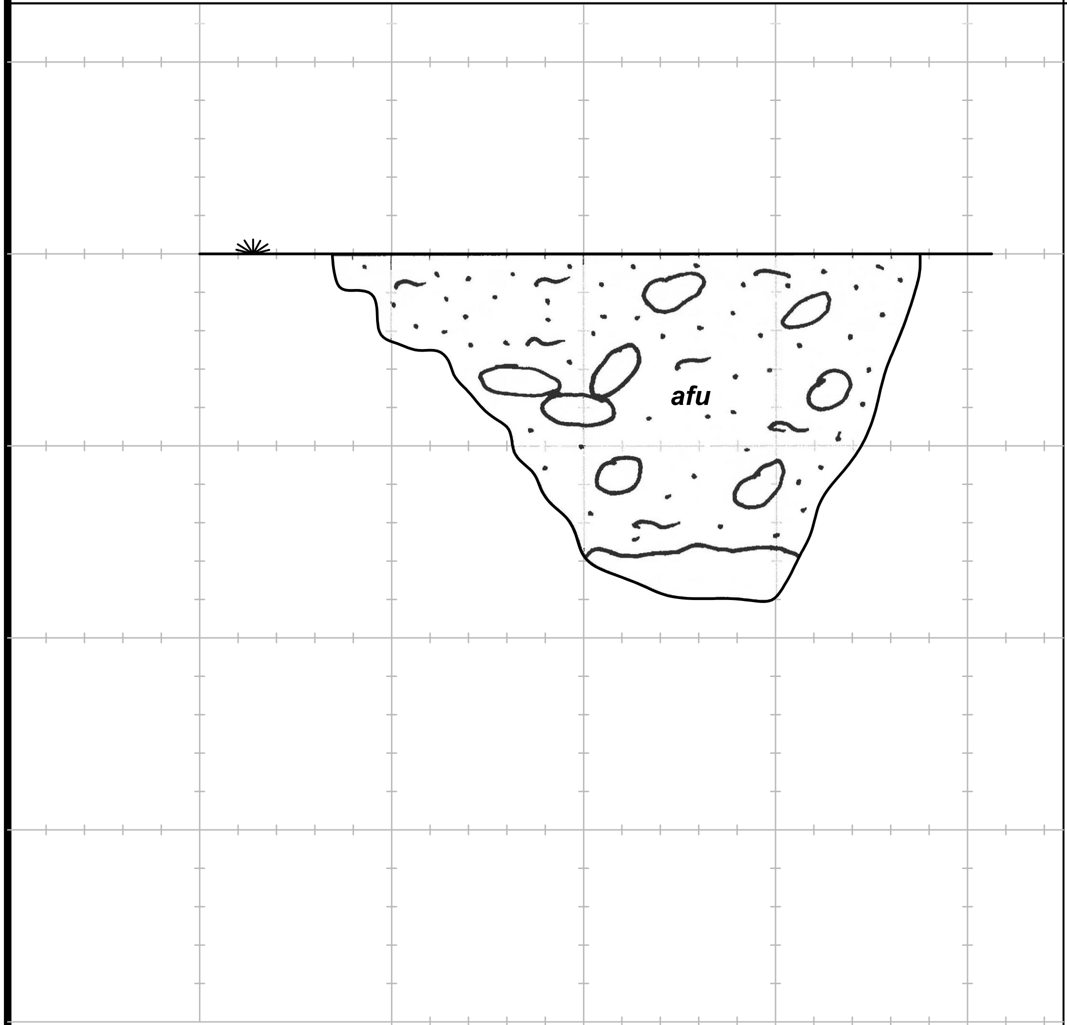
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-9	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1341'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N60W	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



TOTAL DEPTH: 9'

BACKFILLED: YES

COMPACTED: NO

SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH BOULDERS UP TO 2' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, MEDIUM DENSE* *@4' INCREASED MOISTURE TO SLIGHTLY MOIST,* *@9' REFUSAL DUE TO BOULDER OVER 5' IN MAXIMUM DIMENSION*	SM	

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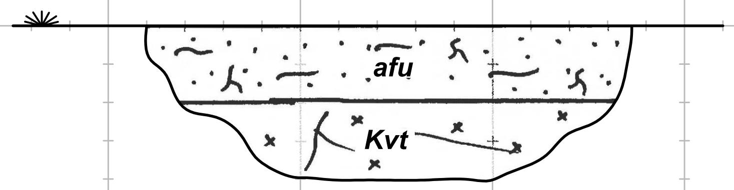
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/18/2020	**TEST PIT NO.:** TP-10	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1346'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N45E	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND; REDDISH BROWN, DRY, MEDIUM DENSE, ROOTLETS*	SM	
CRETACEOUS VAL VERDE TONALITE (Kvt): *@2' TONALITE EXCAVATES TO SAND WITH SILT; LIGHT GRAY, DRY, VERY DENSE, SLIGHTLY WEATHERED* *@4' REFUSAL*	SP	

TOTAL DEPTH: 4'

BACKFILLED: YES

COMPACTED: NO

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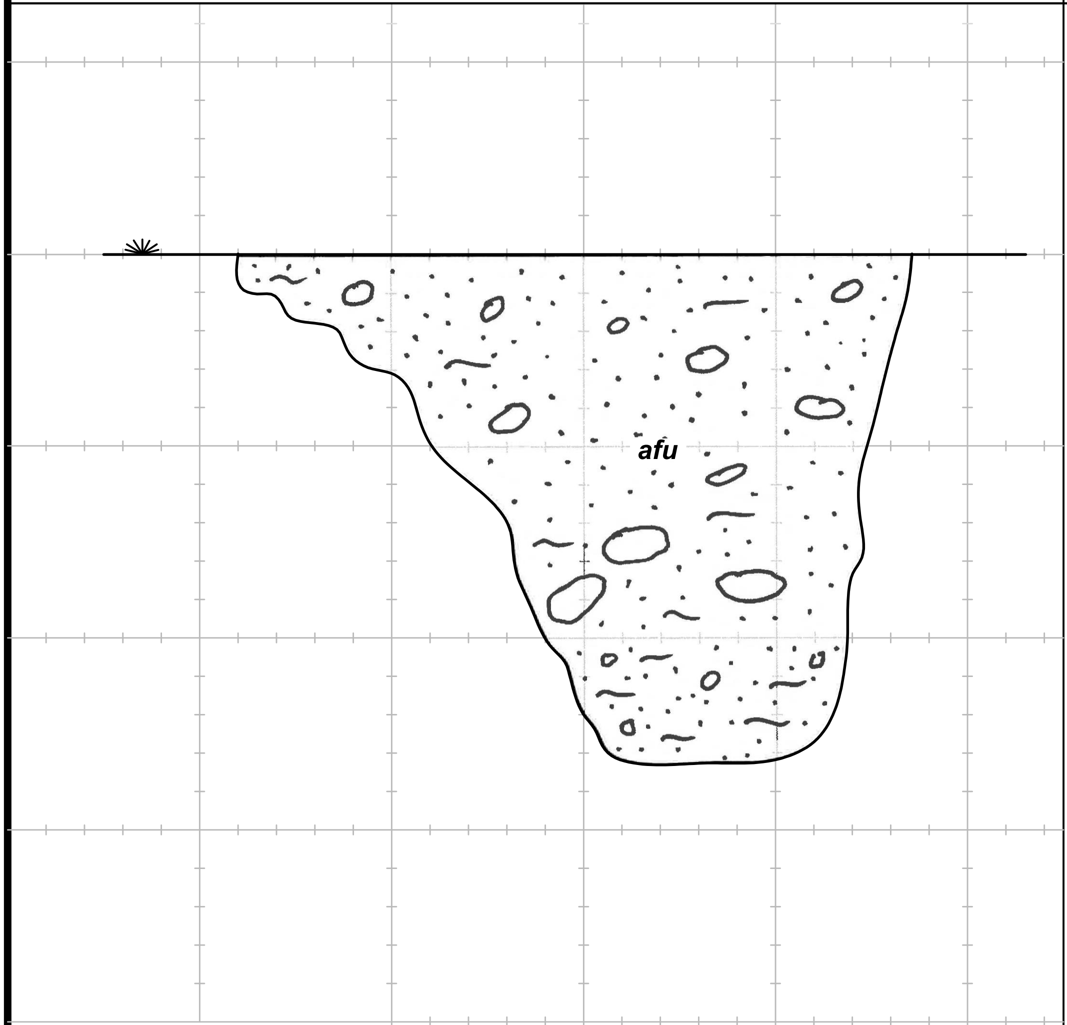
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/19/2020	**TEST PIT NO.:** TP-11	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1351'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N50E	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
<u>UNDOCUMENTED ARTIFICIAL FILL (afu)</u>: *@0' SAND WITH SILT AND COBBLES AND BOULDERS UP TO 1' MAXIMUM DIMENSION; LIGHT BROWN, DRY, LOOSE*	SP-SM	
@2' INCREASED MOISTURE TO SLIGHTLY MOIST, INCREASED DENSITY TO MEDIUM DENSE		B-1 @5'-7'
@7' INCREASED BOULDERS UP TO 2.5' IN MAXIMUM DIMENSION		
@10' SILTY SAND; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, GRAVELS AND COBBLES UP TO 4" DIAMETER	SP	

TOTAL DEPTH: 13'

BACKFILLED: YES

COMPACTED: NO

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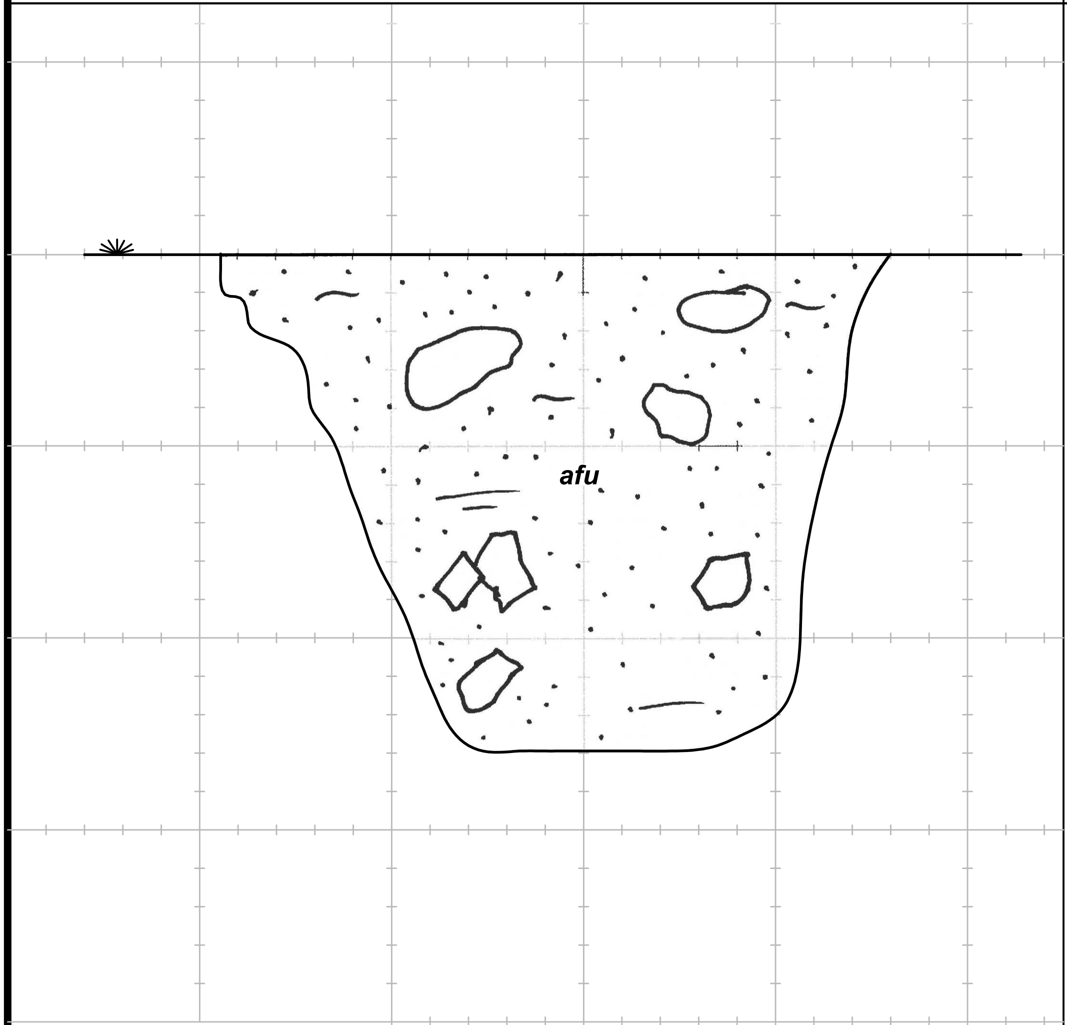
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/19/2020	**TEST PIT NO.:** TP-12	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1335'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N25W	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SAND WITH SILT BOULDERS UP TO 3' MAXIMUM DIMENSION; LIGHT BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE*	SP-SM	
@6' SAND WITH TRACE CLAY AND BOULDERS UP TO 2' MAXIMUM DIMENSION; LIGHT BROWN, SLIGHTLY MOIST, MEDIUM DENSE, ANGULAR BOULDERS	SP	

TOTAL DEPTH: 13'

BACKFILLED: YES

COMPACTED: NO

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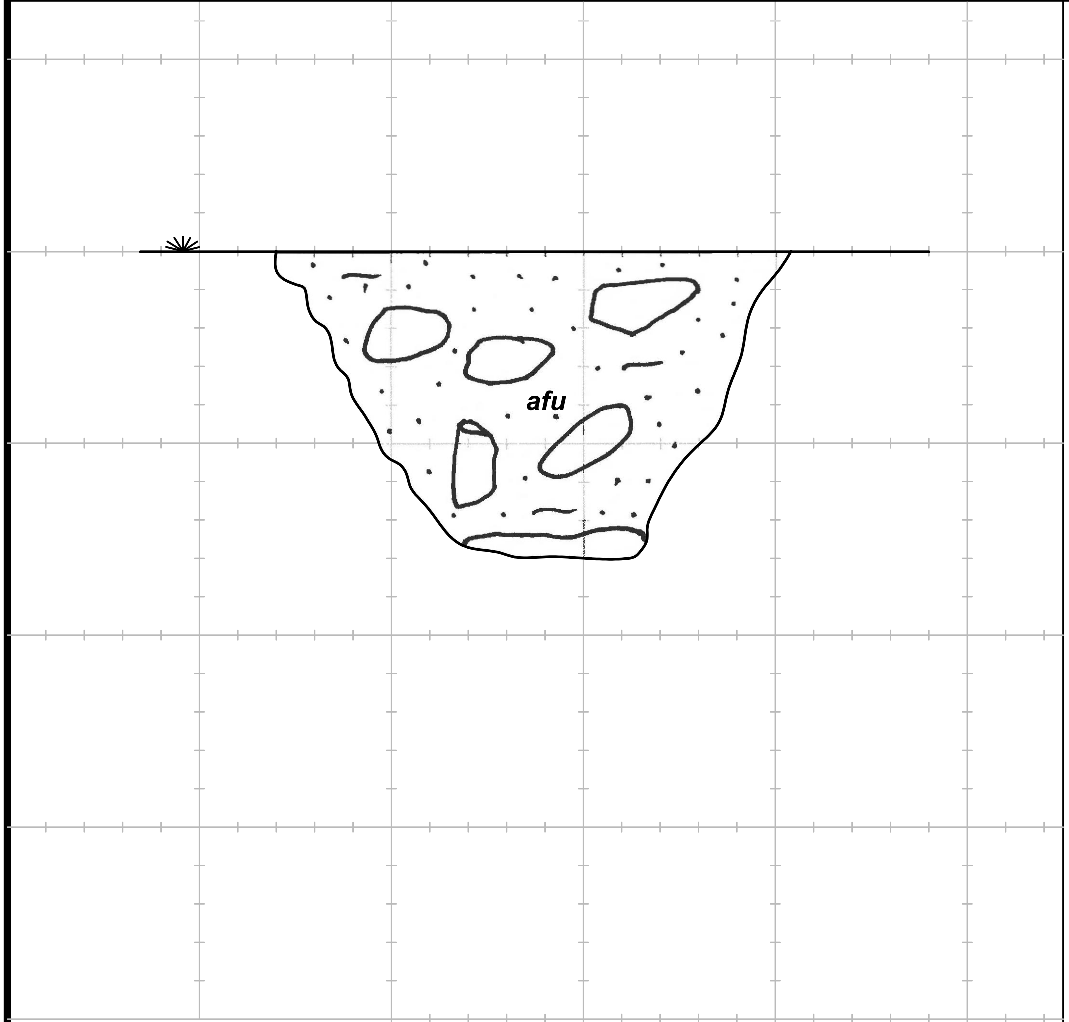
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/19/2020	**TEST PIT NO.:** TP-13	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1313'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N10W	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SAND WITH SILT AND BOULDERS UP TO 3' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY TO SLIGHTLY MOIST BY 4', MEDIUM DENSE* *@8' REFUSAL DUE TO BOULDER*	SP	

TOTAL DEPTH: 8'

BACKFILLED: YES

COMPACTED: NO

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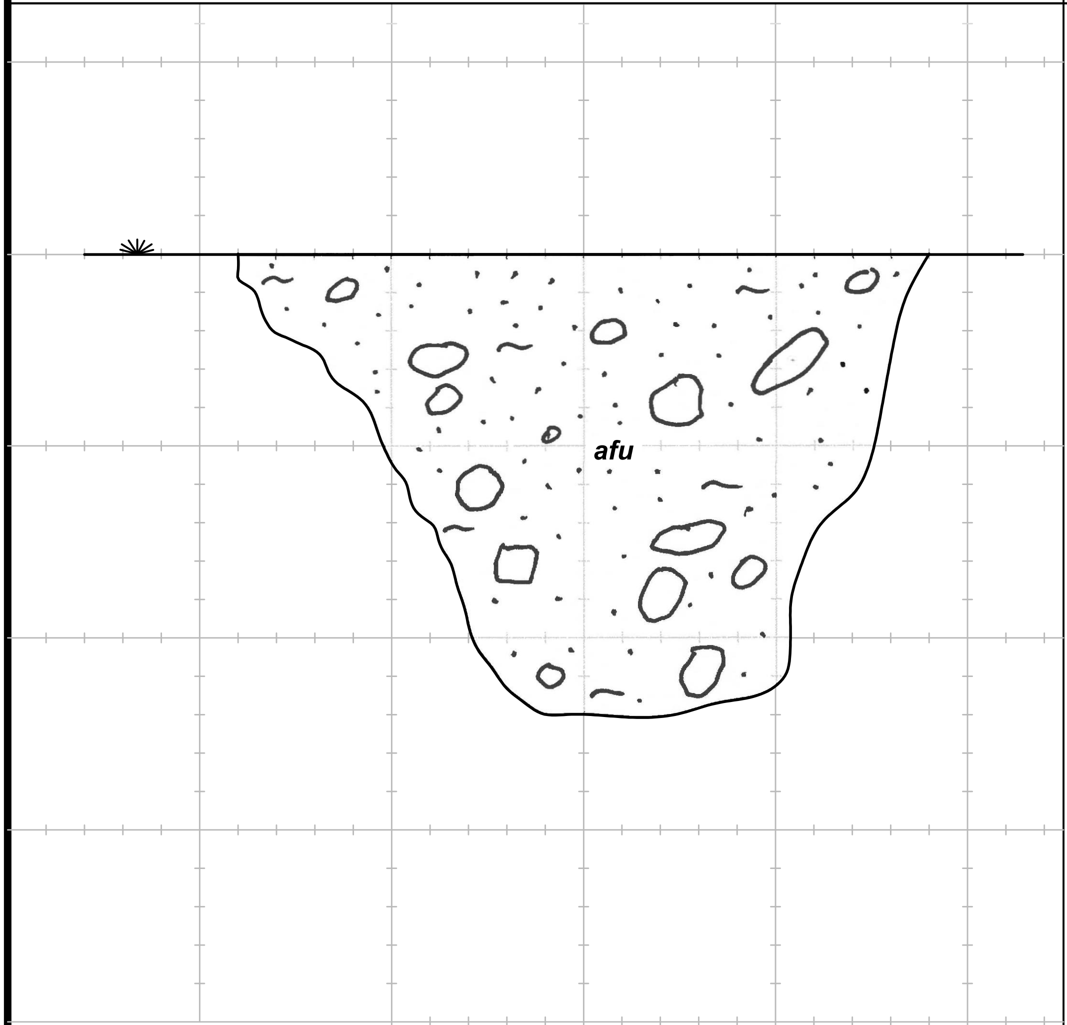
PROJECT NAME: CRESTVIEW APARTMENTS
DATE: 8/19/2020
TEST PIT NO.: TP-14
SURFACE SLOPE: 0°

PROJECT NO.: 3020003
LOGGED BY: CNJ
ELEVATION: 1315'
GROUNDWATER: NONE

EQUIPMENT: CAT 320 EXCAVATOR
REVIEWED BY: CNJ
TREND: N45W
SCALE: 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SAND WITH SILT AND BOULDERS UP TO 2' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, MEDIUM DENSE* *@3' INCREASED MOISTURE TO SLIGHTLY MOIST*	SP-SM	

TOTAL DEPTH: 12'

BACKFILLED: YES

COMPACTED: NO

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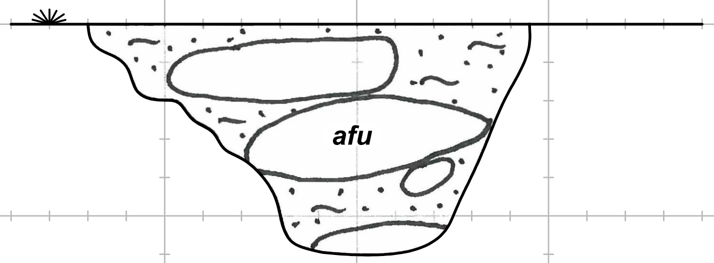
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/19/2020	**TEST PIT NO.:** TP-15	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1332'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N45E	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
<u>**UNDOCUMENTED ARTIFICIAL FILL (afu)**</u>: *@0' SILTY SAND WITH BOULDERS UP TO 6' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE* *@6' REFUSAL DUE TO BOULDER*	SM	

TOTAL DEPTH: 6'

BACKFILLED: YES

COMPACTED: NO

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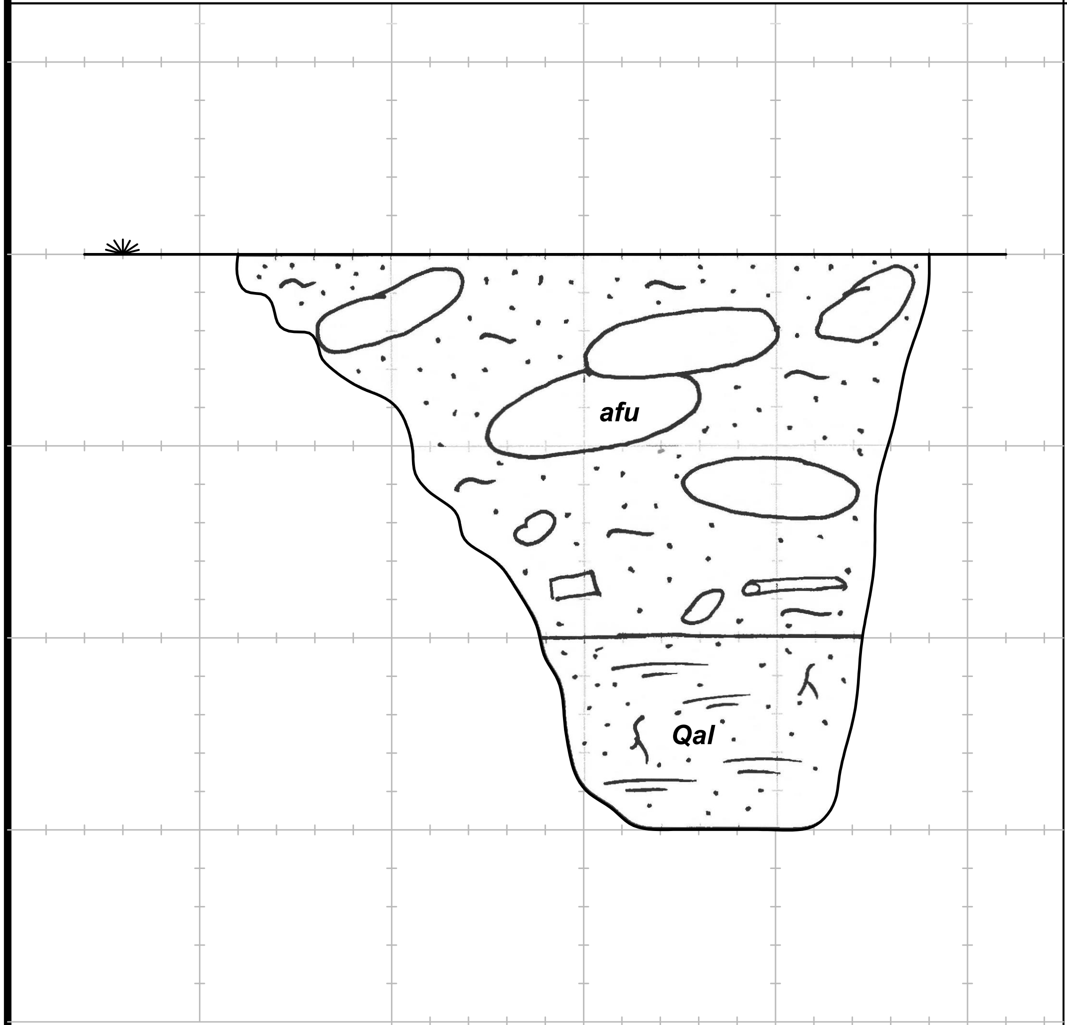
PROJECT NAME: CRESTVIEW APARTMENTS
DATE: 8/19/2020
TEST PIT NO.: TP-16
SURFACE SLOPE: 0°

PROJECT NO.: 3020003
LOGGED BY: CNJ
ELEVATION: 1345'
GROUNDWATER: NONE

EQUIPMENT: CAT 320 EXCAVATOR
REVIEWED BY: CNJ
TREND: N30E
SCALE: 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	USCS	SAMPLE NO.
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH BOULDERS UP TO 5' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, LOOSE* *@2.5' INCREASED MOISTURE TO SLIGHTLY MOIST, INCREASED DENSITY TO MEDIUM DENSE* *@9' BRICK AND PIPE*	SM	
QUATERNARY ALLUVIAL DEPOSITS (Qal): *@10' CLAYEY SAND; REDDISH BROWN, MOIST, MEDIUM DENSE, PINHOLE POROSITY, TRACE ROOTLETS*	SC	

TOTAL DEPTH: 15'

BACKFILLED: YES

COMPACTED: NO

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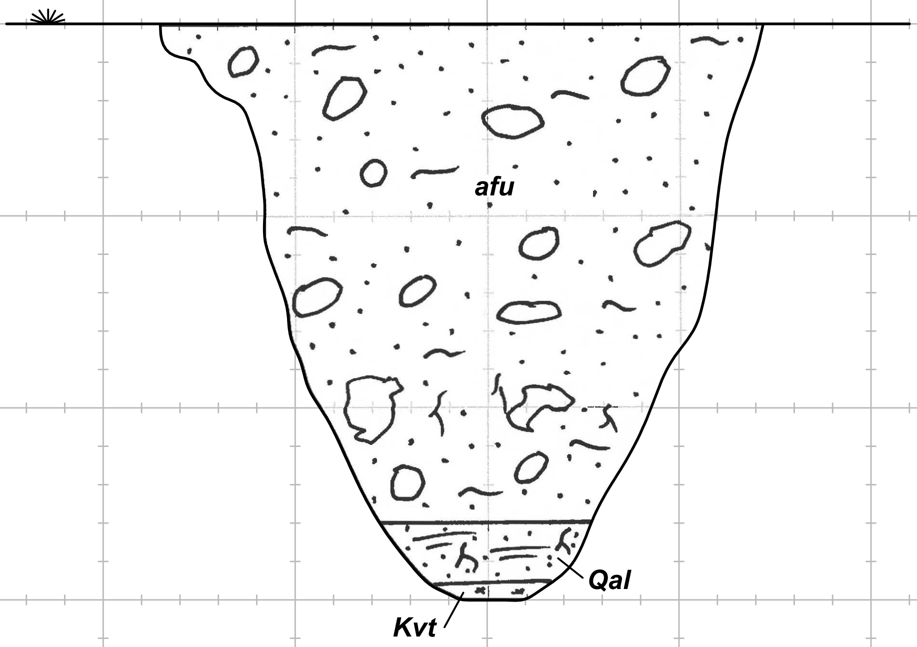
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/19/2020	**TEST PIT NO.:** TP-17	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1351'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N50E	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH BOULDERS UP TO 1.5' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY, LOOSE*	SM	
@3' INCREASED MOISTURE TO SLIGHTLY MOIST, INCREASED DENSITY TO MEDIUM DENSE		
@10' ROOTS AND PLASTIC TRASH		
QUATERNARY ALLUVIAL DEPOSITS (Qal): *@13' CLAYEY SAND; REDDISH BROWN, MOIST, MEDIUM DENSE, ROOTS*	SC	
CRETACEOUS VAL VERDE TONALITE (Kvt): *@15' TONALITE; REFUSAL*	SP	

TOTAL DEPTH: 15'

BACKFILLED: YES

COMPACTED: NO

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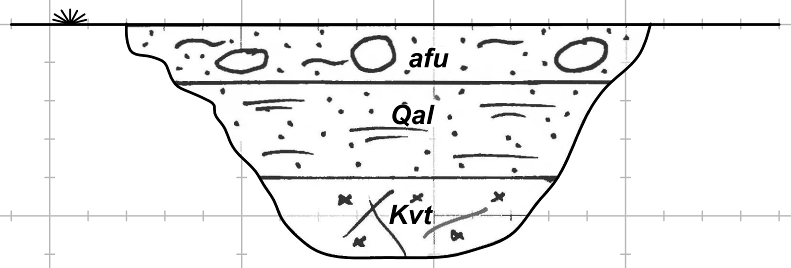
PROJECT NAME: CRESTVIEW APARTMENTS	**DATE:** 8/19/2020	**TEST PIT NO.:** TP-18	**SURFACE SLOPE:** 0°
PROJECT NO.: 3020003	**LOGGED BY:** CNJ	**ELEVATION:** 1348'	**GROUNDWATER:** NONE
EQUIPMENT: CAT 320 EXCAVATOR	**REVIEWED BY:** CNJ	**TREND:** N85W	**SCALE:** 1"=5'

GRAPHICAL REPRESENTATION BELOW:



SOIL DESCRIPTION:	**USCS**	**SAMPLE NO.**
UNDOCUMENTED ARTIFICIAL FILL (afu): *@0' SILTY SAND WITH BOULDERS UP TO 1' IN MAXIMUM DIMENSION; LIGHT BROWN, DRY TO SLIGHTLY MOIST, LOOSE*	SM	
QUATERNARY ALLUVIAL DEPOSITS (Qal): *@1.5' CLAYEY SAND; REDDISH BROWN, MOIST, MEDIUM DENSE*	SC	
CRETACEOUS VAL VERDE TONALITE (Kvt): *@4' TONALITE EXCAVATES TO SAND, SLIGHTLY MOIST, VERY DENSE, VERY WEATHERED*	SP	
@6' REFUSAL		

TOTAL DEPTH: 6'

BACKFILLED: YES

COMPACTED: NO

NOVA

GEOTECHNICAL
MATERIALS
SPECIAL
INSPECTION

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APPENDIX C

LOGS OF EXPLORATORY EXCAVATIONS BY OTHERS

PROJECT NO. T2820-22-01

TEST PIT TP-1

ELEV. (MSL.) **1347** DATE COMPLETED **06/07/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, very loose, dry, strong brown to olive brown			
2								
4								
6								
8					-Caving			
10					-Boulders up to 8' diameter			
12								
14								
16								
18								
20					- Becomes loose; some cobbles			
22	TP-1@22' ◣			SC	**ALLUVIUM (Qal)** Clayey SAND, medium dense, moist, reddish brown; coarse sand; some roots; porosity up to 1/4-inch			
24								
26					**VAL VERDE TONALITE (Kvt)** Completely weathered, weak, moist, reddish brown, GRANITIC BEDROCK; excavates as a silty sand			
28								
30								
					-Becomes highly weathered, moderately weak			

Total Depth 31.5'
Groundwater not encountered
Caving from 7 to 20'
Backfilled with cuttings 6/7/2018

Figure A-1,
Log of Test Pit TP-1, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▩ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-2

ELEV. (MSL.) **1341** DATE COMPLETED **06/07/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, dry, brown; fine to coarse sand; some cobbles up to 1' diameter			
2								
4								
6								
8								
10								
12					-Abundant boulders up to 2' diameter; some wire (blasting cord)			
14								
16					-Boulders up to 4' diameter; nested buoulders with loose fill matrix -Caving to 26'			
18								
20								
22								
24								
26				SM	**ALLUVIUM (Qal)** Silty SAND, loose, moist, strong brown to reddish brown			
28								
30								
32					-Becomes medium dense			
34					Total Depth 34' Groundwater not encountered Caving from 16' to 26' Backfilled with cuttings 06/07/2018			

Figure A-2,
Log of Test Pit TP-2, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-3

ELEV. (MSL.) **1344** DATE COMPLETED **06/07/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, dry, light brown; fine to coarse sand; abundant boulders up to 4' diameter; some debris (pvc pipe, blasting chord)			
2								
4								
6								
8					-Boulder up to 6' diameter			
10					-Nested boulders with loose matrix to 26'			
12								
14								
16								
18								
20								
22								
24								
26				SM	**ALLUVIUM (Qal)** Silty SAND, loose, moist, redish brown to strong brown			

Total Depth 27'
Groundawater not encountered
No Caving
Backfilled with cuttings 06/07/2018

Figure A-3,
Log of Test Pit TP-3, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-4

ELEV. (MSL.) **1339** DATE COMPLETED **06/07/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, dry, light brown; medium to coarse sand; abundant boulders -Wire -Rubber tubing			
2								
4								
6								
8								
10								
12								
14								
16								
18								
20					-Boulders up to 4' diameter; redish brown matrix			
22				SP	Poorly graded SAND, loose, dry to moist, brown; coarse sand; cohesionless			
24								
26				SM	**ALLUVIUM (Qal)** Silty SAND, medium dense, moist, brown; overlying contact dips to the south			
28								
30								

Total Depth 31'
Groundwater not encountered
No Caving
Backfilled with cuttings 06/07/2018

Figure A-4,
Log of Test Pit TP-4, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2820-22-01

TEST PIT TP-5

ELEV. (MSL.) **1315** DATE COMPLETED **06/07/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, loose, dry, grayish brown; cobble and boulders			
2								
4								
6								
8								
10								
12								
14								
16								
18								
20								
22					**VAL VERDE TONALITE Kvt)** Moderately weathered, strong, damp, black/white with reddish yellow staining, GRANITIC BEDROCK; excavates as a gravelly sand; in northern end of trench; falls off to the south			
				SM/SP	**ALLUVIUM (Qal)** Silty SAND to poorly graded sand, loose to medium dense, moist, strong brown; fine to coarse sand			
24								

Total Depth 24'
Groundwater not encountered
No Caving
Backfilled 06/07/2018

Figure A-5,
Log of Test Pit TP-5, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-6

ELEV. (MSL.) **1350** DATE COMPLETED **06/07/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, loose to medium dense, dry to damp, light brown and strong brown; common cobbles			
2								
4								
6					-Debris (pvc pipe)			
8					Debris (metal pipe)			
10								
12								
14								
16								
18								
20								
22								
24					-Boulder, 2' diameter			
26								
28				SM	**ALLUVIUM (Qal)** Silty SAND, loose to medium dense, moist, strong brown; coarse sand			
30								
32								
34					-Some cobble			

Total Depth 34'
Groundwater not encountered
No Caving
Backfilled witj cuttings 06/07/201/

Figure A-6,
Log of Test Pit TP-6, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. T2820-22-01

TEST PIT TP-7

ELEV. (MSL.) **1350** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM/SP	**UNDOCUMTENED FILL (afu)** Silty SAND to poorly graded SAND, damp, grayish brown; fine to coarse sand; some gravel and cobble; micaceous -Becomes reddish brown -Becomes olive brown; increase in cobbles some boulders, up to 2' diameter; slight caving -Trace debris (shot chord)			
2								
4								
6								
8								
10					-Increase in boulders, up to 6' diameter; nested; voids			
12								
14								
16								
18								
20								
22					-Increase in boulders			
24								

Total Depth 25'
Groundwater not encountered
Slight caving from 4' to 22'
Backfilled with cuttings 6/18/2018

Figure A-7,
Log of Test Pit TP-7, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-8

ELEV. (MSL.) **1352** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 17				SM/SP	**UNDOCUMENTED FILL** Silty SAND to poorly graded SAND, dry, reddish brown; fine to coarse sand; some gravel and cobble; micaceous -Becomes damp; olive; increase in cobble -Some boulders up to 3' in diameter -Increase in boulder size, up to 8' in diameter			
17 – 20				SM	**ALLUVIUM** Silty SAND, moist, strong brown; fine to medium sand; some coarse sand; trace gravel; some cobble; micaceous			
20 – 22					**VAL VERDE TONALITE (Kvt)** Highly weathered, weak, moist, brownish yellow, GRANITIC BEDROCK; excavtes as a gravelly sand			

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22

Total Depth 22'
Groundwater not encountered
No caving
Backfilled with cuttings 6/18/2018

Figure A-8,
Log of Test Pit TP-8, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-9

ELEV. (MSL.) **1351** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, dry, reddish brown; fine to coarse sand; some gravell and cobble; micaceous; some debris			
4 6				SM	VAL VERDE TONALITE (Kvt) Soil (grus), weak, moist, brownish red, GRANITIC BEDROCK; fine to coarse sand			
8 10					Highly weathered, moderately weak, moist, gray with reddish yellow staining GRANITIC BEDROCK; excavates as a gravelly sand -Slow advance, moderately weathered; cobble size chunks			
12					Total Depth 12' (Refusal) Groundwater not encountered No caving Backfilled with cuttings 6/18/2018			

Figure A-9,
Log of Test Pit TP-9, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2820-22-01

TEST PIT TP-10

ELEV. (MSL.) **1352** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, loose, dry, light brown; fine to coarse sand; some gravel and cobble; micaceous -Some boulders; some debris; some concrete chunks (possible concrete washout area)			
4 6 8 10 12 14 16 18 20				SM	**ALLUVIUM (Qal)** Silty SAND, medium dense, moist, brown, fine to coarse sand; some mica -Becomes dark brown, some rounded cobbles -Moderately cemented; some porosity up to 1/4-inch; some calcium carbonate			
22					**VAL VERDE TONALITE (Kvt)** Highly weathered, moderately weak, moist, gray with reddish yellow staining, GRANITIC BEDROCK; excavates as a gravelly sand			

Total Depth 23' (Refusal)
Groundwater not encountered
No caving
Backfilled with cuttings 6/18/2018

Figure A-10,
Log of Test Pit TP-10, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. T2820-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP-11** ELEV. (MSL.) **1356** DATE COMPLETED **06/08/2018** EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, loose, dry, grayish brown; fine to coarse sand -Becomes damp; reddish brown			
4 6					**VAL VERDE TONALITE (Kvt)** Moderately weathered, moderately strong, damp, black/white with reddish yellow staining, GRANITIC BEDROCK; excavates as a gravelly sand with cobble			
					Total Depth 7' (Refusal) Groundwater not encountered Slight caving Backfilled with cuttings 6/18/2018			

Figure A-11,
Log of Test Pit TP-11, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-12

ELEV. (MSL.) **1364** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, dry, grayish brown; fine to coarse sand			
2					**VAL VERDE TONALITE (Kvt)** Moderately weathered, moderately strong, damp, black/white, GRANITIC BEDROCK; excavates as a gravelly sand -Some cobble size chunks -Becomes slightly weathered; strong; pegmatitie dike N40W/60S			
4								

Total Depth 5' (Refusal)
Groundwater not encountered
No caving
Backfilled with cuttings 6/18/2018

Figure A-12,
Log of Test Pit TP-12, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-13

ELEV. (MSL.) **1369** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**VAL VERDE TONALITE (Kvt)** Fresh, strong, dry, black/white, GRANITIC BEDROCK; excavates as boulder with cobble (shot rock)			
2								
					-Becomes non-shot rock			

Total Depth 3' (Refusal)
Groundwater not encountered
No caving
Backfilled with cuttings 6/18/2018

Figure A-13,
Log of Test Pit TP-13, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▨ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-14

ELEV. (MSL.) **1366** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**VAL VERDE TONALITE (Kvt)** Fresh, strong, dry, black/white, GRANITIC BEDROCK; excavates as boulder with cobble (shot rock)			
2								
4					-Beomes non-shot rock			
6								
					Total Depth 6.5' (Refusal) Groundwater not encountered No caving Backfilled with cuttings 6/18/2018			

Figure A-14,
Log of Test Pit TP-14, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. T2820-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TEST PIT TP-15 ELEV. (MSL.) **1359** DATE COMPLETED **06/08/2018** EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**VAL VERDE TONALITE (Kvt)** Fresh, strong, dry, black/white, GRANITIC BEDROCK; excavates as boulder with cobble (shot rock) -Becomes non-shot rock			
					Total Depth 1' (Refusal) Groundwater not encountered No caving Backfilled with cuttings 6/18/2018			

Figure A-15,
Log of Test Pit TP-15, Page 1 of 1

T2820-22-01 TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-16

ELEV. (MSL.) **1356** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4				SM/SP	**UNDOCUMENTED FILL (afu)** Silty SAND to poorly graded SAND, loose, dry, light grayish brown; fine to coarse sand; some cobble and boulders -Boulders up to 4' diameter			
6 8 10 12 14 16				SM	**ALLUVIUM (Qal)** Silty SAND, medium dense, moist, brown; fine to coarse sand; micaceous -Some porosity up to 1/4-inch			
18 20					**VAL VERDE TONALITE (Kvt)** Moderately weathered, moderately strong, moist, reddish yellow, GRANITIC BEDROCK; excavates as a gravelly sand			
22					Total Depth 22' (Refusal) Groundwater not encountered No caving Backfilled with cuttings 6/18/2018			

Figure A-16,
Log of Test Pit TP-16, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

GEOCON

PROJECT NO. T2820-22-01

TEST PIT TP-17

ELEV. (MSL.) **1358** DATE COMPLETED **06/08/2018**

EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, dry, grayish brown; fine to coarse sand; some cobbles			
2					**VAL VERDE TONALITE (Kvt)** fresh, strong, damp, black/white, GRANITIC BEDROCK; shot rock			
4					-Becomes non-shot rock			

Total Depth 4.5' (Refusal)
Groundwater not encountered
No caving
Backfilled with cuttings 6/18/2018

Figure A-17,
Log of Test Pit TP-17, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2820-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP-18** ELEV. (MSL.) **1356** DATE COMPLETED **06/08/2018** EQUIPMENT **EXCAVATOR W/ 36" BUCKET** BY: **P. THERIAULT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**UNDOCUMENTED FILL (afu)** Silty SAND, loose, dry, grayish brown; fine to coarse sand; some cobbles			
2					**VAL VERDE TONALITE (Kvt)** Fresh, strong, damp, black/white, GRANITIC BEDROCK; shot rock -Becomes non-shot rock			
					Total Depth 2.5' (Refusal) Groundwater not encountered No caving Backfilled with cuttings 6/18/2018			

Figure A-18,
Log of Test Pit TP-18, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-1			ARTIFICIAL FILL (Qaf):
	0.0-1.0	SM	Silty SAND: Fine- to coarse-grained, brown to reddish-brown, dry, loose, desiccated.
			ALLUVIUM (Qal):
	1.0-14.0	SM	Silty SAND: Fine- to coarse-grained, brown to reddish-brown, dry to damp, loose to medium dense, porous, desiccated to 3 feet.
			BEDROCK (Kgr):
	14.0-16.0		Granitic Rock: Gray-brown, dry to damp, moderately hard, very coarse-grained, moderately weathered.
			Total Depth: 16.0 feet No Caving No Groundwater Backfill not compacted
T-2			ARTIFICIAL FILL (Qaf):
	0.0-13.0	SM	Silty SAND: Fine- to coarse-grained with gravel to boulder-size rocks to 5' in diameter, brown, damp, loose; large pocket of debris (tire fragments, wood, organic material, concrete, asphalt), concrete to 5' in diameter.
			ALLUVIUM (Qal):
	13.0-19.0	SM	Silty SAND: Fine- to coarse-grained, brown, damp to moist, loose, porous.
			BEDROCK (Kgr):
	19.0-21.0		Granitic Rock: Gray-brown, dry to damp, moderately hard, very coarse-grained.
			Total Depth: 21.0 feet Slight Caving of Fill No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-3			ARTIFICIAL FILL (Qaf):
	0.0-20.0	SM	Silty SAND: Fine- to coarse-grained with gravel to boulder-size rocks to 3' in diameter, brown, dry, loose, minor debris.
			@ 5.0-20.0: portion of trench exposes slurry or concrete truck wash out materials.
			@ 12 feet: roll of chain-link fence.
			ALLUVIUM (Qal):
	20.0-24.0	SM	Silty SAND: Fine- to coarse-grained, brown, damp to moist, loose, porous.
			BEDROCK (Kgr):
	24.0-26.0		Granitic Rock: Gray-brown, dry to damp, moderately hard, very coarse-grained.
			Total Depth: 26.0 feet Moderate Caving of fill No Groundwater Backfill not compacted
T-4			BEDROCK (Kgr):
	0.0-4.0		Granitic Rock: Gray-brown, dry, moderately hard to hard, very coarse-grained.
			Total Depth: 4.0 feet No Caving No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-5			ARTIFICIAL FILL (Qaf):
	0.0-33.0	SM	Silty SAND: Fine- to coarse-grained with gravel to boulder-size rocks up to 11' in diameter, brown, damp, loose; extensive debris including wood, tires, metal pipe, etc.
			Total Depth: 33.0 feet Slight Caving No Groundwater Backfill not compacted
T-6	0.0-10.0		BEDROCK (Kgr): Granitic Rock: Mottled black, white and gray, dry to damp, hard to very hard, very coarse-grained.
			@ 3.0 feet: Becomes locally very hard.
			@ 10 feet: Refusal.
			Total Depth: 10.0 feet No Caving No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-7			ARTIFICIAL FILL (Qaf):
	0.0-21.0	SM	Silty SAND: Fine- to coarse-grained, reddish-brown, dry, medium dense. @ 5.0 feet: Becomes brown, damp, gravel to boulder-size rocks. @ 6.0 feet: Concrete and brick debris. @ 12 feet: Increase in boulders up to 5' in diameter, concrete debris continues. @ 15 feet: Misc. debris (plastic bottles, pipe, etc.).
			ALLUVIUM (Qal):
	21.0-28.0	SM	Silty SAND: Fine- to coarse-grained, reddish-brown, moist, medium dense.
			Total Depth: 28.0 feet Slight Caving of Fill No Groundwater Backfill not compacted
T-8			ARTIFICIAL FILL (Qaf):
	0.0-15.0	SM	Silty SAND: Fine- to coarse-grained with gravel and boulder-size rocks up to 2' in diameter, brown, dry, loose; minor debris.
			TOPSOIL (No Map Symbol):
	15.0-18.0	SM	Silty SAND: Fine- to coarse-grained, brown, damp to moist, loose, porous.
			BEDROCK (Kgr):
	18.0-19.0		Granitic Rock: Gray-brown, dry to damp, hard, very coarse-grained.
			Total Depth: 19.0 feet Slight Caving of fill No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

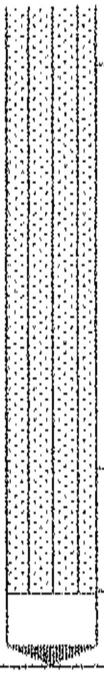
Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-9			ARTIFICIAL FILL (Qaf):
	0.0-18.0	SM	Silty SAND: Fine- to coarse-grained with gravel to boulder-size rocks, brown, damp, loose. @ 2.0 feet: Numerous boulders up to 5' in diameter. @ 10.0 feet: Slight decrease in boulder concentration.
			Total Depth: 18.0 feet Slight Caving No Groundwater Backfill not compacted
T-10			ARTIFICIAL FILL (Qaf):
	0.0-16.0	SM	Silty SAND: Fine- to coarse-grained with gravel to boulder-size rocks, brown, damp, loose, minor debris. @ 2 feet: 5' diameter boulder, caving around boulder. @ 5 feet: Scattered 3' diameter boulders, moderate caving, very loose down to 10 feet.
			TOPSOIL (No Map Symbol):
	16.0-20.0	SM	Silty SAND: Fine- to coarse-grained, reddish-brown, moist, loose to medium dense, slightly porous.
			BEDROCK (Kgr):
	20.0-21.0		Granitic Rock: Gray-brown, dry to damp, hard, very coarse-grained.
			Total Depth: 21.0 feet Slight to Moderate Caving of fill No Groundwater Backfill not compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-11			ARTIFICIAL FILL (Qaf):
	0.0-20.0	SM	Silty SAND: Fine- to coarse-grained with gravel to boulder-size rocks, brown, damp, loose. @ 3.0 feet: Numerous boulders from 18"-24" in diameter. @ 5.0 feet: Becomes medium dense to dense. @ 13.0 feet: 3' to 5' diameter boulders, loose. Total Depth: 20.0 feet Slight Caving No Groundwater Backfill not compacted

PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

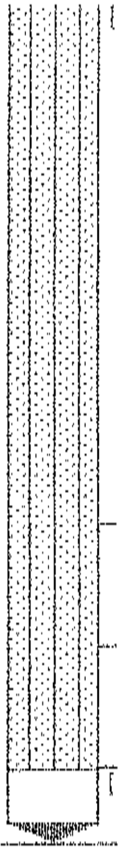
FILE NUMBER: S-8913
TEST PIT NUMBER: 1
REPORT NO.: 5065



Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	[SM] Gray-brown silty fine to coarse sand, dry and moderately dense. FILL
	---	113	4.2	87	2	Light brown silty fine to coarse sand with rock fragments, damp and medium dense. FILL
					3	
	---	110	5.6	84	4	
					5	- becoming loose at 5.0 feet
	---	108	6.5	83	6	
					7	
	---	105	6.3	81	8	Light brown silty fine to medium sand, porous, moist and loose. ORIGINAL GROUND
					9	
					10	[ ] Light gray weathered granitic bedrock, damp and dense Total Depth at 10.5 Feet - No Free Groundwater Encountered

PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

FILE NUMBER: S-8913
TEST PIT NUMBER: 2
REPORT NO.: 5065



Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	[SM] Gray-brown silty fine to coarse sand with rock fragments, damp and moderately dense. FILL
	---	117	4.8	90	2	
					3	- becoming loose at 3.0 feet
	---	108	7.1	83	4	
					5	
	---	104	7.6	80	6	
	---	104	6.4	80	7	
					8	
					9	Light brown silty fine to medium sand with organic debris, moist and loose. FILL
					10	
					11	Light brown silty fine to medium sand, porous, moist and loose. ORIGINAL GROUND
					12	
					13	[ ] Light gray-brown weathered granitic bedrock, damp and dense Total Depth at 13.5 Feet - No Free Groundwater Encountered

PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

FILE NUMBER: S-8913
TEST PIT NUMBER: 3
REPORT NO.: 5065

Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	[SM] Mottled gray and light brown silty fine to coarse sand with boulders up to 30 inches in diameter, and fragments of wire, brick, asphalt concrete, pipe, and concrete, damp and dense. FILL
	---	124	5.0	92	2	
					3	
					4	
					5	- less debris and very dense below 5.0 feet
					6	
	---	129	7.6	96	7	- large concrete fragments with voids below 6.5 feet
					8	
					9	- large concrete slab and footing at 8.5 feet
					10	
					11	
					12	
					13	
					14	
					15	Light red-brown silty fine to coarse sand, very porous, moist and loose. ORIGINAL GROUND
					16	[ ] Light gray weathered granitic bedrock, damp & medium dense Total Depth at 16.0 Feet - No Free Groundwater Encountered

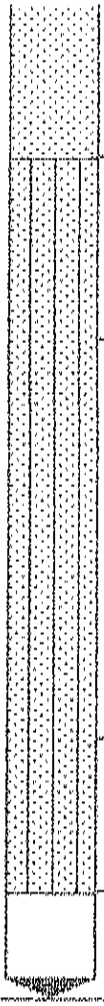
PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

FILE NUMBER: S-8913
TEST PIT NUMBER: 4
REPORT NO.: 5065

Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	[SM] Light brown silty fine to coarse sand with rock fragments, damp and dense. FILL
	---	127	4.4	95	2	
					3	
	---	131	6.6	98	4	Gray-brown silty fine to coarse sand with rock fragments and tire debris, damp and very dense. FILL
					5	
	---	132	6.0	99	6	- becoming medium dense at 6.0 feet
					7	
					8	- boulders up to 4 inches in diameter at 8.0 feet
					9	
					10	
					11	
					12	
					13	Light brown silty fine to coarse sand, very porous, moist and loose. ORIGINAL GROUND
					14	
					15	
					16	[ ] Light red-yellow weathered granitic bedrock, damp and dense Total Depth at 16.5 Feet No Free Groundwater Encountered

PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

FILE NUMBER: S-8913
TEST PIT NUMBER: 5
REPORT NO.: 5065



Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	(SP) Light gray-white gravelly fine to coarse sand with concrete debris up to 4.0 feet in diameter, dry and loose. FILL
					2	
					3	(SM) Gray-brown silty fine to coarse sand with gravel and concrete debris, damp and loose. FILL
	---	86	6.4	71	4	
					5	- becoming moist at 5.5 feet
	---	113	11.0	84	6	Dark brown silty fine to medium sand with a trace of clay, porous, moist and loose. ORIGINAL GROUND
					7	
	---	110	9.0	82	8	
					9	
					10	- very porous at 10.0 feet
					11	
					12	
					13	Light brown-gray silty fine to coarse sand with a trace of caliche, porous, slightly cemented, damp and medium dense
					14	
					15	( ) Gray-brown weathered granitic bedrock, moist and very dense Refusal on very dense bedrock at 16.0 feet No Free Groundwater Encountered
					16	

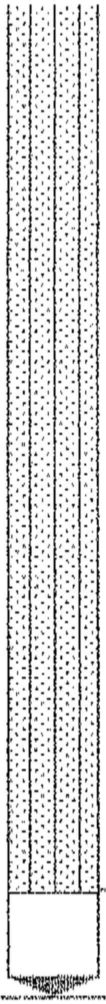
PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

FILE NUMBER: S-8913
TEST PIT NUMBER: 6
REPORT NO.: 5065

Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	(SM) Gray mottled-light brown silty fine to coarse sand with voids and gravel, cobbles, and boulders (up to 5.0 feet in diameter), dry and loose. FILL
					2	
					3	
					4	
					5	
					6	
					7	
					8	Refusal on large nested boulders at 8.0 feet No Free Groundwater Encountered

PROJECT: Canyon Crest Courtyard
PROJECT LOCATION: Riverside, CA
ENCLOSURE NO.: 2
DATE OF TESTS: 8/30/97

FILE NUMBER: S-8813
TEST PIT NUMBER: 7
REPORT NO.: 5065



Blows Per Foot	Driving Energy k-ft/ft	Dry Density PCF	Moist. Content %	Rel. Comp. %	Feet	Visual Classification
					1	[SM] Light gray silty fine to coarse sand with rock fragments, dry and moderately dense. FILL
	---	120	3.8	98	2	
					3	
	---	119	4.7	98	4	
					5	
					6	
					7	
					8	[ ] Light gray weathered granitic bedrock, damp and dense. ORIGINAL GROUND Total Depth at 8.0 Feet - No Free Groundwater Encountered

APPENDIX D

SEISMIC REFRACTION SURVEY REPORT

TERRA GEOSCIENCES

SEISMIC REFRACTION SURVEY

CRESTVIEW APARTMENT PROJECT

NWC OF CENTRAL AVE. AND SYCAMORE CANYON BLVD.

CITY OF RIVERSIDE, CALIFORNIA

Project No. 203480-1

August 24, 2020

Prepared for:

NOVA Services, Inc.
944 Calle Amanecer, Suite F
San Clemente, CA 92672

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

NOVA Services, Inc.
944 Calle Amanecer, Suite F
San Clemente, CA 92673

August 24, 2020
Project No. 203480-1

Attention: Ms. Chelsea Jaeger, Project Geologist

Regarding: Seismic Refraction Survey
Crestview Apartment Project
NWC of Central Ave. & Sycamore Canyon Blvd.
City of Riverside, California
NOVA Project No. 3020003

EXECUTIVE SUMMARY

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

For this study, six survey traverses were performed across the subject property, as directed by your office. The traverses were located in the field by use of Google™ Earth imagery (2020) and GPS coordinates. The approximate locations of these traverses are shown on the Seismic Line Location Map, Plate 1, of which the base map is a captured Google™ Earth image (2020).

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

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

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

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INTRODUCTION

The subject property is located at the northwest corner of Central Avenue and Sycamore Canyon Boulevard, in the City of Riverside, California. The site is currently vacant, generally covered by annual weeds and grasses, with scattered brush. We understand that the site has been previously graded and is currently generally flat-lying, sloping gently towards the south. Westerly-facing slopes up to 25± feet high are present along the western portion of the site, with a 40± foot high hill in the northwest.

Locally, as shown on Figure 1 below, surficial geologic mapping by Morton and Cox (2001), indicate the subject property to underlain by Cretaceous age granitic rocks, which consists predominantly of gray-weathering, relatively homogeneous, massive to well-foliated, medium- to coarse-grained, biotite-hornblende tonalite (map symbol Kvt), locally referred to as the Val Verde Tonalite. Bedrock exposures were observed locally at the site, which appear to be characteristic of tonalitic bedrock. These granitic rocks were formed during the emplacement of the Cretaceous Age Peninsular Ranges Batholith and are included within the northernmost portion of the Val Verde Pluton. The structure of the tonalitic bedrock within this portion of the pluton is dominated by the foliation, which generally strikes in a northwest-southeast direction, being parallel to the regional structural grain of the batholith.

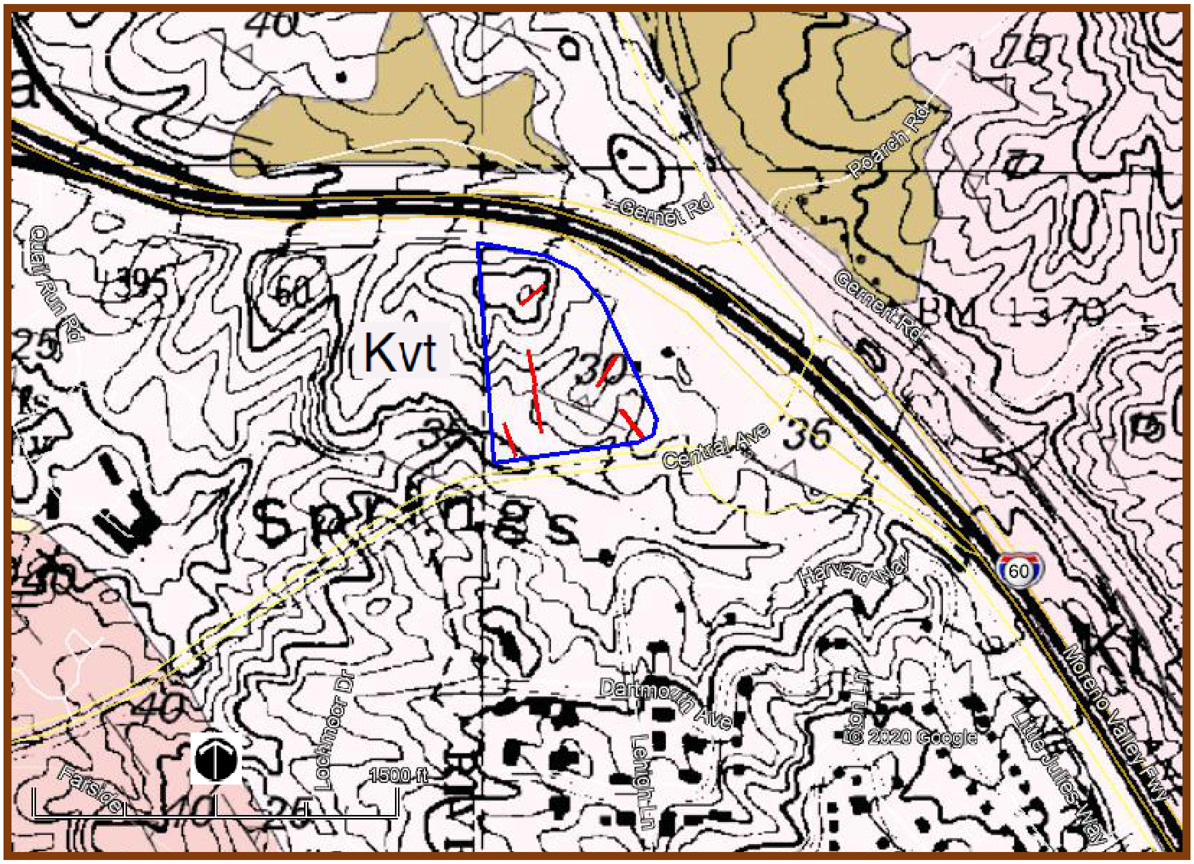


FIGURE 1- Geologic Map (Morton and Cox, 2001), site outlined in blue, seismic lines in red.

SEISMIC REFRACTION SURVEY

Methodology

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

Field Procedures

Six seismic refraction survey lines (Seismic Lines S-1 through S-6) have been performed along representative areas across the site as selected by you. The traverses were located in the field by use of Google™ Earth imagery (2020) and GPS coordinates and have been delineated on the Seismic Line Location Map, as presented on Plate 1. The survey traverses ranged from 125 to 200 feet in length, which consisted of a total of twenty-four 14-Hertz geophones, spaced at regular five- to eight-foot intervals, in order to detect both the direct and refracted waves. A 16-pound sledge-hammer was used as the energy source to produce the seismic waves at seven locations along each survey traverse. Multiple hammer impacts were utilized at each shot point in order to increase the signal to noise ratio, which enhanced the primary seismic “P”-waves.

The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. Seven shot points were utilized along each spread using forward, reverse, and several intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.064 to 0.120 seconds. No acquisition filters were used during data collection.

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

Data Processing

The recorded seismic data was subsequently transferred to our office computer for processing and analyzing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004); **Refractor** (Geogiga, 2001-2019); and **Rayfract™** (Intelligent Resources, Inc., 1996-2020). All of the computer programs perform their individual analyses using exactly the same input data, which includes the first-arrival times of the “P”-waves and the survey line geometry.

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

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

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

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

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

SUMMARY OF GEOPHYSICAL INTERPRETATION

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

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

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

- ❑ ***Velocity Layer V1***: This uppermost velocity layer (V1) is most likely comprised of artificial fill, colluvium, topsoil, alluvium, and/or completely-weathered and fractured bedrock materials. This layer has an average weighted velocity of 1,948 to 2,370 fps, which is typical for these types of unconsolidated surficial earth materials.

- ❑ ***Velocity Layer V2***: The second layer (V2) yielded a seismic velocity range of 3,340 to 3,504 fps, which is typical for highly-weathered granitic bedrock materials. This velocity range may indicate the presence of homogeneous weathered bedrock with a relatively wide spaced joint/fracture system and/or the possibility of buried relatively-fresher boulders within a very highly-weathered bedrock matrix. Additionally, the presence of older alluvial sediments, such as mapped by Morton and Cox (2001) in the local area, may also be locally present in this velocity layer based upon the degree of sediment induration.

- ❑ ***Velocity Layer V3***: The third layer (V3) indicates the presence of moderately to slightly-weathered bedrock, having a seismic velocity range of 5,789 to 8,246 fps. These higher velocities signify the decreasing effect of weathering as a function of depth and could indicate a slightly-weathered bedrock matrix that has a wide-spaced fracture system, or possibly the presence of abundant widely-scattered buried fresh large crystalline boulders in a moderately-weathered matrix, which based on the abundant large surface rock outcrops exposed along the northwest, appears likely.

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

TABLE 1- VELOCITY SUMMARY OF SEISMIC SURVEY LINES

Seismic Line	V1 Layer (fps)	V2 Layer (fps)	V3 Layer (fps)
S-1	**2,095**	**-------**	**8,227**
S-2	**-------**	**3,340**	**5,789**
S-3	**2,097**	**3,504**	**-------**
S-4	**2,370**	**-------**	**8,246**
S-5	**1,948**	**-------**	**7,045**
S-6	**2,024**	**-------**	**7,048**

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

In general, the seismic velocity of the bedrock gradually increases with depth, with occasional lateral velocity differentials suggesting the local presence of buried corestones and/or dike structures. These corestones are expected as numerous bedrock outcrops are scattered across the subject property. The colors representing the velocity gradients have been standardized on all of the models for comparative purposes.

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

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

TABLE 2- CATERPILLAR RIPPABILITY CHART (D9 Ripper)

Granitic Rock Velocity	Rippability
< 6,800	Rippable
6,800 – 8,000	Moderately Rippable
> 8,000	Non-Rippable

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

TABLE 3- STANDARD CALTRANS RIPPABILITY CHART

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

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

TABLE 4- SUMMARY OF ROCK ENGINEERING PROPERTIES

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

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

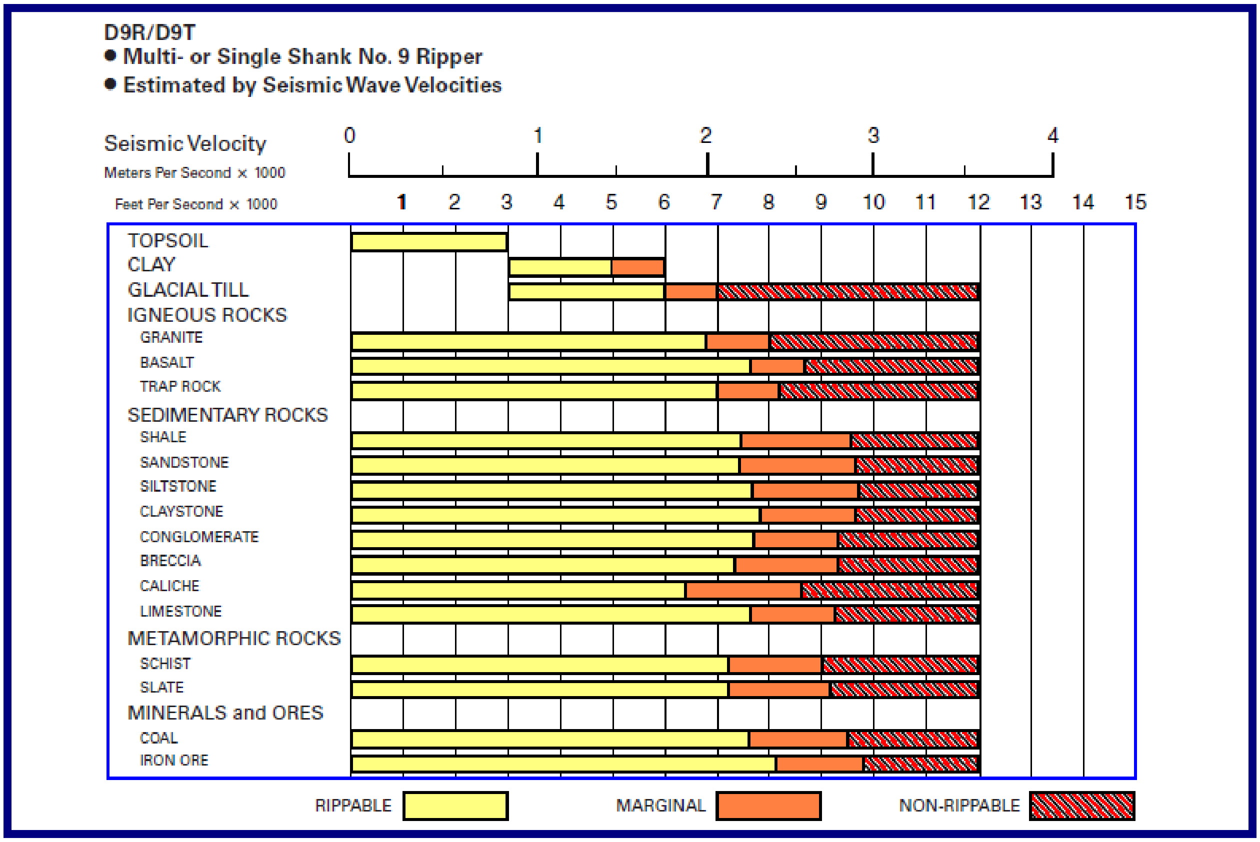


FIGURE 2- Caterpillar D9R Ripper Performance Chart (2018).

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

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

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

These average seismic velocity ranges are summarized below:

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

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

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

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

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

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

GEOLOGIC & EARTHWORK CONSIDERATIONS

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

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

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of fair to good quality with moderate amounts of ambient “noise” that was introduced during our survey, originating from vehicular traffic along the nearby 215 Freeway and adjacent roadways. Analysis of the data and picking of the primary “P”-wave arrivals was therefore performed with minor difficulty, with interpolation of some data points being necessary. Based on the results of our comparative seismic analyses of the computer programs **SIPwin**, **Refractor**, and **Rayfract™**, the seismic refraction survey line models appear to generally coincide with one another, with some minor variances due to the methods that these programs process, integrate, and display the input data. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- ***Velocity Layer V1***:

  No excavating difficulties are expected to be encountered within the uppermost, low-velocity layer V1 (average weighted velocity of 1,948 to 2,370 fps) and should excavate with conventional ripping. This layer is expected to be comprised of artificial fill, topsoil, colluvium, alluvium, and/or completely-weathered and fractured bedrock. Localized boulders could be anticipated based on exposures and/or have been placed in fill, which may require more significant excavation techniques.

Velocity Layer V2:

The second layer V2 (average weighted velocity of 3,340 to 3,504 fps) is believed to consist of highly-weathered granitic bedrock and/or possibly indurated and cemented older alluvial sediments. Using the rock classifications as presented within Tables 1 through 3, seismic wave velocities of less than 6,800± fps are generally noted to be within the threshold for conventional ripping. Isolated floaters (i.e., boulders, corestones, etc.) should be expected to be present within this layer and could produce somewhat difficult conditions locally. A wide range of moderate to very difficult ripping conditions should be anticipated. Placement of infrastructure within this velocity layer may require some breaking and/or light blasting to obtain desired grade.

- **Velocity Layer V3**:

  The third V3 layer is believed to consist of moderately- to slightly-weathered granitic bedrock. Hard excavation difficulties within this velocity layer (average weighted velocity range of 5,789 to 8,246 fps) should be anticipated if encountered during grading. This layer may consist of relatively homogeneous bedrock with wide-spaced fracturing, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. Significant blasting should be anticipated throughout this layer to achieve desired grade, including any infrastructure. Caterpillar (2018; see Figure 2) indicates this velocity range to be “moderately-rippable” to “non-rippable” using a D9R dozer or equivalent. Larger equipment may facilitate excavation potentials within this higher velocity layer.

It should be noted that Seismic Lines S-1 and S-4 through S-6 do not indicate a second velocity layer (V2), which may be due to the previous grading that had removed the highly-weathered bedrock before the overlying fill was placed. Locally along Seismic Line S-2, there does not appear to be an overlying low-velocity layer (V1), most likely due to the previous grading that has stripped this layer to the weathered bedrock surface. Locally along Seismic Line S-3, the harder bedrock velocity layer (V3) was not encountered to a depth of at least 40± feet.

The ray sampling coverage of the subsurface seismic waves that were modeled during the processing of the tomographic models appeared to be of good quality which was verified by having a Root Mean Square Error (RMS) of 3.2 to 5.9 percent (see lower right-hand corner of each model). The RMS error (misfit between picked and modeled first break times) is normalized, which calculates the average picked time over of all traces modeled. This error is automatically calculated during the processing routine, with a value of less than 5.0% being preferred, of which all of the models obtained with the exception of Seismic Lines S-3 and S-66 (RMS of 5.1 and 5.9, respectively). These RMS values are very close to the preferred value and are still considered to be good models. The ambient background noise recorded during our survey and the subsequent necessary interpolations most likely resulted in these higher values.

Based on the tomographic models and typical excavation characteristics observed within granitic bedrock of the southern California region, anticipation of gradual increasing hardness with depth should be anticipated during grading. Significant lateral velocity variations will most likely be encountered across the predominance of the site generally due to the presence of buried corestones and/or dikes such as imaged in some of the tomographic refraction modes and as also expressed as scattered outcrops across the subject site.

CLOSURE

The field geophysical survey was performed on August 19, 2020 by the undersigned using "state of the art" geophysical equipment and techniques along the selected traverse location. The seismic data was further evaluated using recently developed computerized tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface velocity and structural conditions. It should be noted that our data presented within this report was obtained along six specific locations therefore other areas in the local may contain different velocity layers and depths not encountered during our field survey. Additional survey traverses may be necessary to further evaluate the excavation characteristics across other portions of the site where cut grading will be proposed, if warranted. Estimates of layer velocity boundaries as presented in this report are generally considered to be within 10± percent of the total depth of the contact.

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

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

SEISMIC LINE LOCATION MAP

Base Map: Google™ Earth imagery (2020); Seismic traverses shown as yellow lines.

PROJECT NO. 203480-1

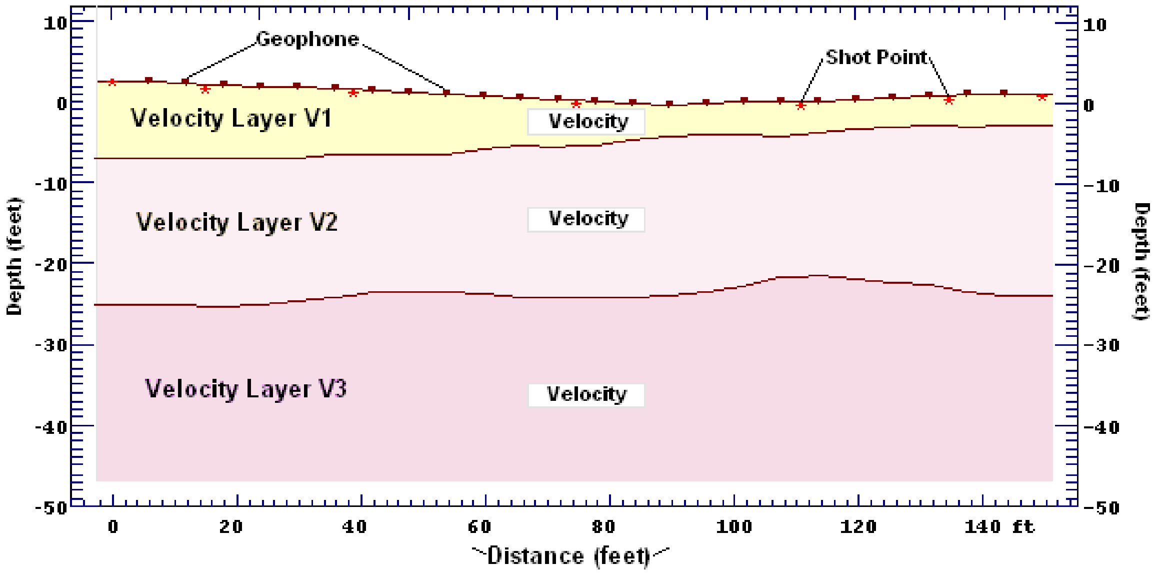
PLATE 1

APPENDIX A

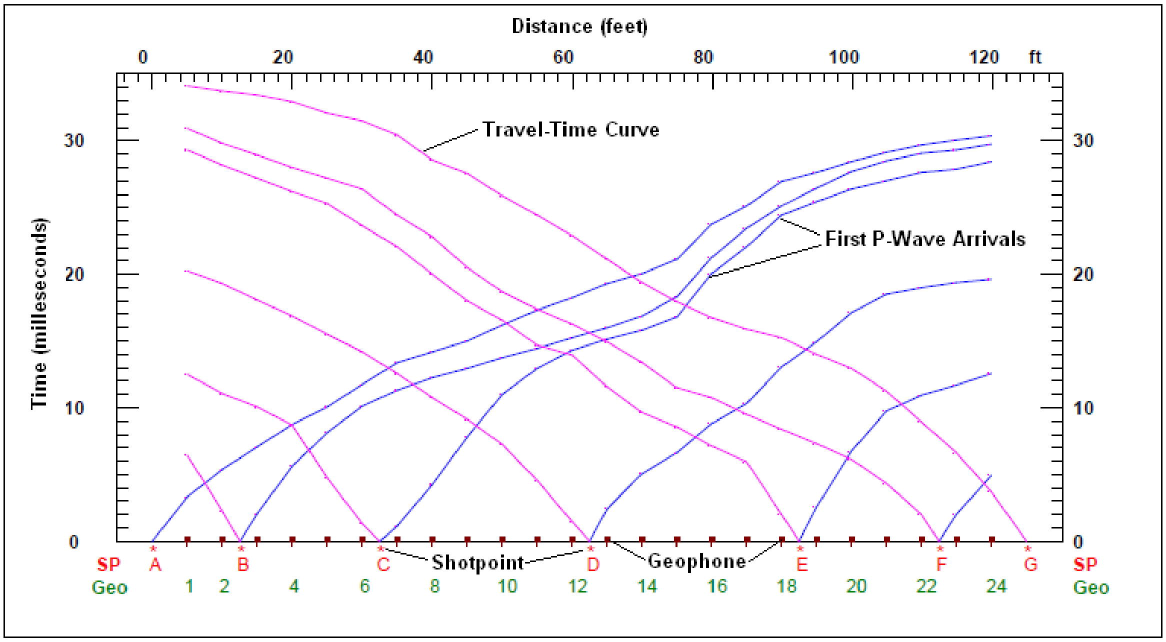
LAYER VELOCITY MODELS

LAYER VELOCITY MODEL LEGEND

LAYER VELOCITY MODEL



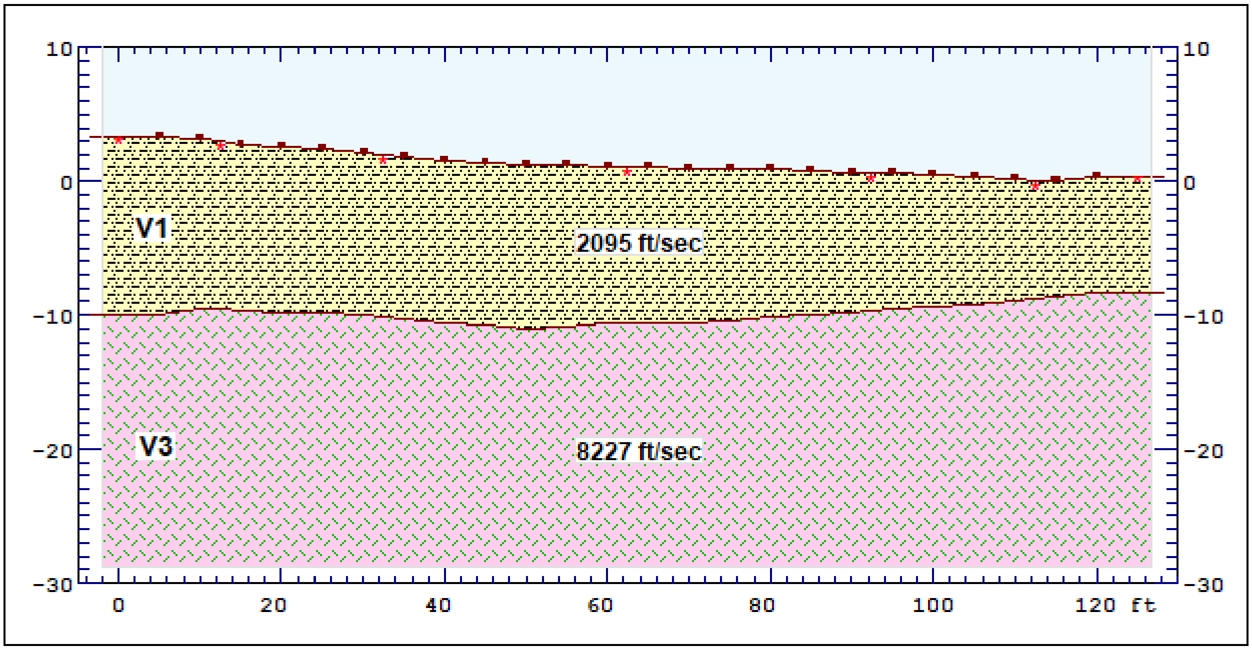
TIME-DISTANCE PLOT



SEISMIC LINE S-1

North 51° East >

LAYER VELOCITY MODEL



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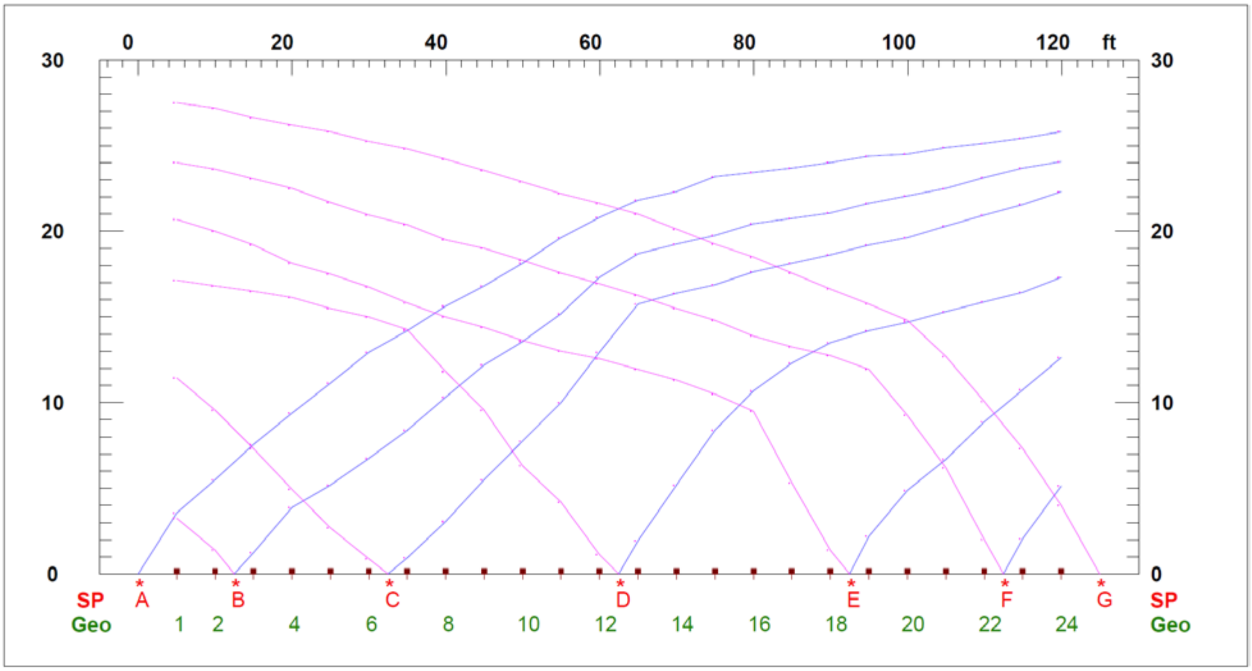
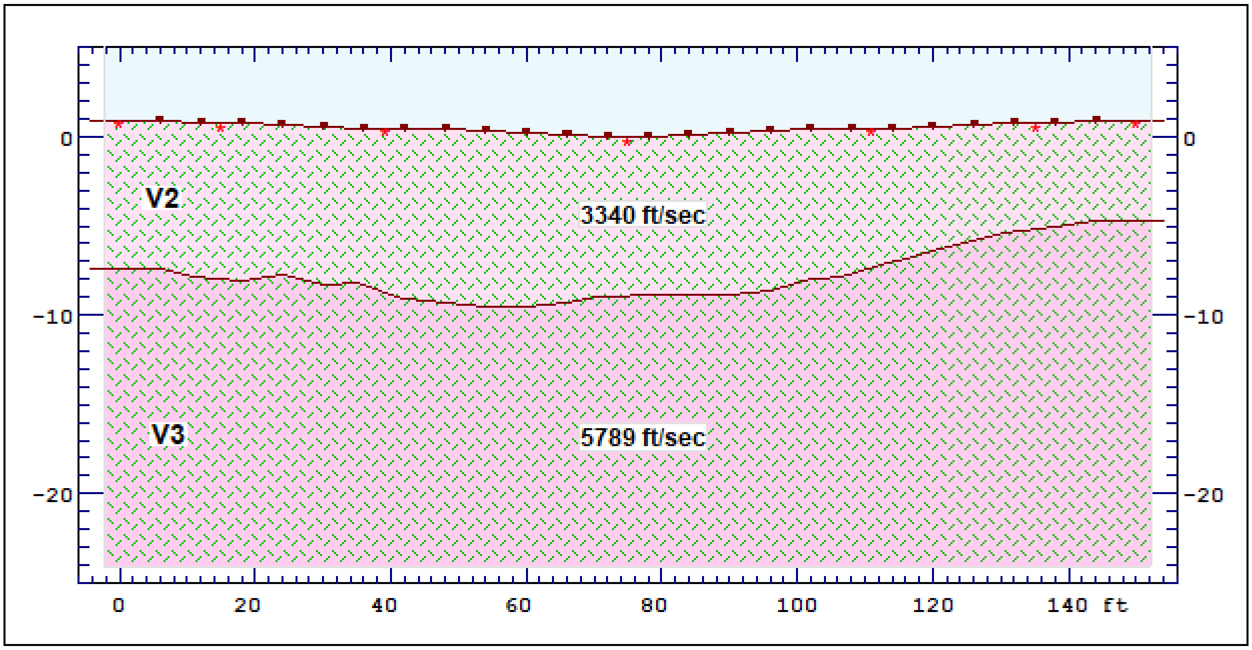


SEISMIC LINE S-2

North 33° East >

LAYER VELOCITY MODEL



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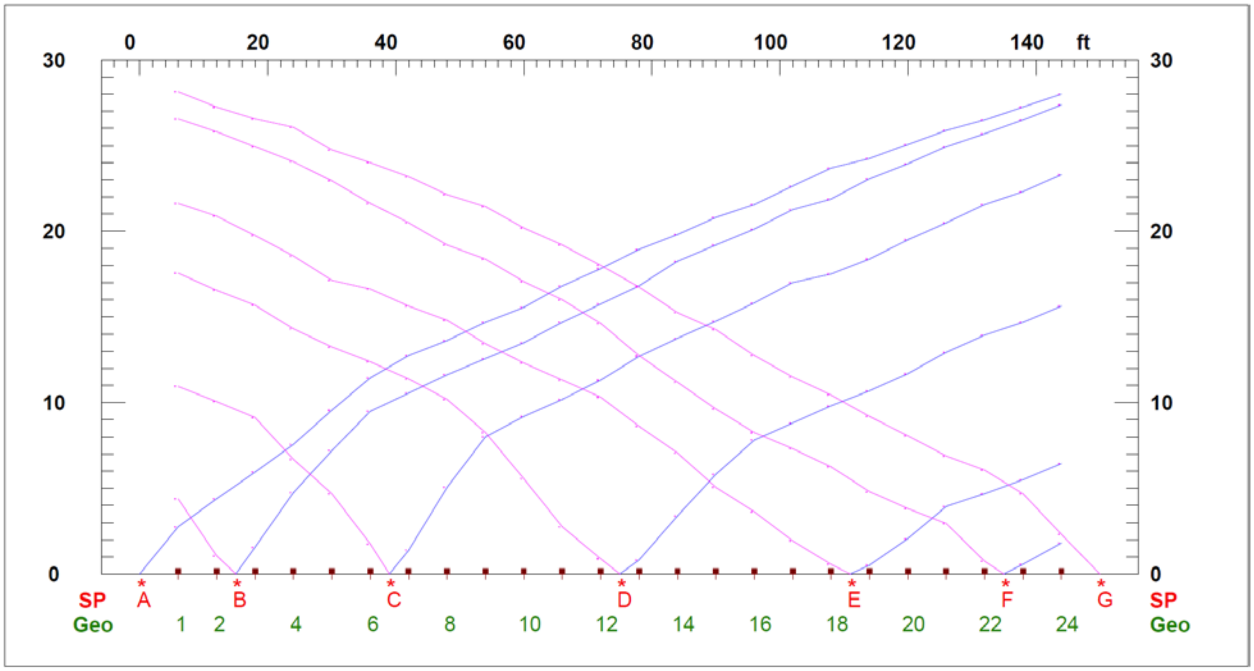
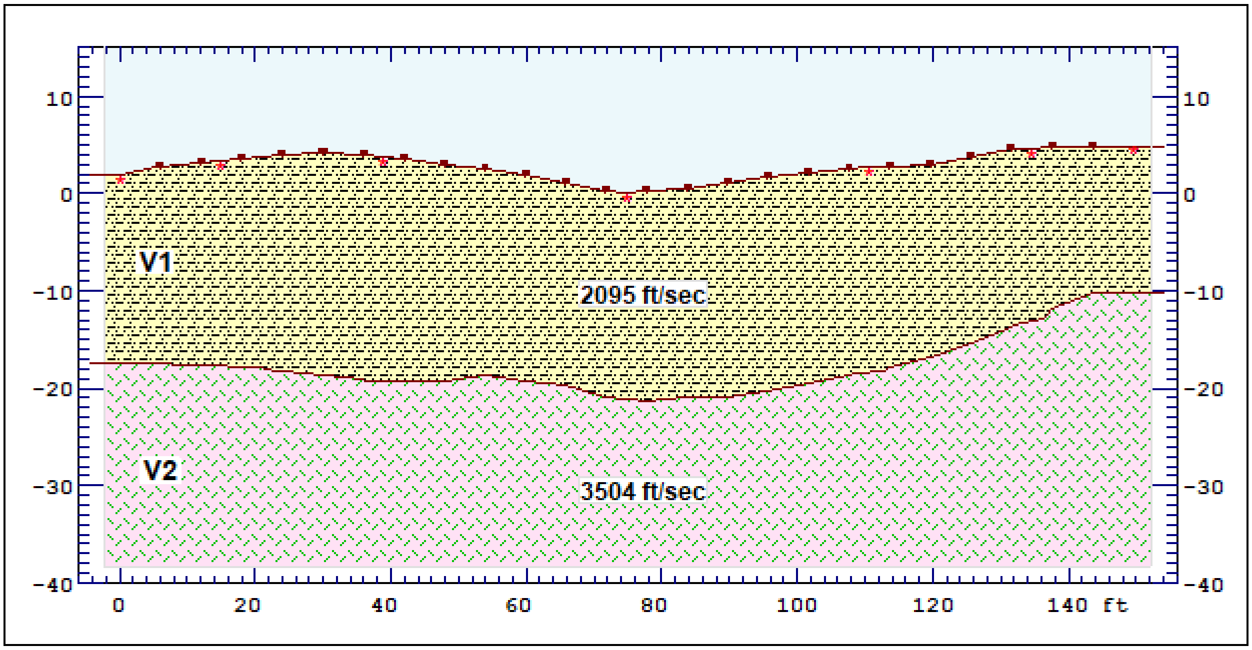


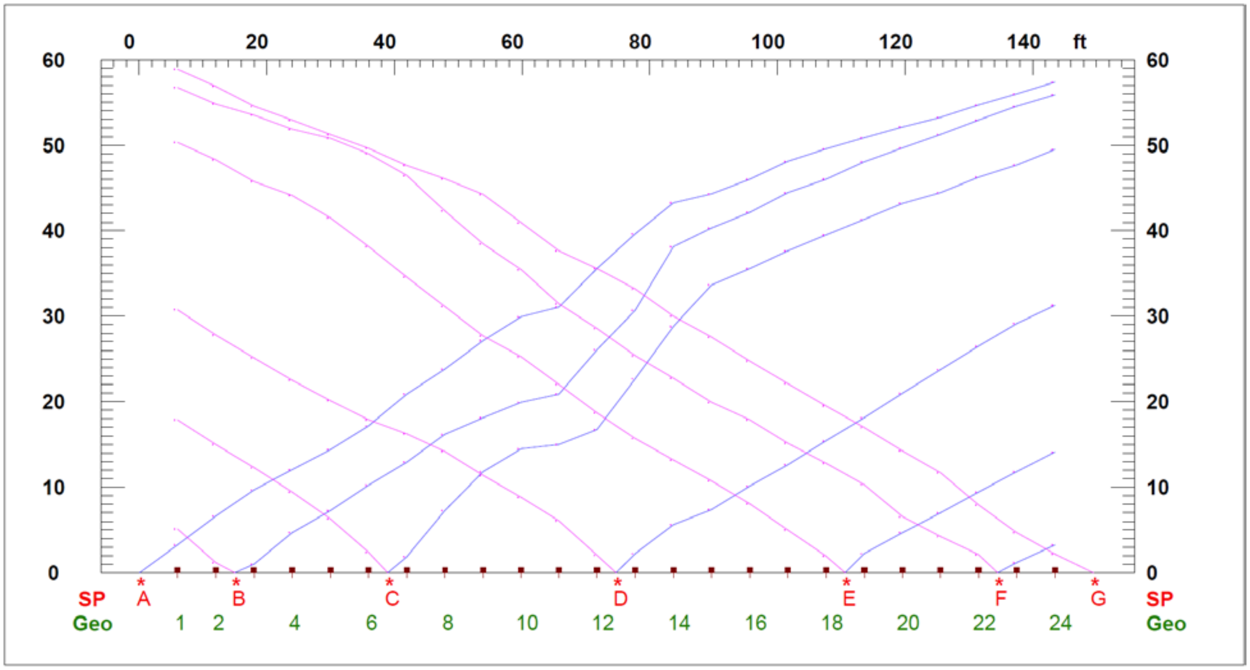
SEISMIC LINE S-3

North 38° West >

LAYER VELOCITY MODEL



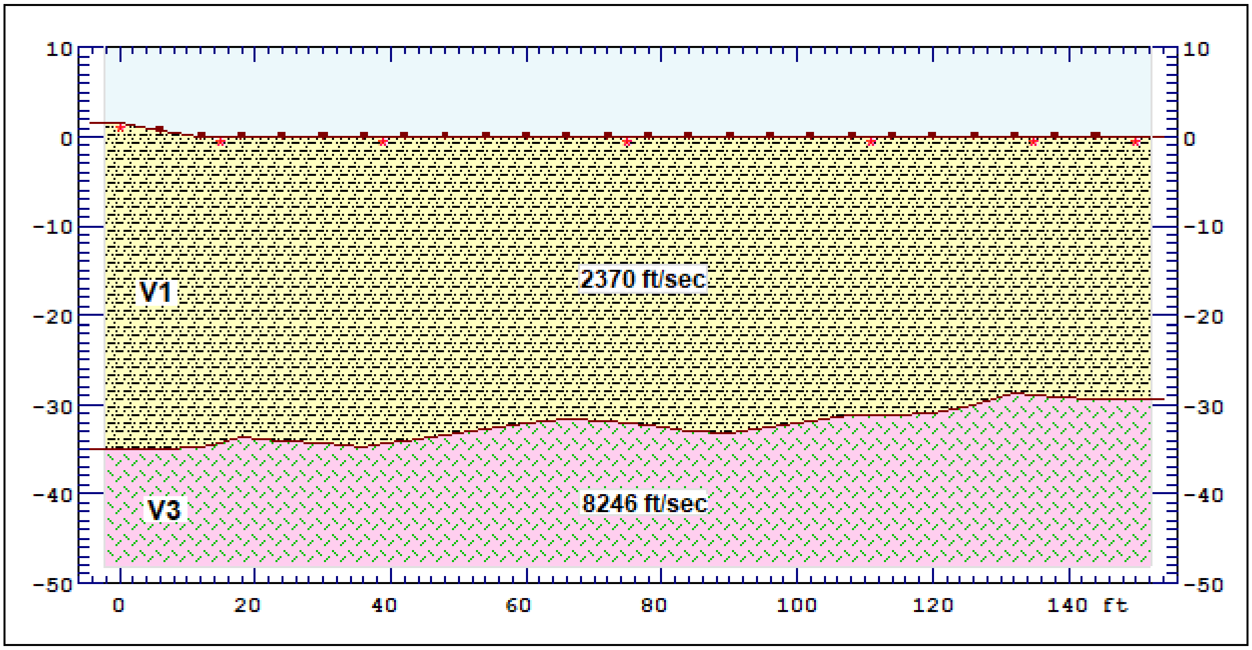
TIME-DISTANCE PLOT



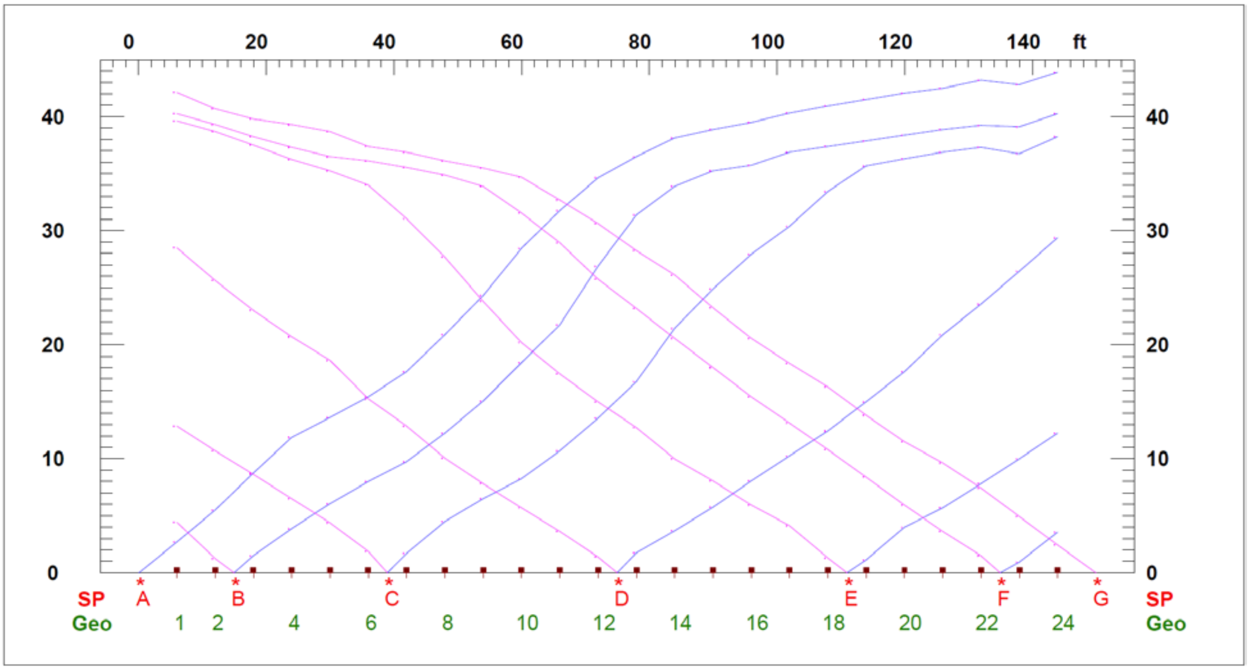
SEISMIC LINE S-4

North 19° West >

LAYER VELOCITY MODEL



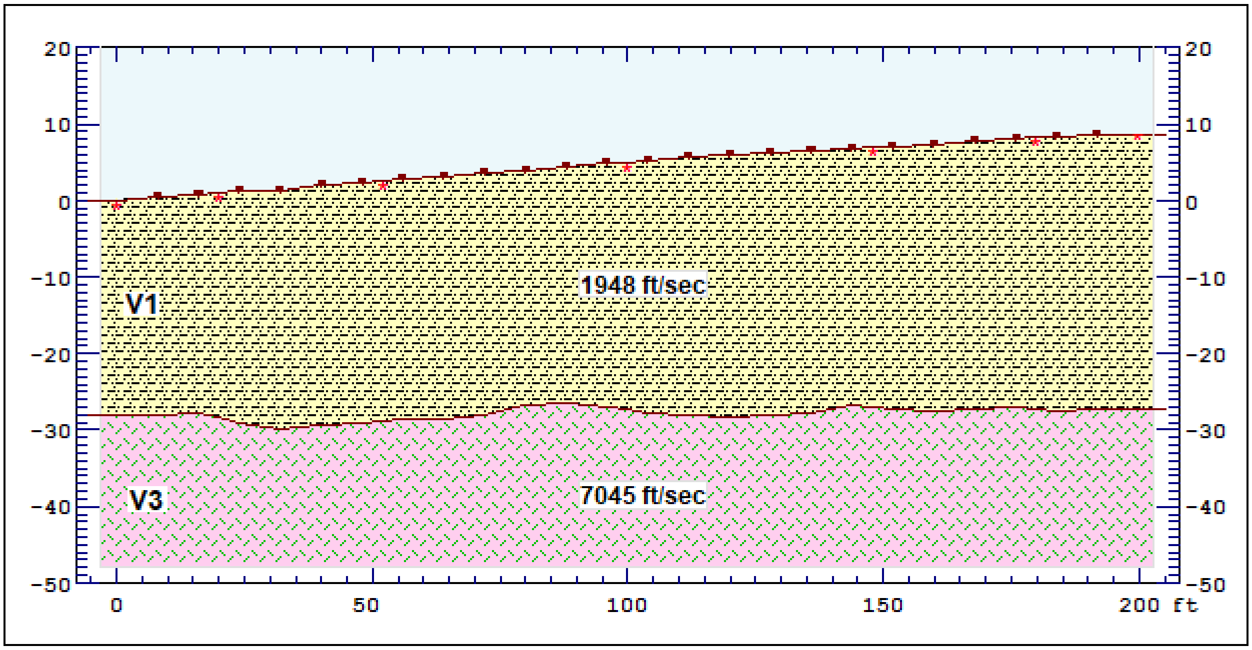
TIME-DISTANCE PLOT



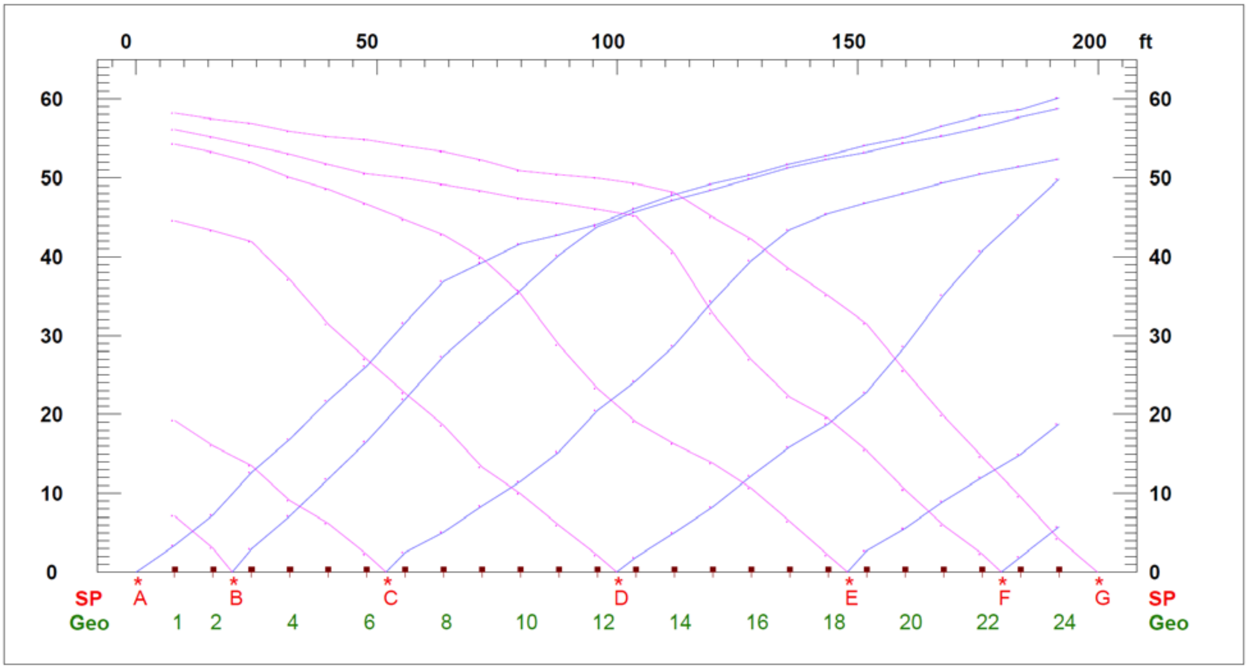
SEISMIC LINE S-5

North 9° West >

LAYER VELOCITY MODEL



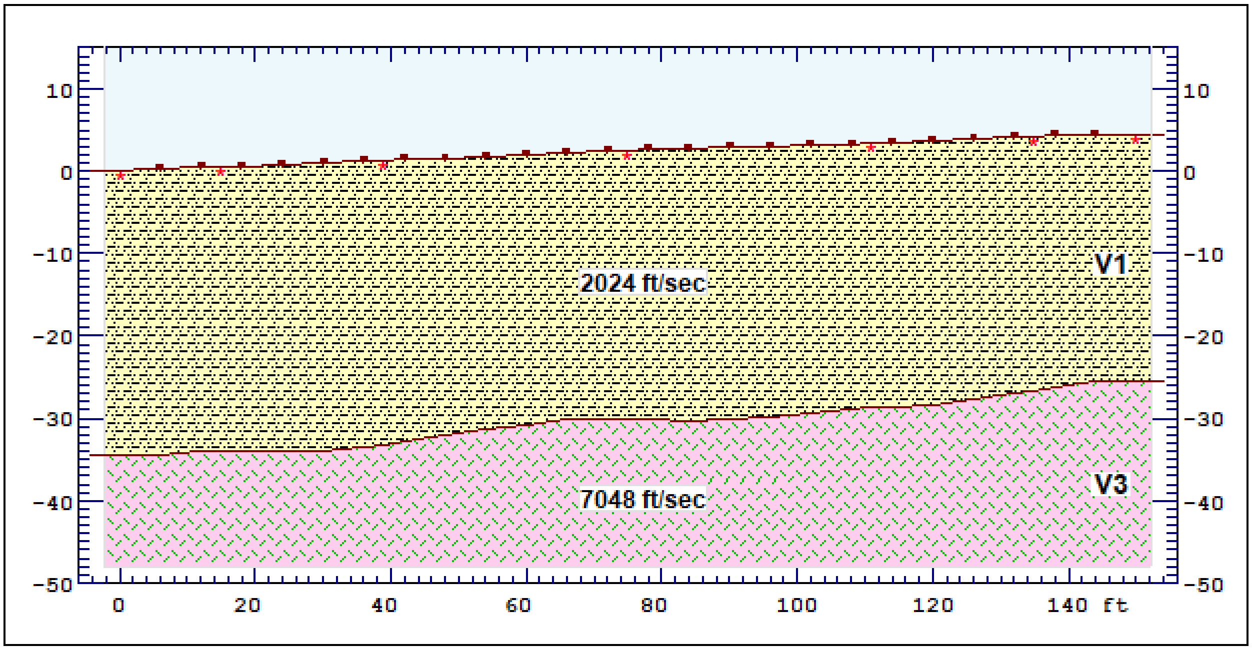
TIME-DISTANCE PLOT



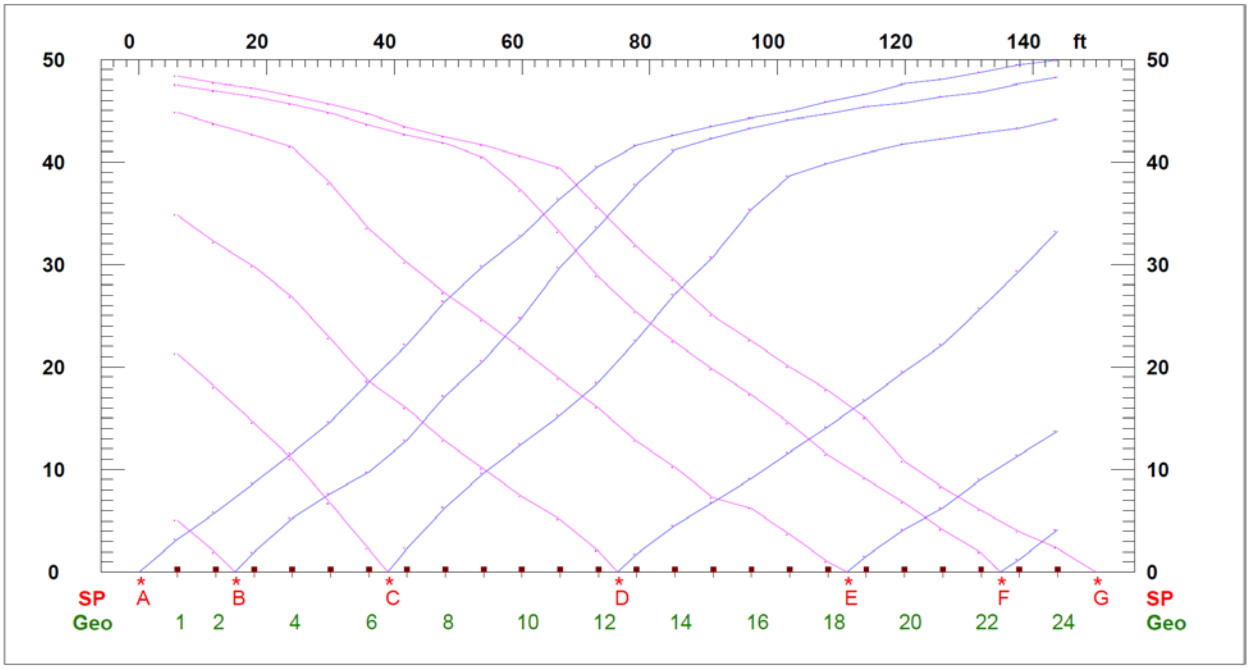
SEISMIC LINE S-6

North 12° West >

LAYER VELOCITY MODEL



TIME-DISTANCE PLOT



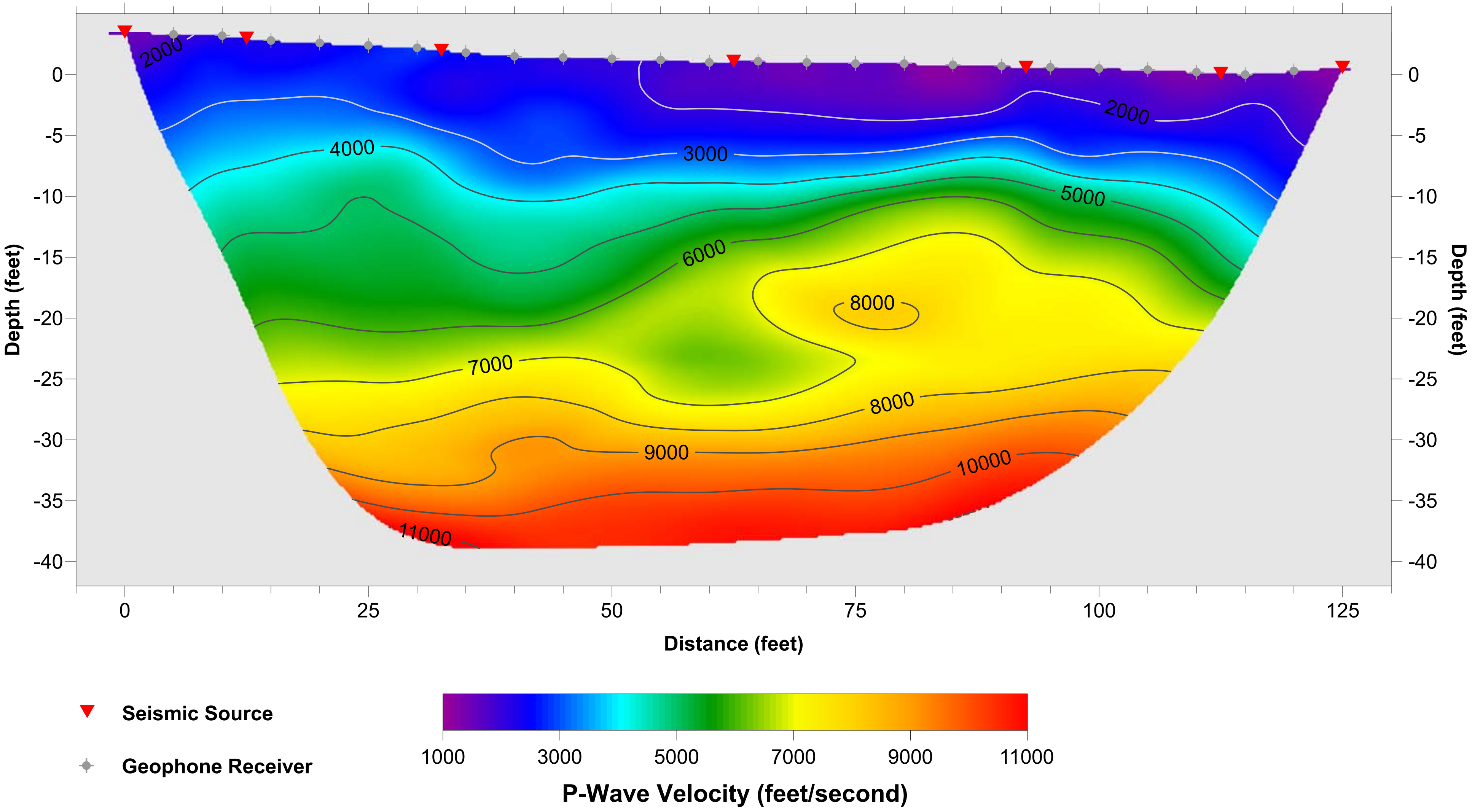
APPENDIX B

REFRACTION TOMOGRAPHIC MODELS

SEISMIC LINE S-1

North 51° East →

REFRACTION TOMOGRAPHIC MODEL



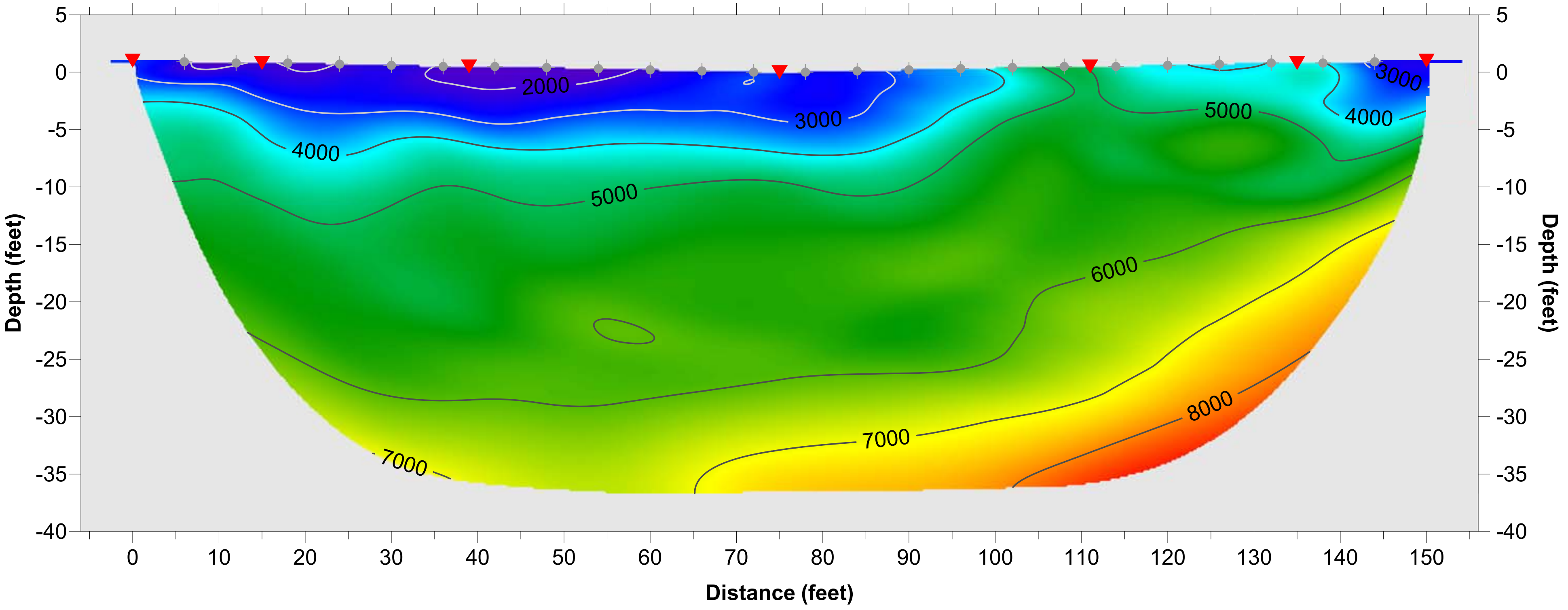
SCALE: Vertical Exaggeration 1.25X

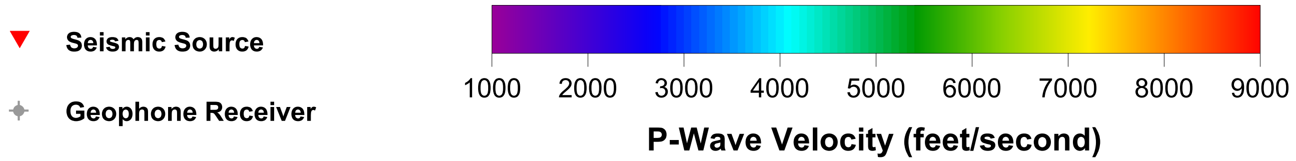
RMS error 3.7%; Rayfract Version 4.01

SEISMIC LINE S-2

North 33° East →

REFRACTION TOMOGRAPHIC MODEL



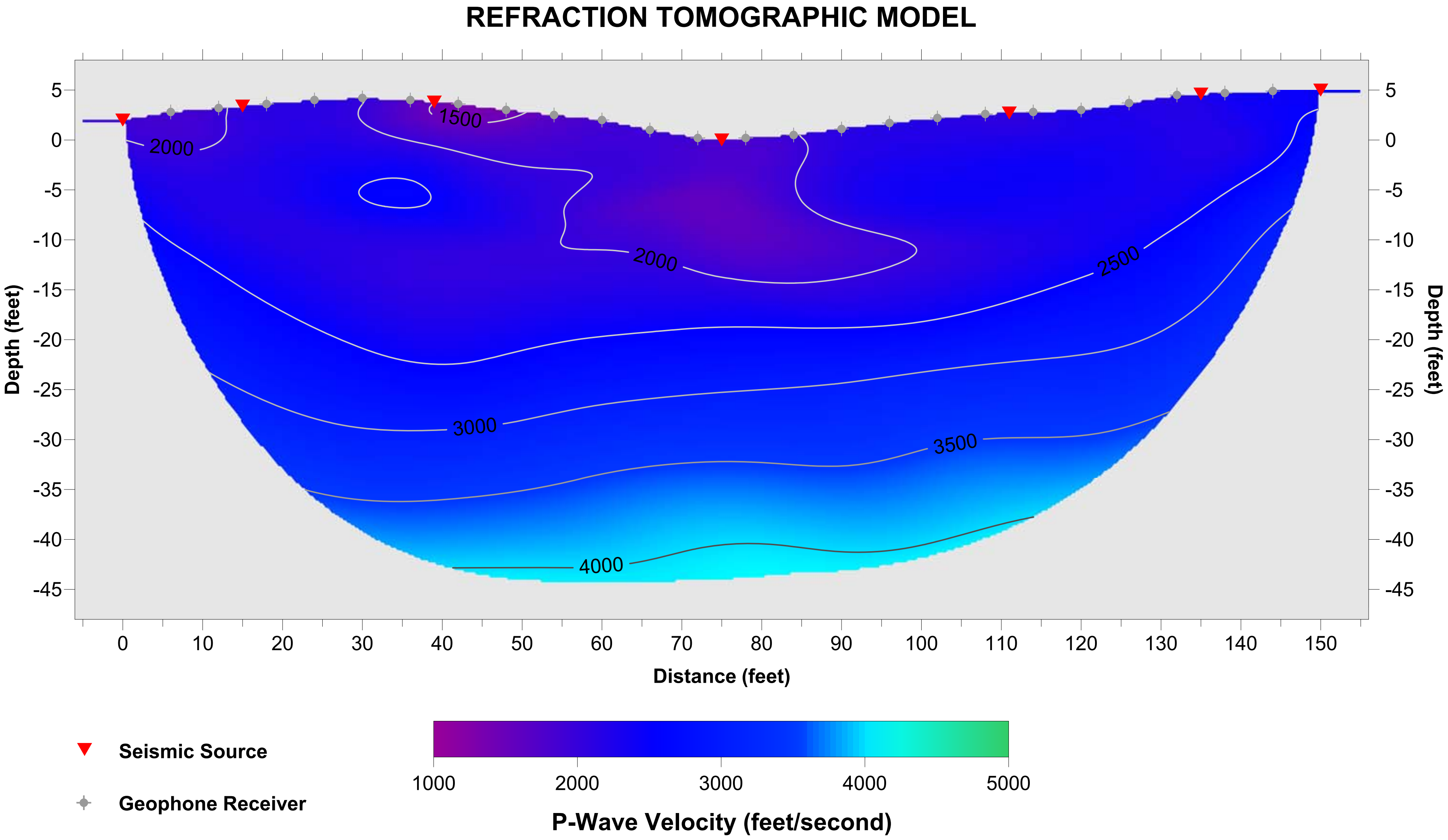


SCALE: Vertical Exaggeration 1.33X

RMS error 3.2%; Rayfract Version 4.01

SEISMIC LINE S-3

North 38° West →



REFRACTION TOMOGRAPHIC MODEL

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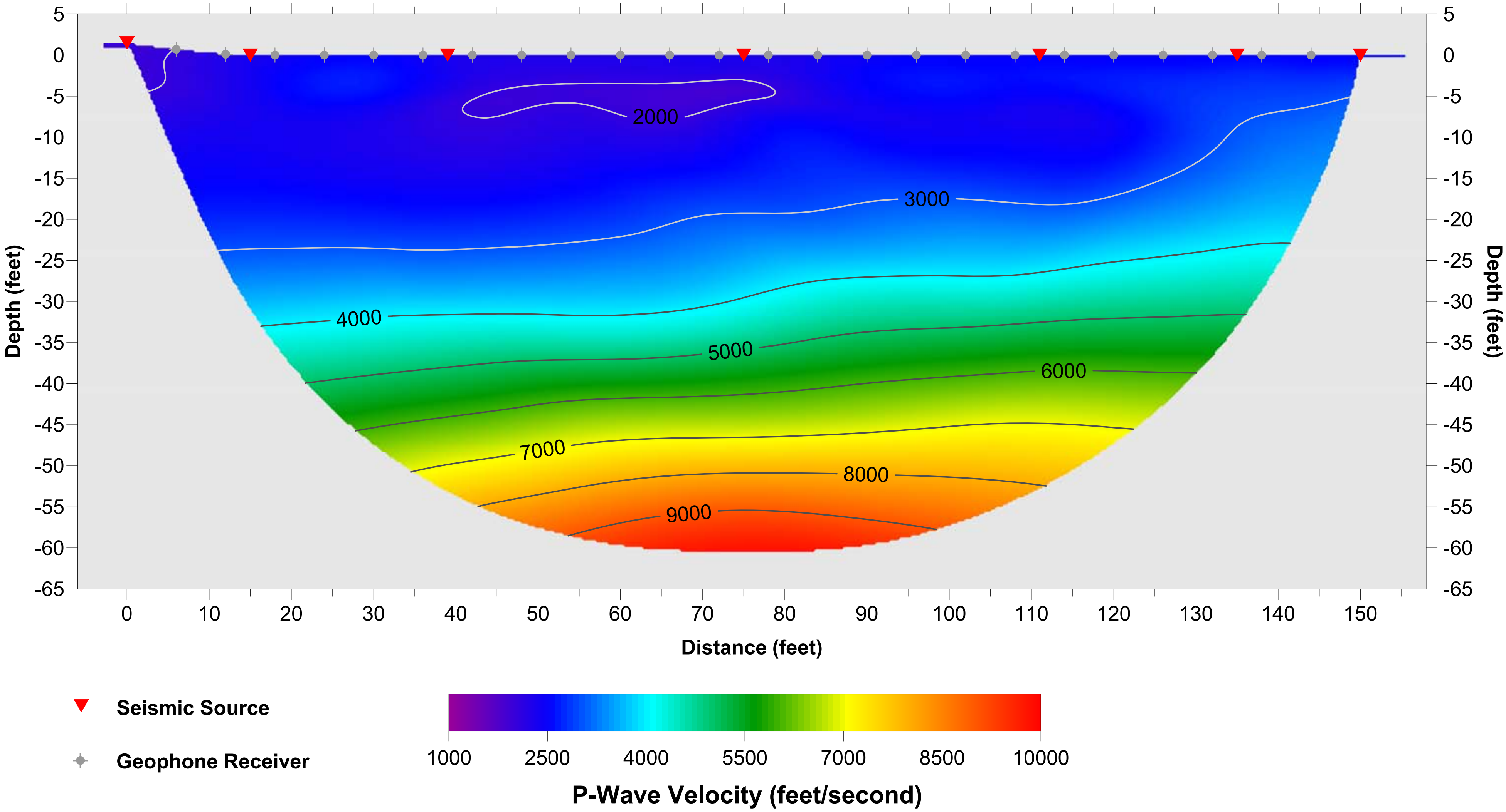
RMS error 5.1%; Rayfract Version 4.01

SEISMIC LINE S-4

North 19° West →

REFRACTION TOMOGRAPHIC MODEL



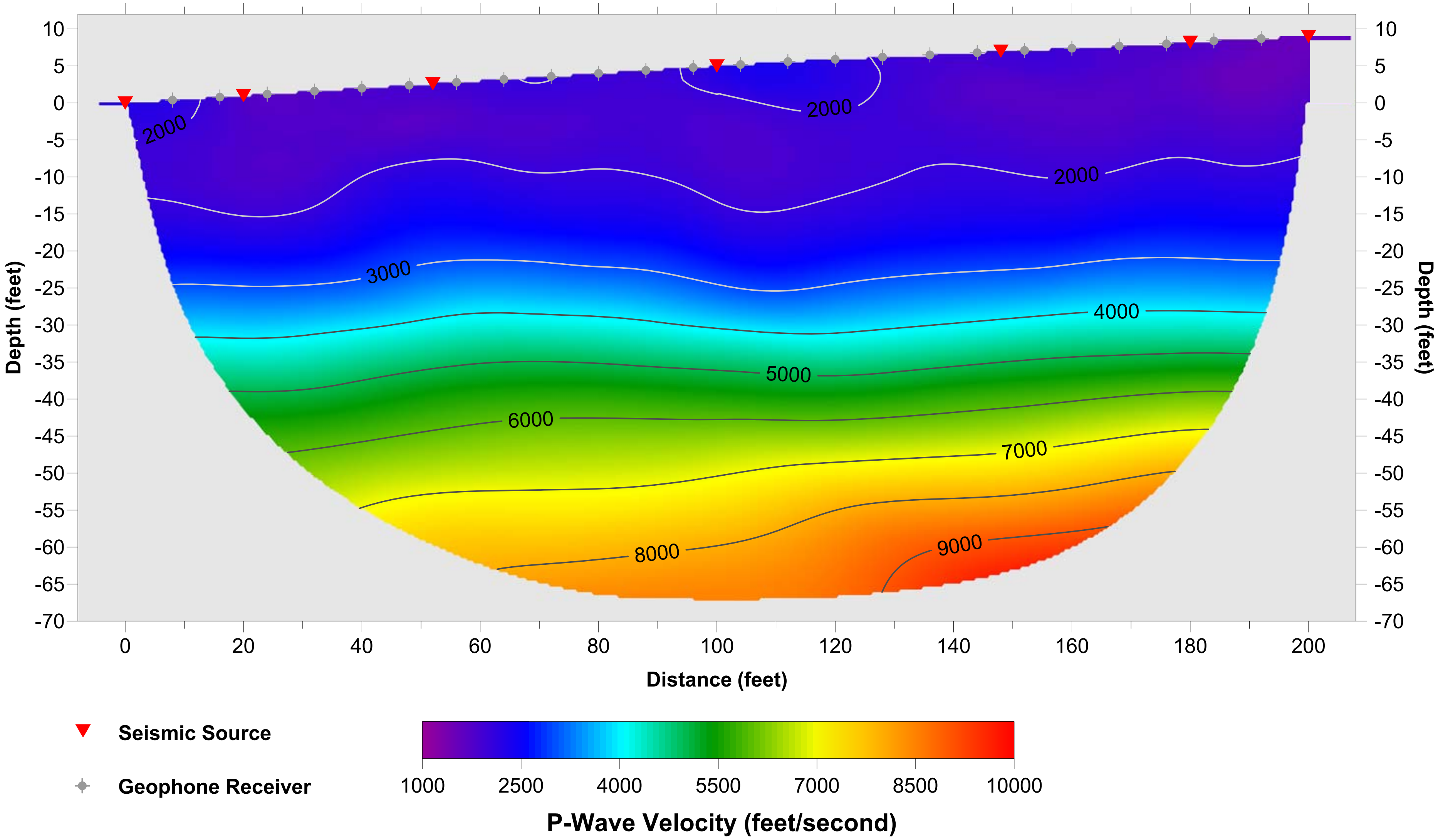
SCALE: Vertical Exaggeration 1.33X

RMS error 4.9%; Rayfract Version 4.01

SEISMIC LINE S-5

North 9° West →

REFRACTION TOMOGRAPHIC MODEL



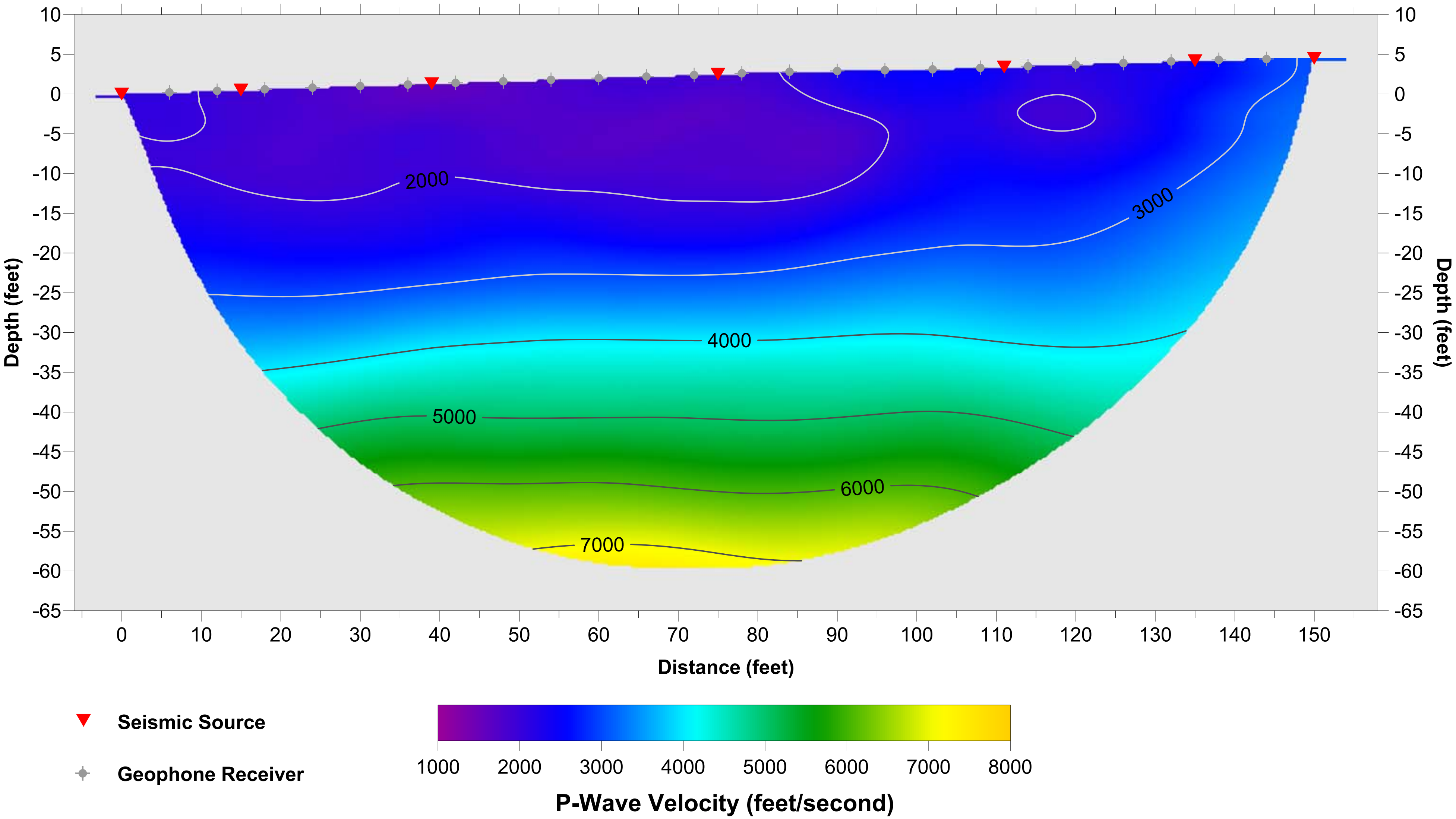
SCALE: Vertical Exaggeration 1.33X

RMS error 4.9%; Rayfract Version 4.01

SEISMIC LINE S-6

North 12° West →

REFRACTION TOMOGRAPHIC MODEL



SCALE: 1:1 (Horizontal = Vertical)

RMS error 5.9%; Rayfract Version 4.01

APPENDIX C

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

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

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

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

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

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

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

APPENDIX D

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REFERENCES

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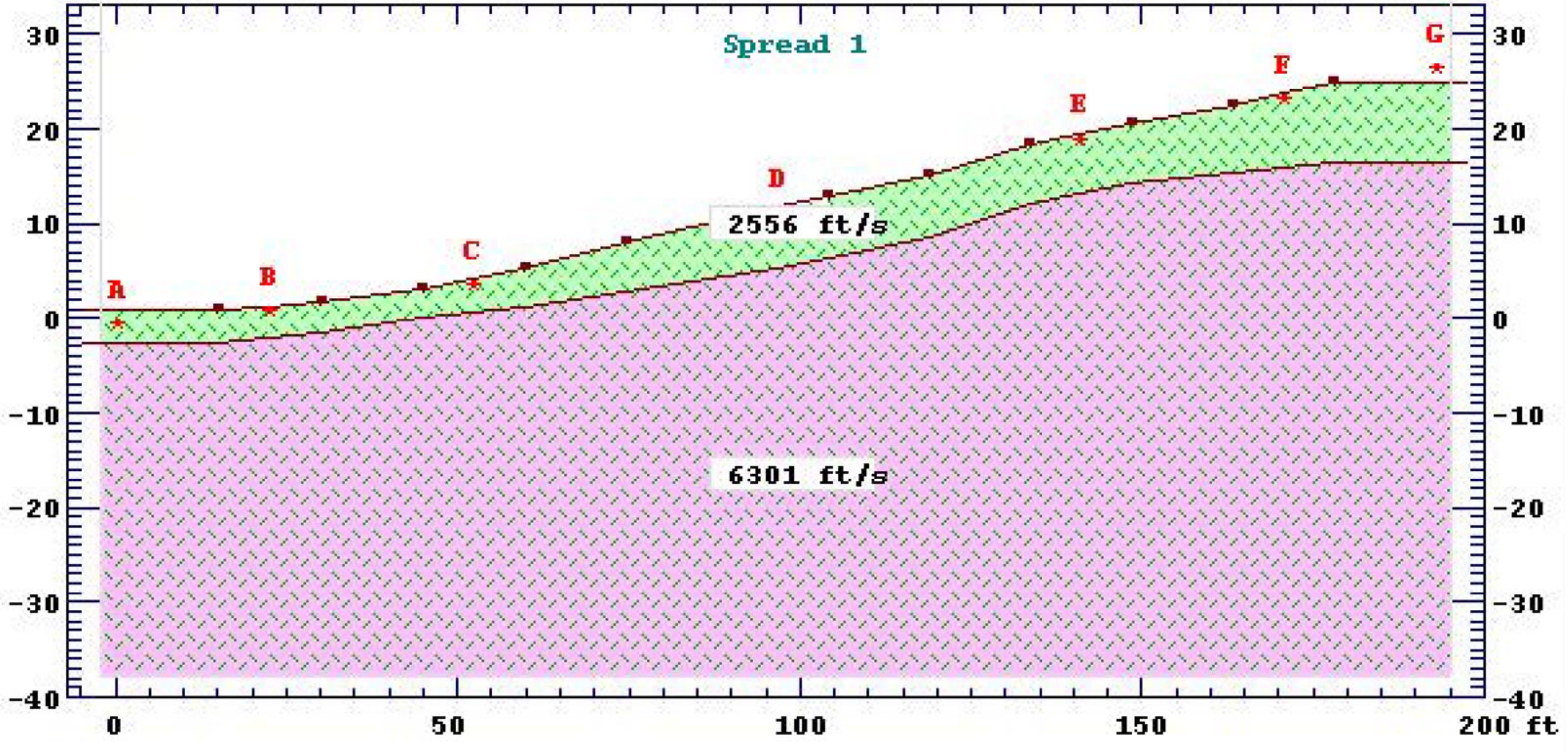
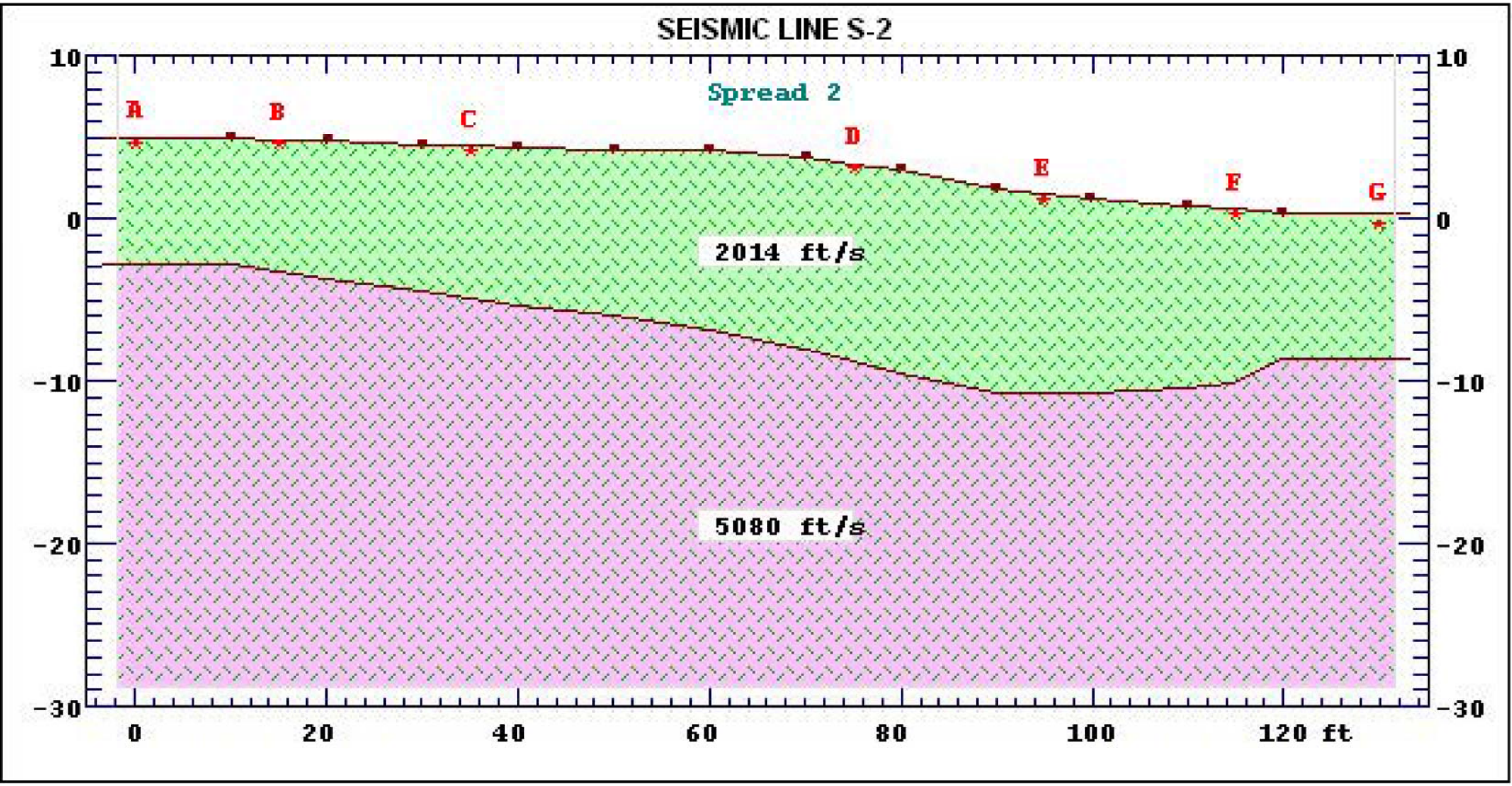
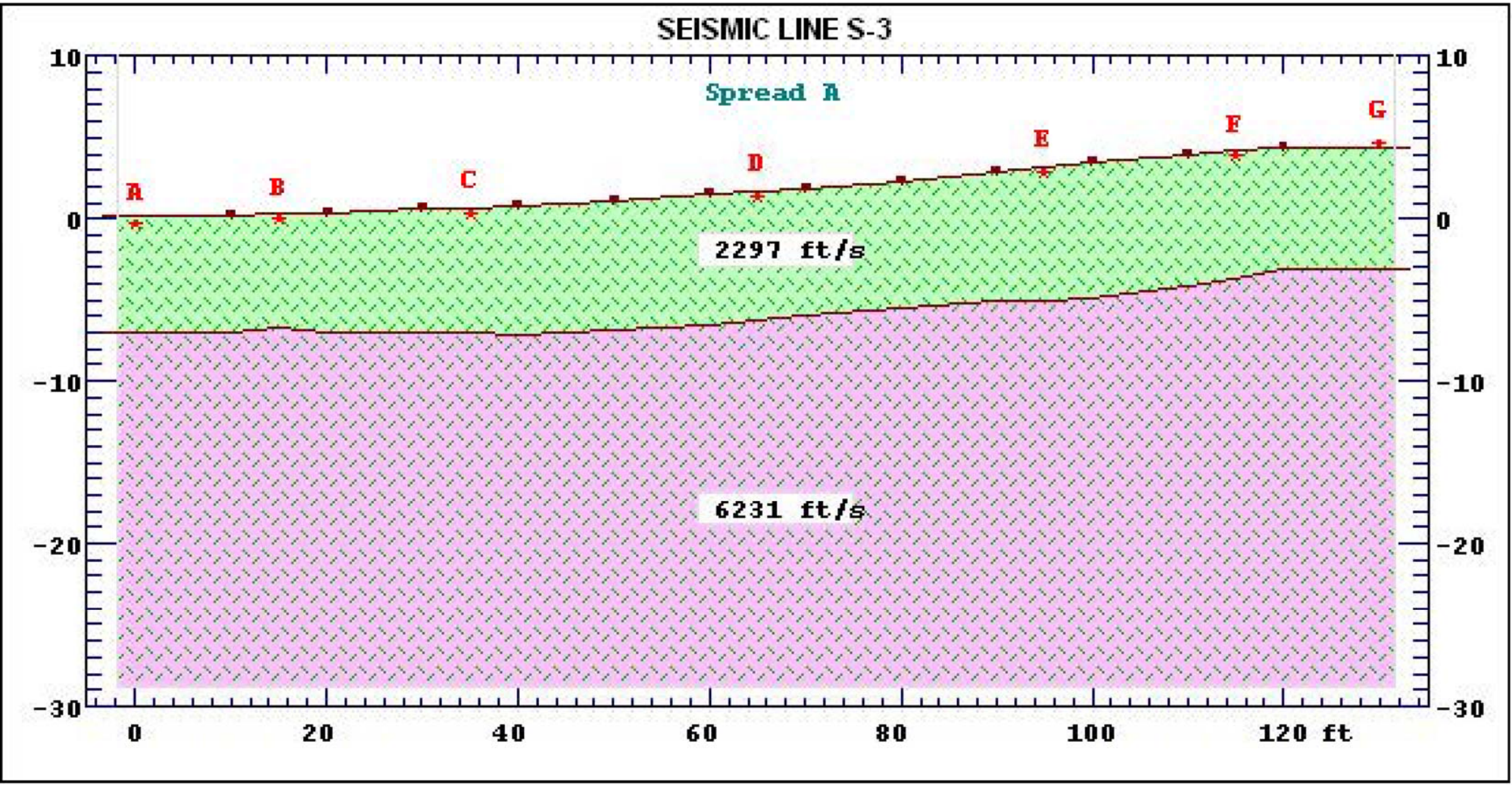
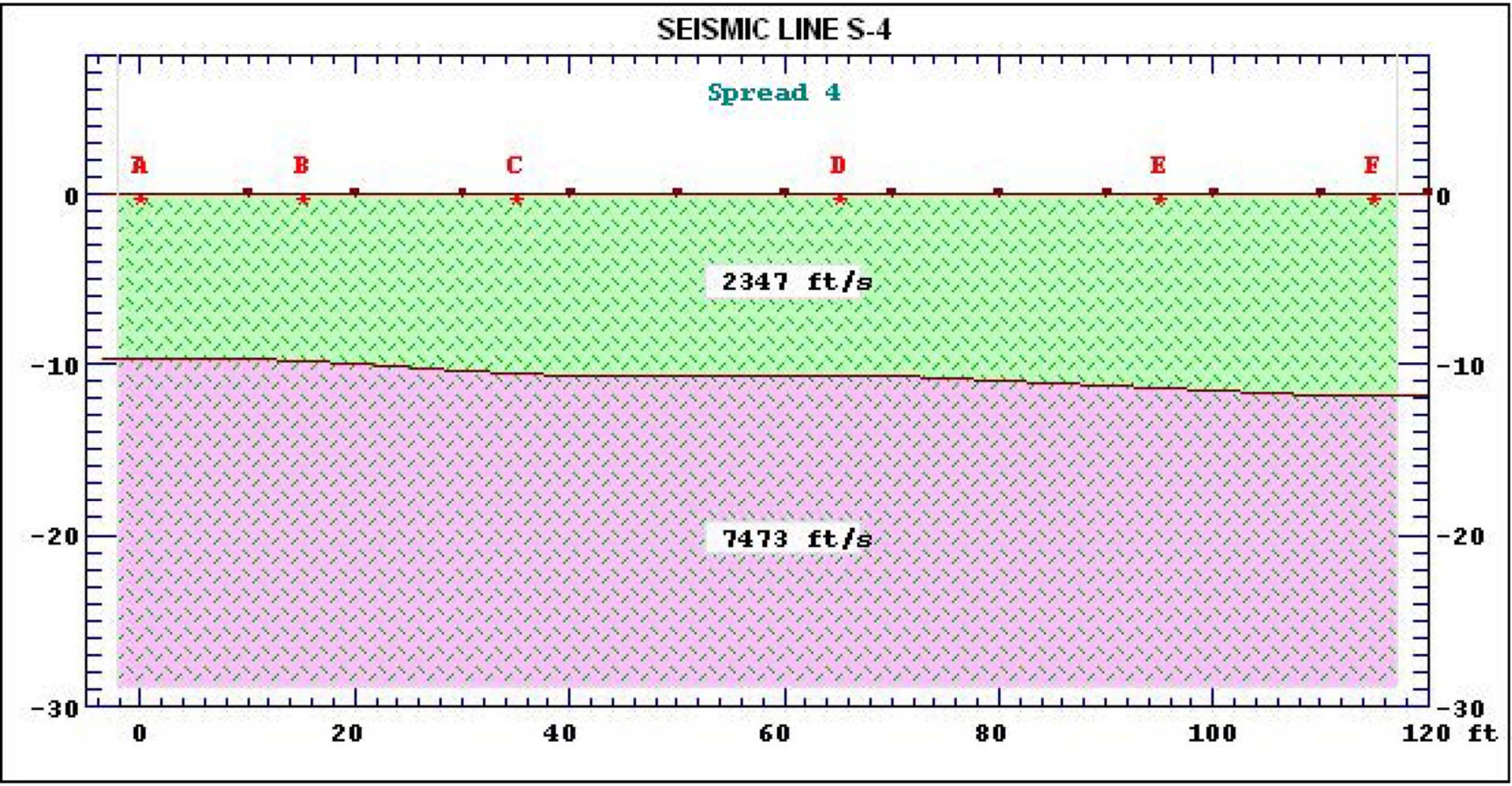
APPENDIX E
SEISMIC TRAVERSES BY OTHERS

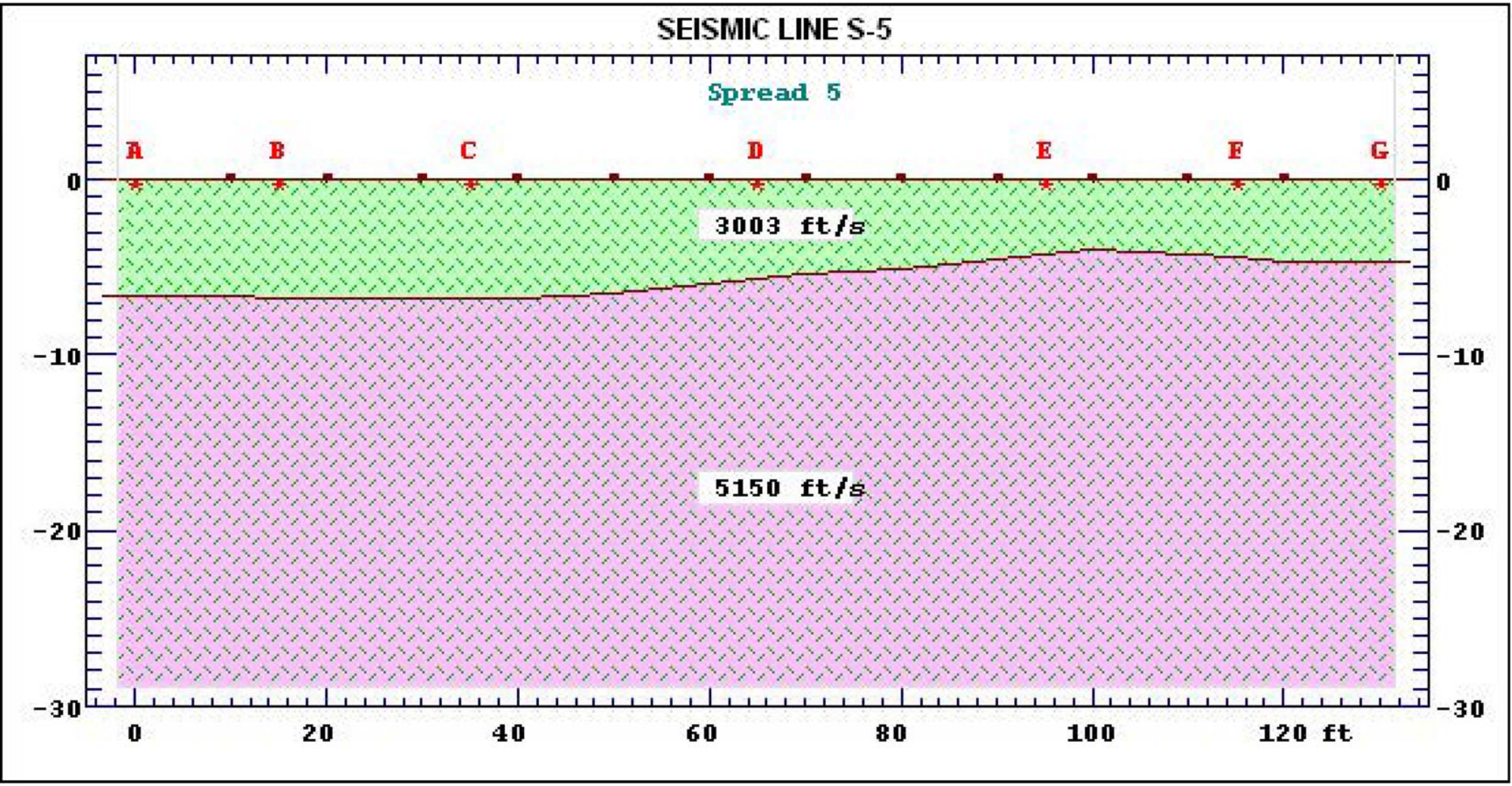
SEISMIC LINE S-1




SEISMIC LINE S-2
Spread 2
A
B
C
D
E
F
G
2014 ft/s
5080 ft/s
10
0
-10
-20
-30
0
20
40
60
80
100
120 ft


SEISMIC LINE S-3
Spread A
A
B
C
D
E
F
G
2297 ft/s
6231 ft/s
10
0
-10
-20
-30
0
20
40
60
80
100
120 ft


SEISMIC LINE S-4
Spread 4
A
B
C
D
E
F
2347 ft/s
7473 ft/s
0
-10
-20
-30
0
20
40
60
80
100
120 ft



SEISMIC LINE S-5

APPENDIX F
LABORATORY ANALYTICAL RESULTS

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 in the exploration logs.*
- **MOISTURE CONTENT (ASTM D2216):** *Tests were performed on selected represenative soil samples to evaluate the water (moisture) content by mass of soil, rock, and similar materials where the reduction in mass by drying is due to loss of water. Test sample is dried in an oven at a temperature of 110° ± 5°C to a constant mass. The loss of mass due to drying is considered to be water. The water (moisture) content were determined in general accordance with ASTM D2216.*
- **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.*
- **DIRECT SHEAR TEST (ASTM D3080):** *Direct shear tests were performed on remolded and relatively undisturbed samples in general accordance with ASTM D3080 to evaluate the shear stregth characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions.*
- **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.*
- **R-VALUE (ASTM D2844):** *The resistance Value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D2844. 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.*
- **GRADATION ANALYSIS (ASTM C 136 and/or ASTM D422):** *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 C 136 and/or ASTM D422.*

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LAB TEST SUMMARY

CRESTVIEW APARTMENTS
SYCAMORE CANYON BLVD. & CENTRAL AVE.
RIVERSIDE, CALIFORNIA

BY: DTW
DATE: SEPTEMBER 2020
PROJECT: 3020003

Moisture Content Test (ASTM D2216)

Sample Location	Sample Depth (ft)	Soil Description	Moisture (%)
TP-1	0.0'-5.0'	Sand with Silt	2.0
TP-2	5.0'-8.0'	Clayey Sand	3.9
TP-7	0.0'-5.0'	Silty Sand	4.5
TP-11	5.0'-7.0'	Sand with Silt	3.2

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

Sample Location	Sample Depth (ft.)	Soil Description	R-Value
TP-7	0.0' - 5.0'	Silty Sand	70

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP-1	0.0' - 5.0'	Sand with Silt	127.4	8.8
TP-7	0.0' - 5.0'	Silty Sand	132.5	9.7

Direct Shear (ASTM D3080)

Sample Location	Depth (feet)	Soil Description	Friction Angle Peak/Ultimate (degrees)	Apparent Cohesion Peak/Ultimate (psf)
TP-1	0.0'-5.0'	Sand with Silt	41/36	542/496
TP-7	0.0'-5.0'	Silty Sand	40/36	283/195

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-1	0.0'-5.0'	8.0	6700	15	0.002	11	0.001
TP-7	0.0'-5.0'	8.2	2600	96	0.010	21	0.002

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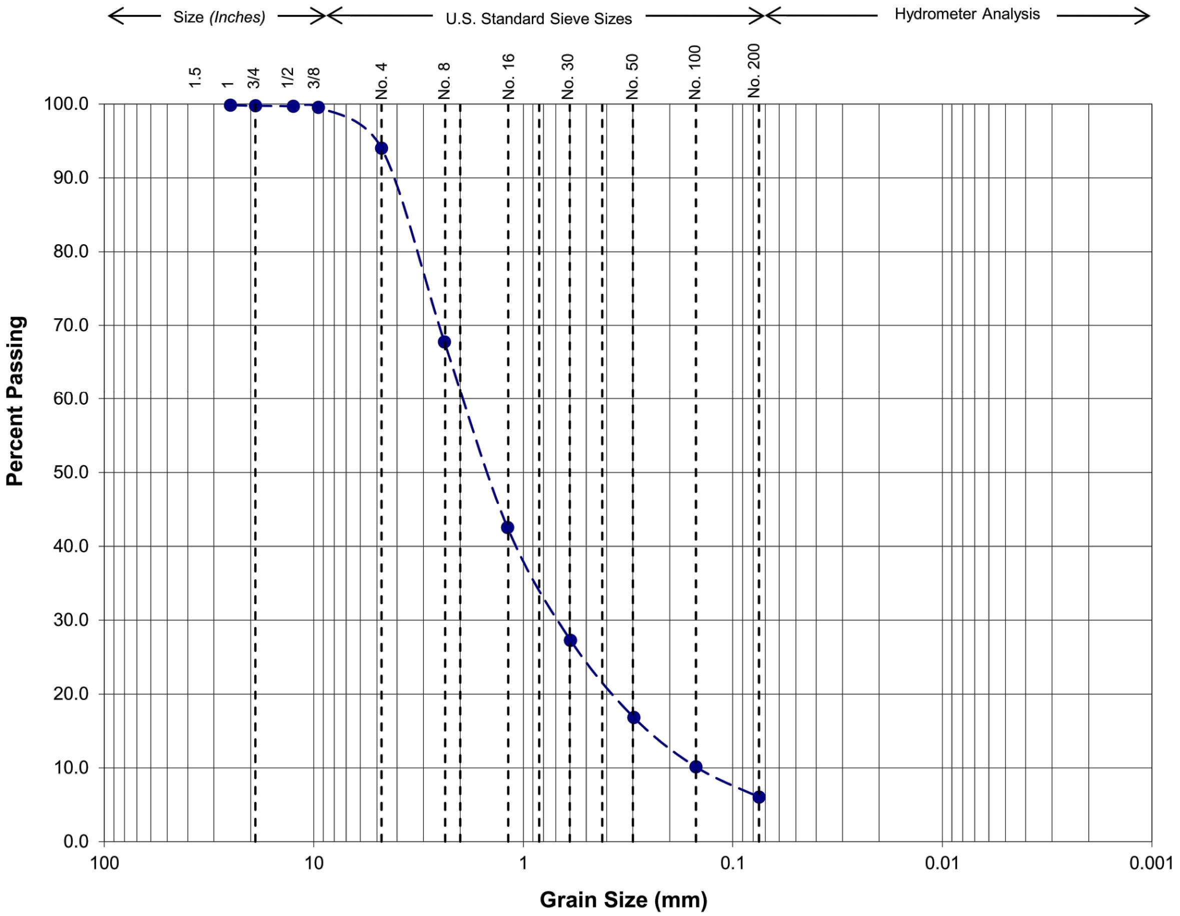
LAB TEST RESULTS

CRESTVIEW APARTMENTS

SYCAMORE CANYON BLVD. & CENTRAL AVE.
RIVERSIDE, CALIFORNIA

BY: DTW	DATE: SEPTEMBER 2020	PROJECT: 3020003



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-1

Depth (ft): 0.0'-5.0'

USCS Soil Type: SP-SM

Passing No. 200 (%): 6

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GRADATION ANALYSIS TEST RESULTS

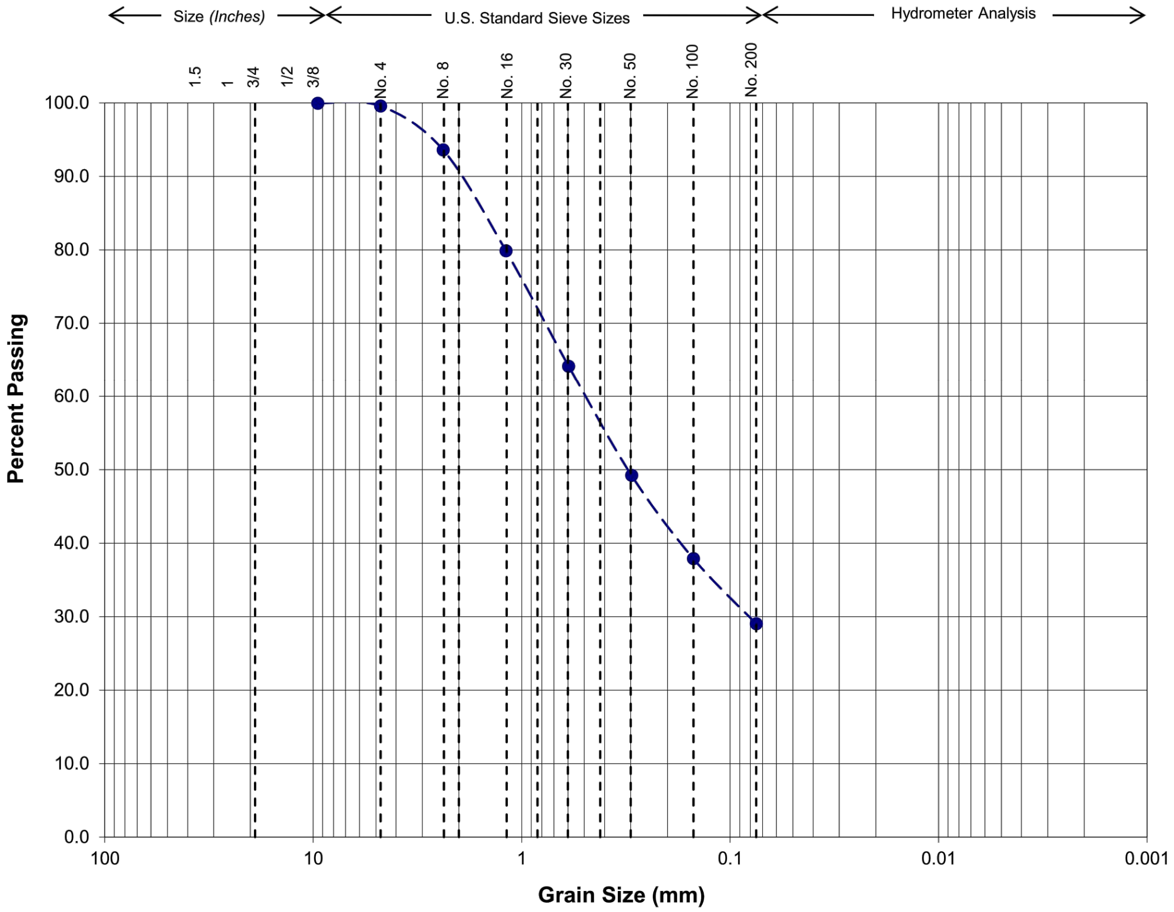
CRESTVIEW APARTMENTS

SYCAMORE CANYON BLVD. & CENTRAL AVE.
RIVERSIDE, CALIFORNIA

BY: DTW

DATE: SEPTEMBER 2020

PROJECT: 3020003



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

Sample Location: TP-2

Depth (ft): 5.0'-8.0'

USCS Soil Type: SC

Passing No. 200 (%): 29

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GRADATION ANALYSIS TEST RESULTS

CRESTVIEW APARTMENTS

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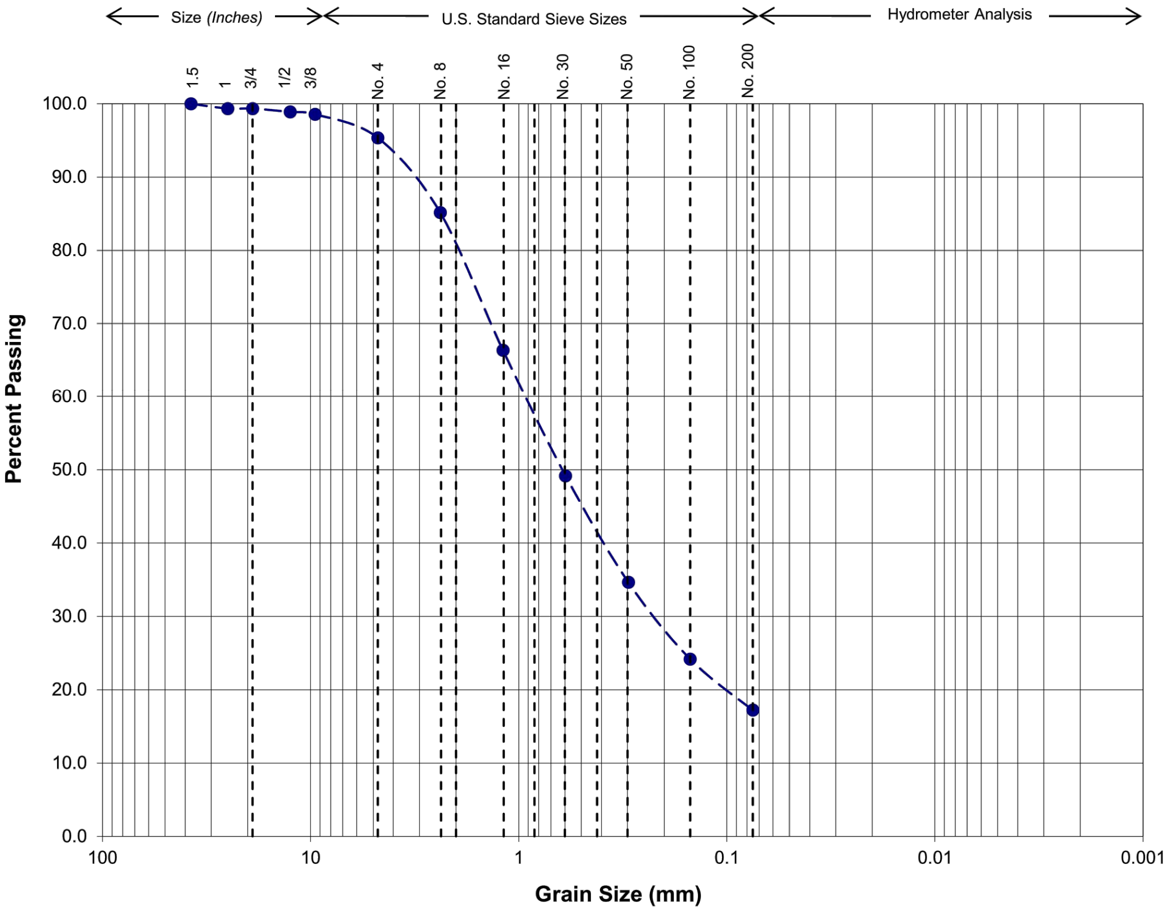
BY: DTW

DATE: SEPTEMBER 2020

PROJECT: 3020003



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: TP-7

Depth (ft): 0.0'-5.0'

USCS Soil Type: SM

Passing No. 200 (%): 17

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GRADATION ANALYSIS TEST RESULTS

CRESTVIEW APARTMENTS

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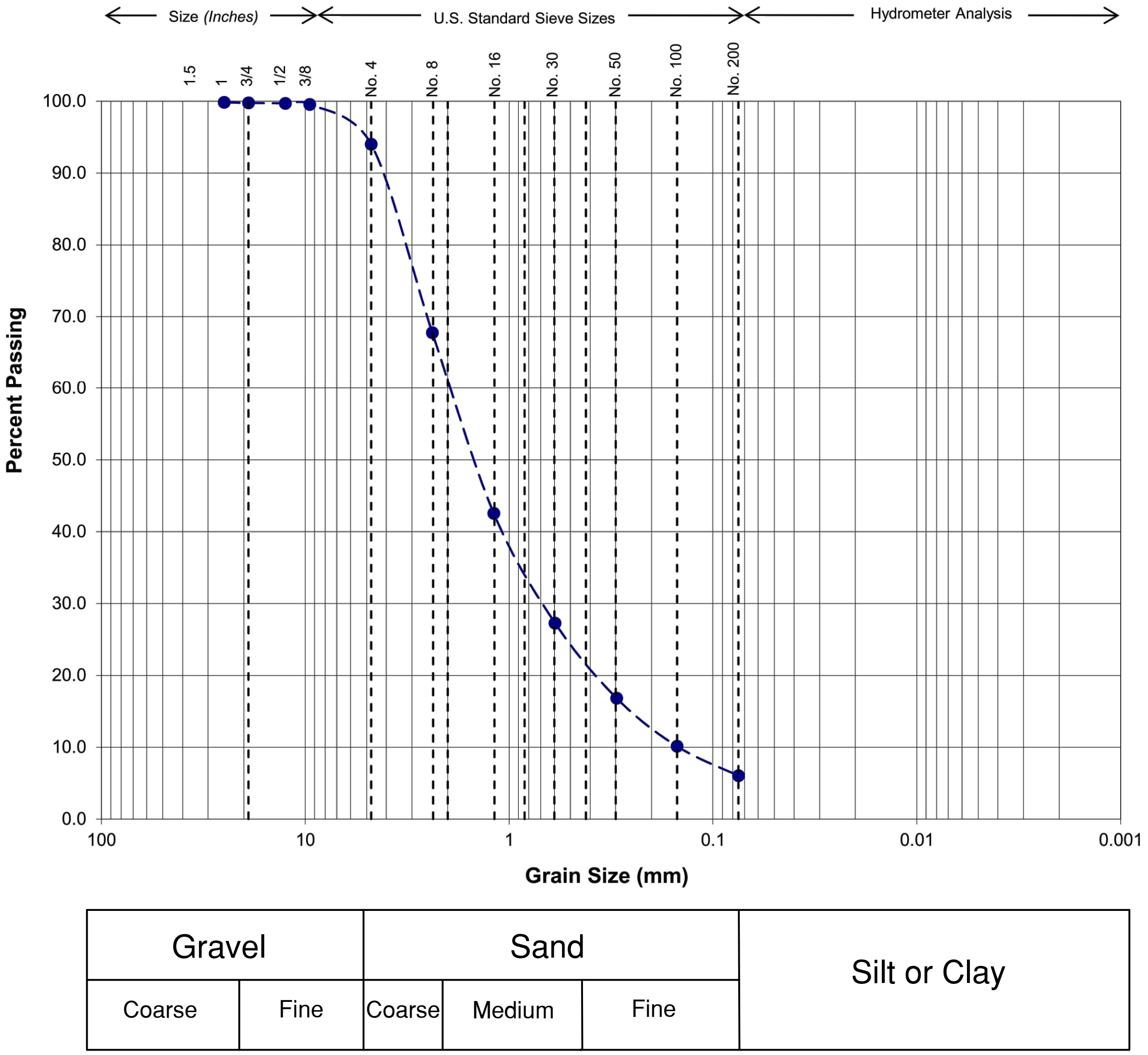
RIVERSIDE, CALIFORNIA

BY: DTW

DATE: SEPTEMBER 2020

PROJECT: 3020003



Sample Location:	TP-11
Depth (ft):	5.0'-7.0'
USCS Soil Type:	SP-SM
Passing No. 200 (%):	9

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GRADATION ANALYSIS TEST RESULTS

CRESTVIEW APARTMENTS

SYCAMORE CANYON BLVD. & CENTRAL AVE.
RIVERSIDE, CALIFORNIA

BY: DTW

DATE: SEPTEMBER 2020

PROJECT: 3020003

APPENDIX G
LABORATORY ANALYTICAL RESULTS BY OTHERS

LABORATORY TESTING PROGRAM

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content were performed on a representative sample of the site materials obtained from the field. The test was performed in accordance with ASTM D 1557-02. Pertinent test values are given in Table C.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from the subsurface exploration. The test was performed in general conformance with ASTM D 3080-80. The sample was remolded to 90 percent of maximum dry density and 2% over optimum. Three specimens were prepared for the test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate C-1.

Expansion Potential

An Expansion Index test was performed on a selected sample in accordance with Uniform Building Code Standard 18-2. The expansion potential classification was determined from C.B.C. Table 18-I-B on the basis of the expansion index value. The test result and expansion potential are presented on Table C.

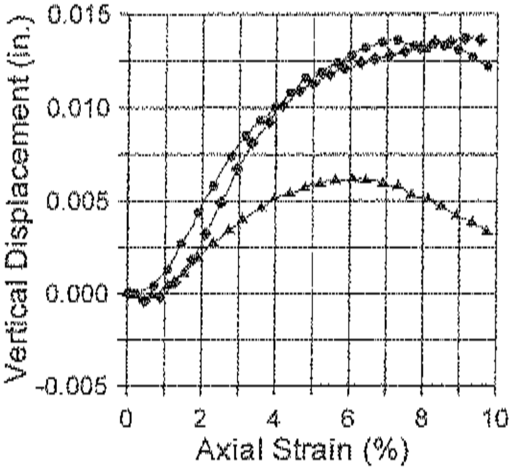
Corrosion Analysis

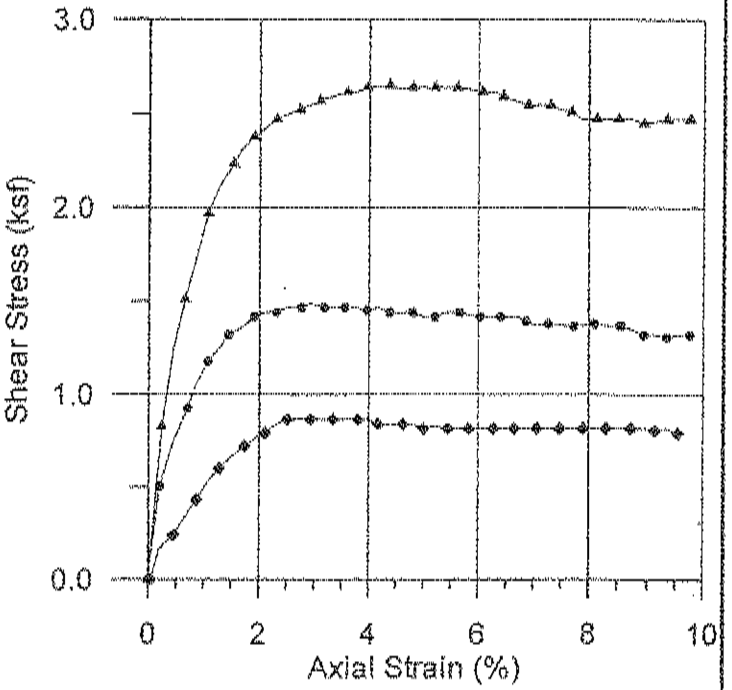
Corrosion analyses, which include soluble sulfate content, chloride content, minimum resistivity, and pH, were performed on a selected sample. The tests were performed in accordance with California Test Method (CTM) 417, CTM 422, CTM 643 and CTM 643, respectively. The test results are included in Table C.

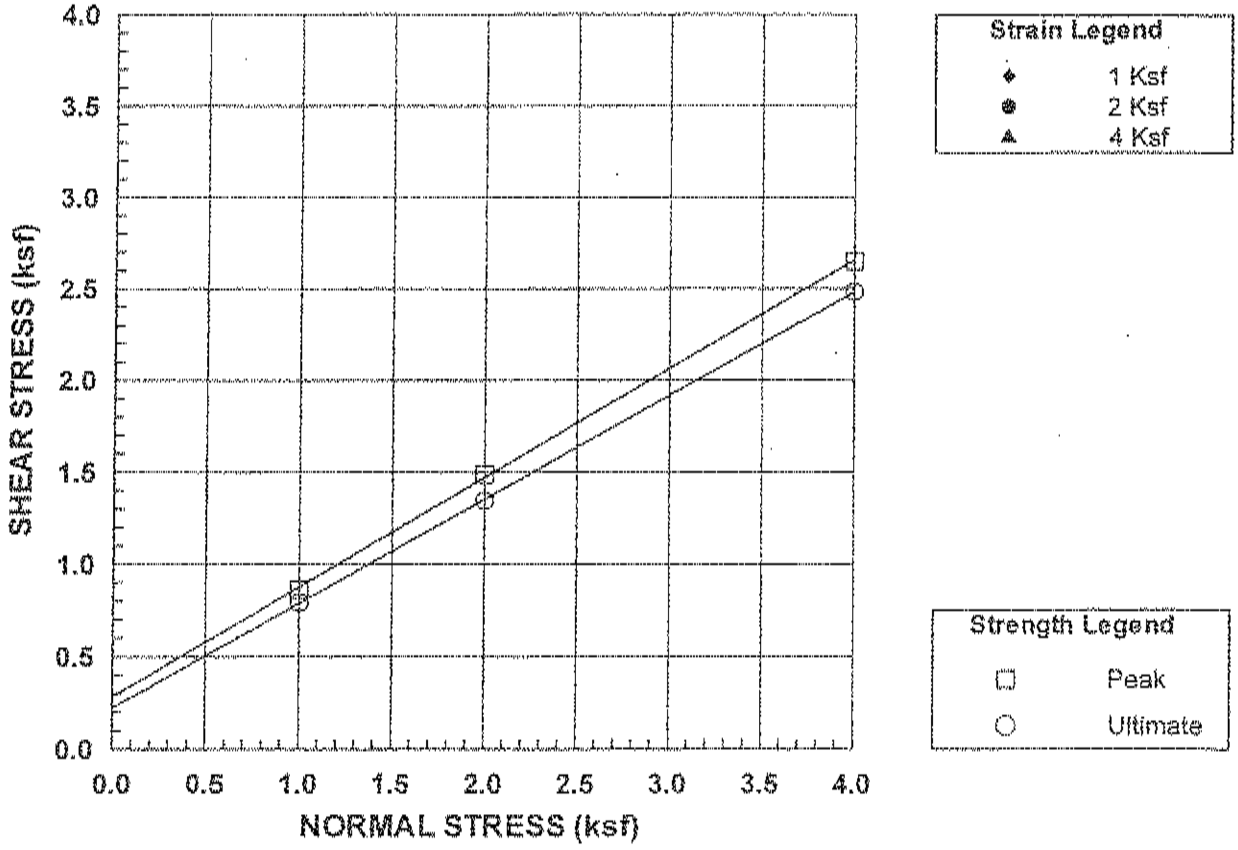
TABLE C
SUMMARY OF LABORATORY TEST RESULTS

Sample No.	Soil Description	Test Results	
S-2 @ 0-1'	Silty Sand (SM) with decomposed granite	Maximum Dry Density:	131.5 pcf
		Optimum Moisture Content:	7.0%
		Soluble Sulfate:	0.001%
		Expansion Index:	1
		Expansion Potential:	Very Low
		pH:	7.6
		Minimum Resistivity:	2400 ohm-cm
		Chloride Content:	42.5 ppm

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	0.864	1.488	2.652
Peak Displacement (in)	0.0605	0.071	0.106
Ultimate Stress (ksf)	0.792	1.344	2.484
Ultimate Displacement (in)	0.242	0.246	0.247
Initial Dry Density	118.4	118.4	118.4
Initial Moisture Content (%)	7.479	7.479	7.479
Strain Rate (in/min)	0.040		







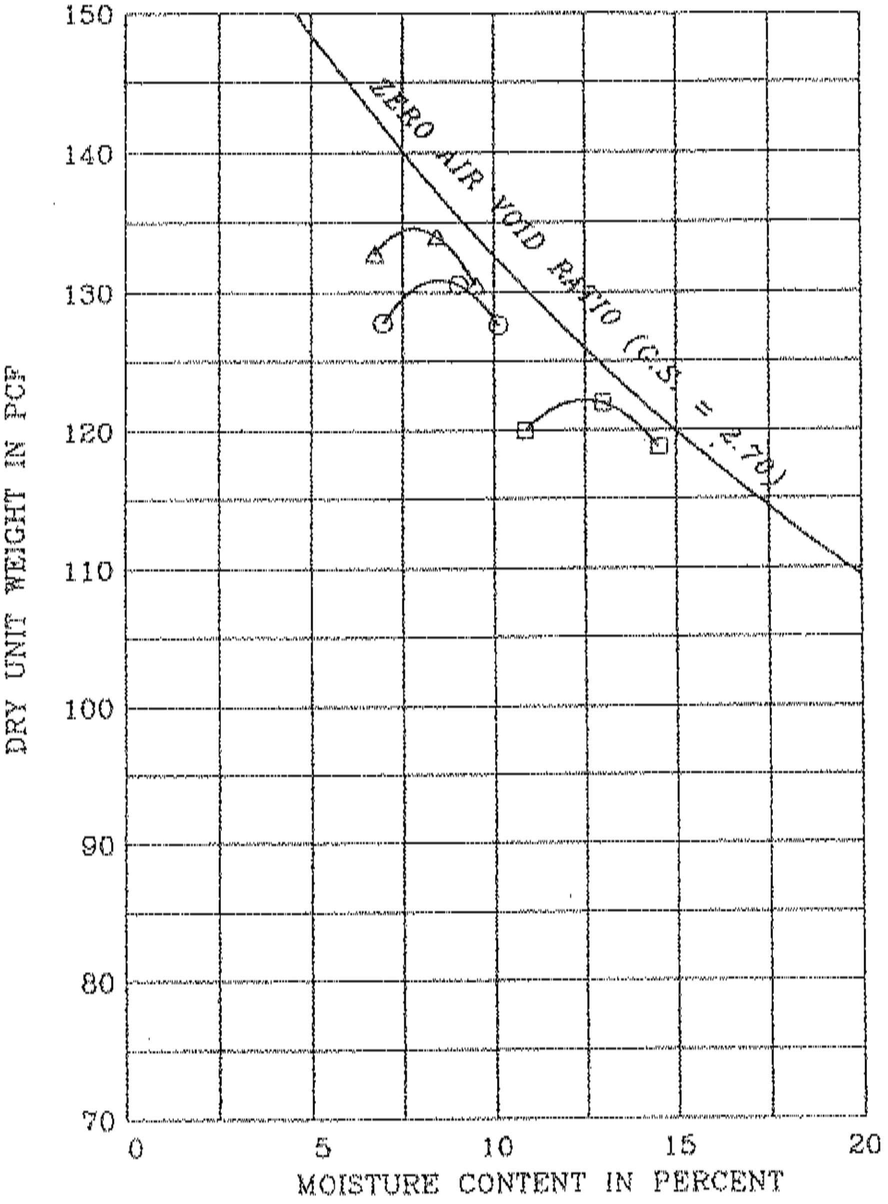
SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
S-2 @ 0 - 1'	Remolded - Saturated	Silty Sand (SM)

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

DIRECT SHEAR

Job No: 1566.00

Plate No: C-1



SYMBOL	SAMPLE LOCATION	DEPTH (ft)	DESCRIPTION	TEST METHOD	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
○	Test Pit 1	3.0	Brn si f-c sand w/gvl SM	ASTM D 1557	8.4	130.8
□	Test Pit 5	3.0	Gray si f-c sand w/gvl SM	ASTM D 1557	12.6	122.2
△	Test Pit 5	5.5	Brn si f-c sand w/gvl SM	ASTM D 1557	7.9	134.6

Remark :

File No.: S-8913

Canyon Crest Courtyard, Riverside, CA

John R. Byerly
INCORPORATED

COMPACTION TEST

Enclosure 3
Rpt. No.: 5065

APPENDIX H

SEISMIC SHEAR WAVE SURVEY REPORT

TERRA
GEOSCIENCES

SEISMIC SHEAR-WAVE SURVEY

CRESTVIEW APARTMENT PROJECT

NWC OF CENTRAL AVE. AND SYCAMORE CANYON BLVD.

CITY OF RIVERSIDE, CALIFORNIA

Project No. 203480-2

August 24, 2020

Prepared for:

NOVA Services, Inc.
944 Calle Amanecer, Suite F
San Clemente, CA 92672

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

NOVA Services, Inc.
944 Calle Amanecer, Suite F
San Clemente, CA 92673

Attention: Ms. Chelsea Jaeger, Project Geologist

Regarding: Seismic Shear-Wave Survey
Crestview Apartment Project
NWC of Central Ave. & Sycamore Canyon Blvd.
City of Riverside, California
NOVA Project No. 3020003

INTRODUCTION

As requested, this firm has performed a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods for the above-referenced site. The purpose of this survey was to assess the one-dimensional average shear-wave velocity structure, at various depth intervals, beneath the subject survey area, to a depth of at least 100 feet. Geologic mapping Morton and Cox (2001), indicate the subject property to underlain by Cretaceous age granitic rocks, which consists predominantly of gray-weathering, relatively homogeneous, massive to well-foliated, medium- to coarse-grained, biotite-hornblende tonalite, locally referred to as the Val Verde Tonalite.

The location of the seismic traverse has been approximated on a captured Google™ Earth image (Google™ Earth, 2020), which is presented as the Seismic Line Location Map, Plate 1, for reference. Additionally, photographic views of the survey line are presented on Plate 2, for visual and reference purposes. As authorized by you, the following services were performed during this study:

- **Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site.**
- **Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.**
- **Preparation of this report, presenting the results of our findings with respect to the shear-wave velocities of the subsurface earth materials.**

Accompanying Map, Illustrations, and Appendices

Plate 1 - Seismic Line Location Map
Plate 2 - Survey Line Photographs
Appendix A - Shear-Wave Model and Data
Appendix B - References

SUMMARY OF SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse (Seismic Line SW-1) was performed across the southern portion of the site, as selected by you, as approximated on the Seismic Line Location Map, Plate 1. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph (Geometrics, 2004). This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM surveys used the same linear geometry array that consisted of a 184-foot long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals. For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 30-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple hammer impacts (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of any artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2016). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within Appendix A.

Summary of Data Analysis

Data acquisition went very smoothly and the quality was considered to be very good. The seismic model data indicates that the average shear-wave velocity beneath the survey traverse has numerous velocity layers that initially increase in velocity with depth, with a velocity reversal beginning around a depth of 130± feet. This velocity reversal does not affect the average V_{100} seismic velocity. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet (V_{100}) of the subject survey area is **2,109.1** feet per second as shown on the Shear-Wave Model for Seismic Line SW-1, as presented within Appendix A. This average velocity classifies the underlying soils to that of Site Class "**C**" ("Very Dense Soil and Soft Rock"), which has a velocity range from 1,200 to 2,500 ft/sec (ASCE, 2017; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100).

$$\mathbf{Vs = 100/[(d1/v1) + (d2/v2) + ...+ (dn/vn)]}$$

Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 218-foot profile where locally measured. The constrained data is represented by the dark-gray shading on the shear-wave model. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within this appendix, for reference purposes.

CLOSURE

The field survey was performed by the undersigned on August 19, 2020, using "state of the art" geophysical equipment and techniques along the selected portion of the subject study area as directed by you. It is important to note that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

It should be noted that when compared with traditional borehole shear-wave surveys, which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent. Client should understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and, in the interpretation, and that the results of this survey may not represent actual subsurface conditions.

These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied. If the client does not understand the limitations of this geophysical survey, additional input should be sought from the consultant.

Respectfully submitted,
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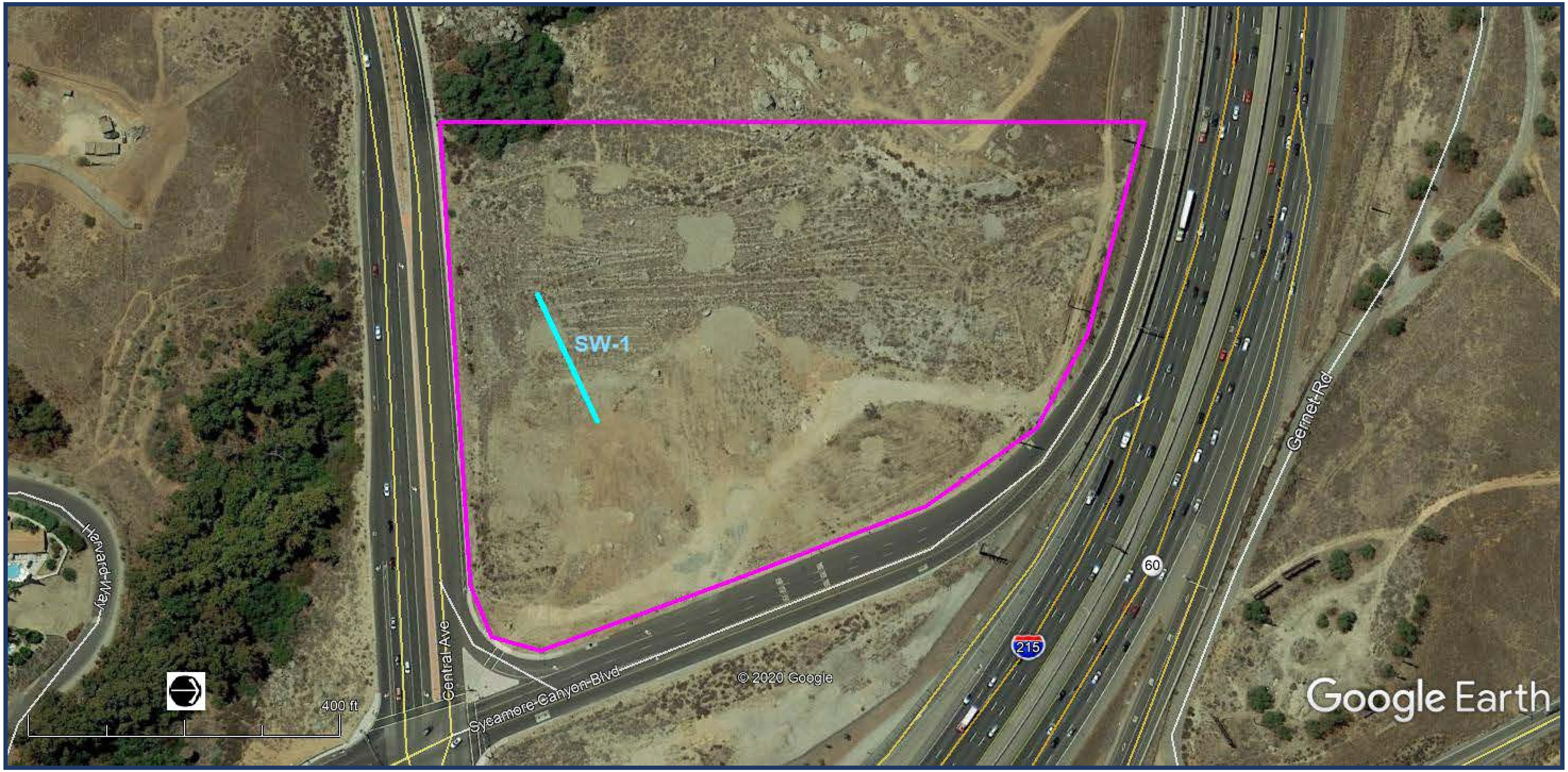
SEISMIC LINE LOCATION MAP



Base map from Google™ Earth imagery (2020); Seismic shear-wave traverse SW-1 shown as blue line.

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PLATE 1

SURVEY LINE PHOTOGRAPHS

View looking northeasterly along Seismic Line SW-1.

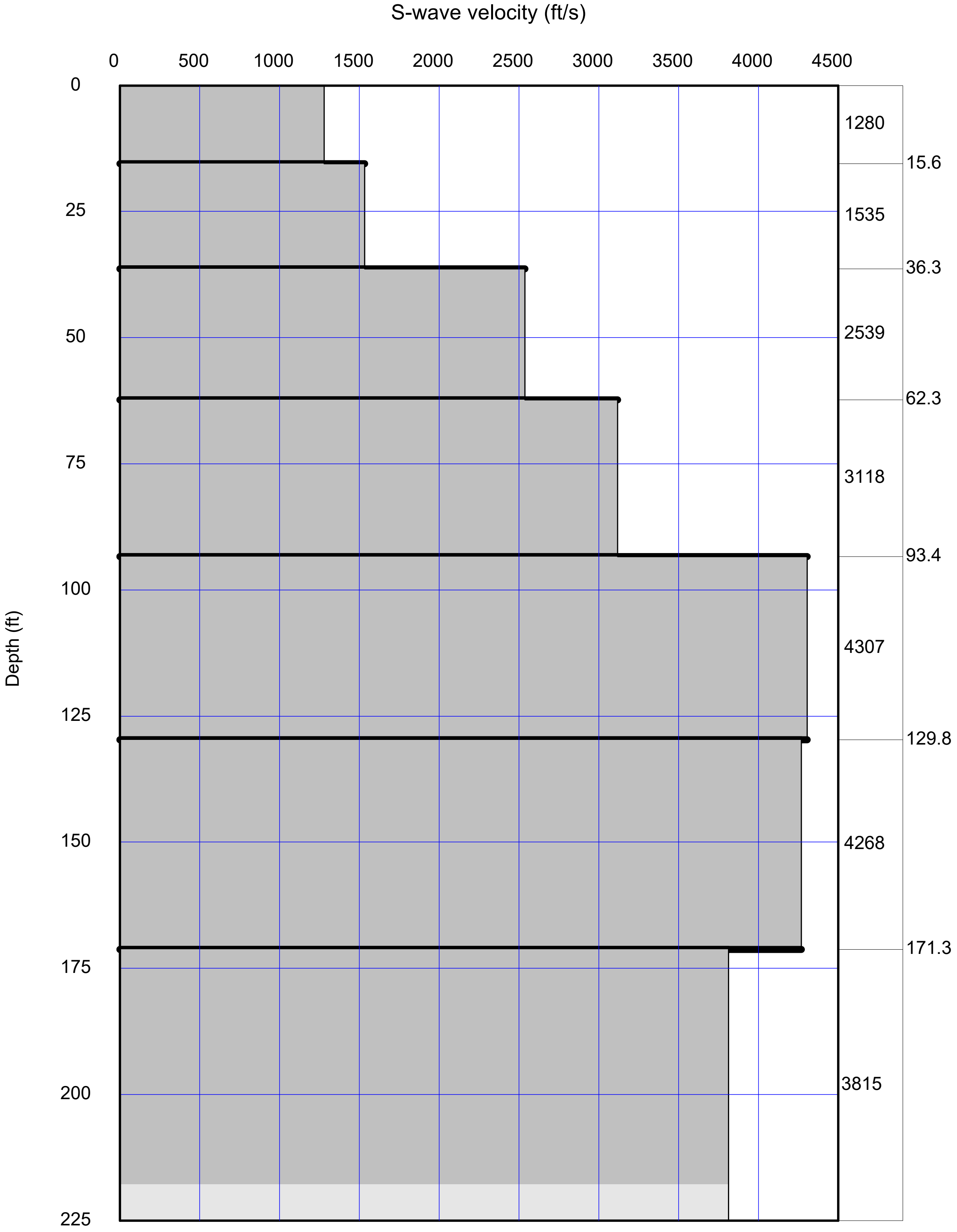
View looking southwesterly along Seismic Line SW-1.

APPENDIX A

SHEAR-WAVE MODEL AND DATA

SEISMIC LINE SW-1

SHEAR-WAVE MODEL

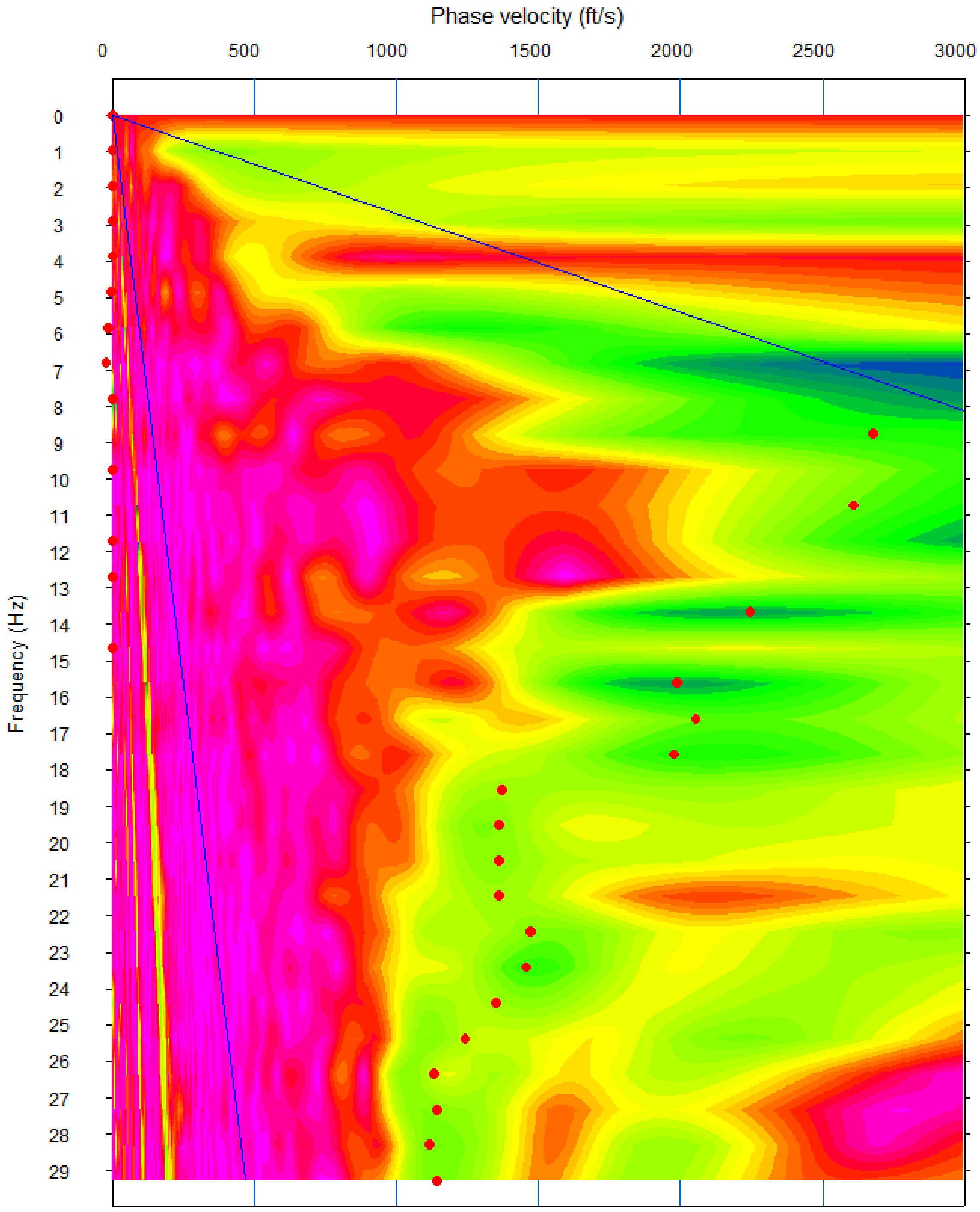


S-wave velocity model : Final.rst **Average Vs 100ft = 2109.1 ft/sec**

SHEAR-WAVE MODEL SW-1

COMBINED DISPERSION CURVE

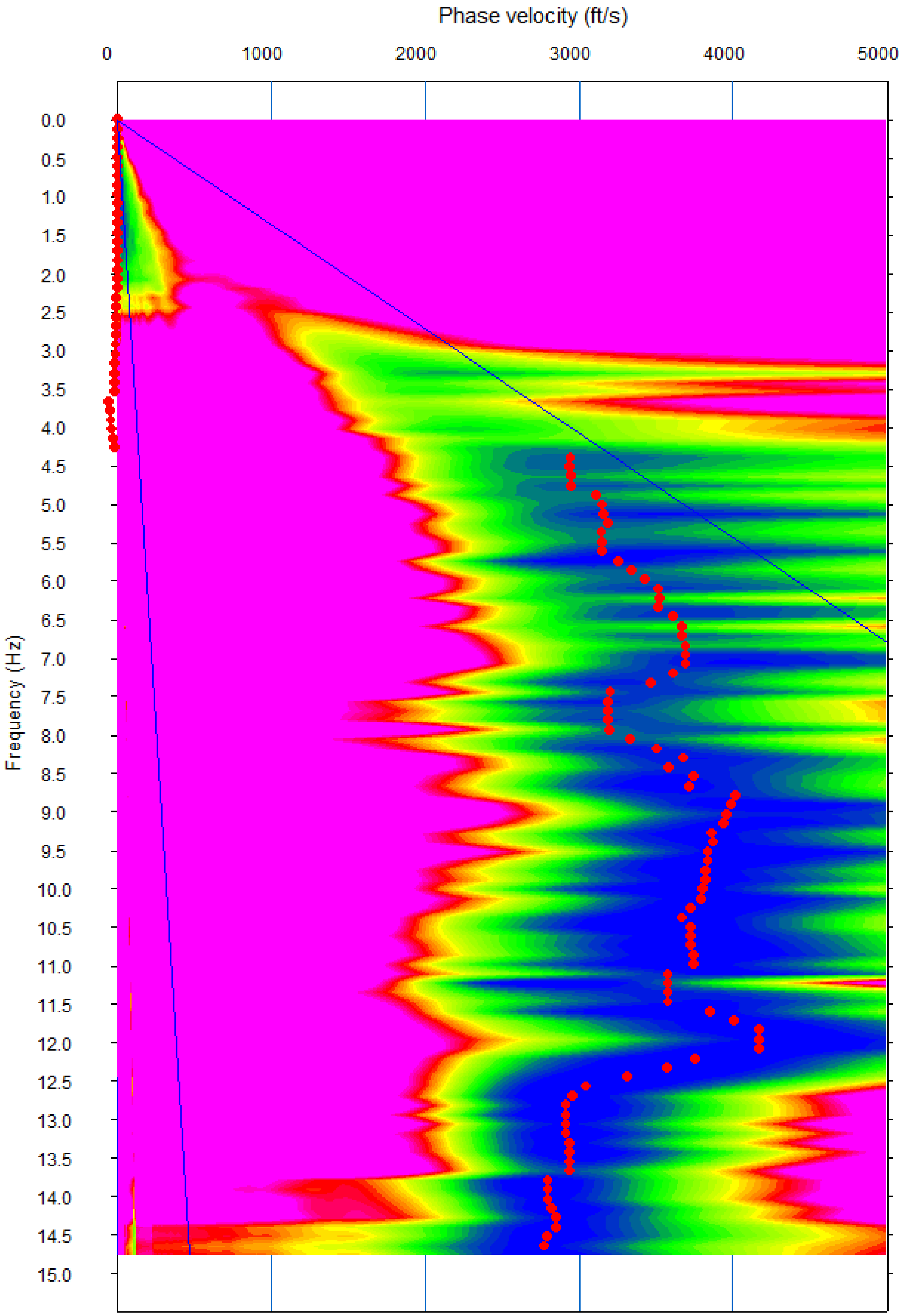
SEISMIC LINE SW-1



Dispersion Curve: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

APPENDIX B

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REFERENCES

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